SECTION 5.3 OF THIS REPORT DETAILING THE PROPOSED PAVEMENT DESIGN HAS BEEN REPLACED BY THE PLANS AND TECHNICAL MEMORANDUM PREPARED BY ATKINS DATED JUNE 12, 2023.

GEOTECHNICAL INVESTIGATION

HOUSTON AIRPORT SYSTEM
INTERSECTION IMPROVEMENTS FOR STANDIFER ROAD AND LEE ROAD
HOUSTON, TEXAS

Reported to

Atkins North America, Inc.
Houston, Texas

by

Aviles Engineering Corporation
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REPORT NO. G148-19

February 2020

AVILES
ENGINEERING CORP.

February 7, 2020

Mr. Edmond Woods, P.E.
Atkins North America, Inc.
17220 Katy Freeway, Suite 200
Houston, Texas 77094

Reference: Geotechnical Investigation
Houston Airport System
Intersection Improvements for Standifer Road and Lee Road
Houston, Texas
AEC Report No. G148-19

Dear Mr. Woods,

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 by Mr. John Verburg, P.E., Senior Technical Manager of Atkins North America, Inc. (Atkins) on October 2, 2019. Project terms and conditions were in accordance with Task Order No. 1 and the Master Subcontract Agreement between Atkins and AEC, dated June 24, 2019. The project scope of services is in accordance with AEC Proposal No. G2019-06-01, dated June 5, 2019

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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GEOTECHNICAL INVESTIGATION

HOUSTON AIRPORT SYSTEM
INTERSECTION IMPROVEMENTS FOR STANDIFER ROAD AND LEE ROAD
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 Project Description

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for a Houston Airport System (HAS) project which includes evaluation of a sinkhole that is currently located at the southeast corner of the intersection of Standifer Road and Lee Road, east of Bush Intercontinental Airport (BIAH) in Houston, Texas (Houston/Harris County Key Map No.: 334Z). A vicinity map is presented on Plate A-1, in Appendix A.

Based on AEC's site visit on June 4, 2019, there is an approximately 5 foot wide by 8 foot long sinkhole along the north side of an existing abandoned building that is located at the southeast corner of the intersection of Standifer Road and Lee Road. The sinkhole is approximately 8 feet deep, and currently extends beneath the building, as well as some of the curb and asphalt pavement of Standifer Road. Several plastic/PVC pipes have been exposed within the sinkhole. The sinkhole area is currently blocked from roadway traffic by plastic barricades. After the sinkhole is repaired, the asphalt pavement at the entrance of Standifer Road will be reconstructed.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil and groundwater conditions at the project site and determine possible cause(s) of the sinkhole, and provide recommendations for remediation measures of the sinkhole and backfilling it, as well as design and construction of pavement at the entrance of Standifer Road. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling one soil boring to 30 feet below existing grade adjacent to the sinkhole;
2. Performing soil laboratory testing on selected soil samples;
3. Evaluation of potential cause(s) for the sinkhole;
4. Engineering analyses and recommendations for reconstruction of pavement, including pavement thickness design and subgrade preparation for flexible pavement;
5. Construction recommendations for the repair/remediation of the sinkhole and pavement.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling one soil boring to a depth of 30 feet, adjacent to the sinkhole. Boring location was selected by Atkins and then marked in the field by AEC personnel. The total drilling footage of the boring is 30 feet. The boring location is presented on Boring Location Plan on Plate A-2, in Appendix A. After completion of drilling, the boring location was surveyed by Landtech, Inc. Boring survey data (in State Plane Grid Coordinates, Texas South Central Zone, US Survey Feet) is presented on the representative boring log (see Plate A-3, in Appendix A).

The boring was drilled using a truck-mounted drill rig and was initially advanced using dry auger method, and then using wet rotary method once groundwater was encountered. Undisturbed samples of cohesive soils were obtained from the boring 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 log. 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 obtained during drilling and upon completion of drilling. After completion of drilling, further groundwater readings were obtained up to 30 minutes at 5 minute intervals. After the final groundwater readings were obtained, the borehole was backfilled with bentonite chips and existing pavement surface was patched with cold-mix asphalt patch. Details of the soils encountered in our boring are presented on Plate A-3, in Appendix A.

3.0 LABORATORY TESTING

Soil laboratory testing was performed by AEC personnel. Samples from the boring 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, and dry unit weight tests were performed on selected samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were estimated by means of torvane (TV), unconfined compression (UC), and unconsolidated-undrained (UU) triaxial tests performed on undisturbed samples. The test results are presented on the representative boring log. A key to the boring log, classification of soils for engineering purposes, terms used on boring log, and reference ASTM Standards for laboratory testing are presented on Plates A-4 through A-7, in Appendix A.

4.0 SITE CONDITIONS

4.1 Subsurface Conditions

Details of the soils encountered during drilling are presented in the boring log (see Plate A-3, in Appendix A). Soil strata encountered in our boring are summarized below. At the time of drilling, no void was detected beneath the pavement surface at the discrete boring location.

Boring	Depth (ft)	Description of Stratum
B-1	0 - 0.7	Pavement: 8” asphalt
	0.7 - 1.9	Base: 14.8” cement-stabilized sand with gravel
	1.9 - 4	Stiff to very stiff, Sandy Lean Clay (CL)
	4 - 12	Firm to very stiff, Lean Clay with Sand (CL), with silty sand partings
	12 - 14	Medium dense, Silty Sand (SM), with lean clay pockets, wet
	14 - 27	Soft to very stiff, Fat Clay (CH), with slickensides
	27 - 29.8	Soft, Lean Clay with Sand (CL), with silty clay partings
	29.8 - 30	Silty Sand (SM), wet

Subsurface Soil Properties: The cohesive soils encountered in Boring B-1 have slight to very high plasticity (see Plate A-5, in Appendix A), with Liquid Limits (LL) ranging from 27 to 74, and Plasticity Indices (PI) ranging from 9 to 47. The cohesive soils encountered are classified as “CL” and “CH” type soils and the granular soils are classified as “SM” type soils in accordance with ASTM D 2487. “CH” soils undergo significant volume changes due to seasonal changes in soil moisture contents. “CL” soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content.

Groundwater: Groundwater levels and boring cave-in depths encountered in Boring B-1 are summarized in Table 1.

Table 1. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth (ft)	Boring Cave-in Depth (ft)
B-1	08/21/19	30	14 (Drilling) 9.1 (15 min.) 5.2 (Complete) 8.1 (30 min. after complete)	10.8 (Drilling) 26.8 (Complete)

The information in this report summarizes conditions found on the date the boring was drilled. However, 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. However, AEC notes that the presence of potential hazardous material at other locations within the project area cannot be discounted based upon the very small and limited number of samples taken.

4.3 Geologic Conditions

As requested by Landtech, Inc., AEC performed a preliminary desktop fault study without field observations, which included a review of public maps, available literature, and aerial photographs. According to the published maps "*Principal Active Faults of the Houston Area (after O'Neill and Van Siclen, May 1984)*", and "*Principal Surface Faults in the Central Houston Metropolitan Area (after O' Neill, Van Siclen, with additions by C. Norman, May 13, 2004)*", no documented faults are located in the project area. The closest fault to the project area is the Jetero Fault which crosses Lee Road approximately 0.8 of a mile south of the intersection of Lee Road and Standifer Road. The Lee Fault crosses Lee Road approximately 1.7 miles south of the intersection of Lee Road and Standifer Road. The search of available literature did not reveal any faults located at or near the project site. A series of aerial photographs from 1944 to 2019 were reviewed on Google Earth on the internet. No evidences of faults in or near the project site were observed.

AEC does not recommend any further fault studies be performed for the project area.

Limitations: The preliminary fault investigation provided in this report is limited to a review of available literature, aerial photographs, and maps. Distances are scaled from maps. Faults may exist in, cross, or adjoin the project site which were not identified in this report due to the following reasons: limitations of the scope of work and cost, no field observations were conducted; lack of documentation in the literature; and faults may have not been visible on the aerial photographs due to clarity of the aerial photographs, the presence of vegetation and environmental features, and modification of the land surface by human activities. Faults may also be present below ground but do not currently have surface expressions. Identification of these faults is beyond the scope of work for this study.

4.4 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 the boring location.

Clay soils in the Greater Houston area typically have secondary features such as slickensides, calcareous/ferrous nodules, and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring log is based on 3-inch diameter soil samples which were generally obtained from the boring at intervals of 2 feet continuously from the ground surface to a depth of 20 feet below grade, then at 5 foot intervals thereafter to the boring termination depth of 30 feet below existing grade. 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 AEC's log shows some soil secondary features, it should not be assumed that the features are absent where not indicated on the log.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

Based on AEC's site visit on June 4, 2019, there is an approximately 5 foot wide by 8 foot long sinkhole along the north side of an existing abandoned building that is located at the southeast corner of the intersection of Standifer Road and Lee Road. The sinkhole is approximately 8 feet deep, and currently extends beneath the building, as well as some of the curb and asphalt pavement of Standifer Road. Several plastic/PVC pipes have been exposed within the sinkhole. The sinkhole area is currently blocked from roadway traffic by plastic barricades. After the sinkhole is repaired, the asphalt pavement at the entrance of Standifer Road will be reconstructed.

AEC understands that the existing abandoned building adjacent to the sinkhole at the project site will be demolished and additional borings will be performed in the future (by AEC) for demolition of the building. Further recommendations for demolition of the building and backfill of the sinkhole beneath the building will be provided after the additional borings are performed by AEC.

5.1 Potential Reasons for Sinkhole

Soil and Groundwater Conditions near Sinkhole: The soils encountered in Boring B-1 generally consist of soft to very stiff lean/fat clay (CL/CH). Approximately 2 feet of medium dense silty sand (SM) was encountered at a depth of 12 to 14 feet in Boring B-1. A thin layer of silty sand (SM) was encountered near the bottom of our boring at approximately 29.8 feet below existing grade. Based on Table 1 in Section 4.1 of this report, groundwater was encountered at a depth of 14 feet below existing grade during drilling in our boring, and subsequently rose to a depth of 9.1 feet approximately 15 minutes after initial encounter. Groundwater was measured at 5.2 feet below grade after completion of drilling in our boring. Groundwater was observed at a depth of 8.1 feet approximately 30 minutes after completion of drilling. AEC also notes that the borehole caved-in to a depth of 10.8 feet during drilling.

Potential Reasons for Sinkhole Formation: It is AEC's opinion that the most likely cause for the formation of sinkhole is leakage from existing underground pipes (such as waterlines, irrigation lines, sanitary sewers, or storm sewers) or manholes at or near the project site. Over time, the leaking water from damaged underground utilities first erodes and carries away subsurface soils (especially for sanitary sewers or storm sewers); granular soils, such as the silty sand (SM) strata encountered at a depth of 12 to 14 feet in Boring B-1 are especially vulnerable to this type of erosion, although clay soil strata can also experience erosion and loss over time. These granular soils will ultimately erode and carry away the overlaying cohesive materials, causing a sinkhole to form. Although less likely to occur, surface runoff and ponding water could also cause erosion of surface soils and formation of a sinkhole over time. Drawings provided by Atkins indicate that numerous underground utilities are located in the immediate vicinity of the sinkhole, including waterlines, sanitary sewers, and storm sewers. Several utilities are shown to cross through the sinkhole itself (as described above, these broken pipes were noted by AEC in the field).

5.2 Remediation Measures for Sinkhole

Since leaking underground pipes is the most likely culprit for the sinkhole formation, AEC recommends that CCTV surveys be performed within nearby storm sewers, sanitary sewers, and manholes to determine if there are leaks or breaks in existing pipes, manholes, or joints between pipes and manholes, including utilities that may be present beneath the adjacent abandoned building. Leaking waterlines are likely to present themselves in the form of seeps or the presence of soft/wet soil at the ground surface. **Damaged underground utilities (if any) must be found and properly repaired/replaced prior to remediation of the sinkhole.** Otherwise, the sinkhole is likely to reform even after it is backfilled. Care must be taken to ensure proper installation and compaction of bedding, haunching, and backfill of repaired/replaced underground utilities, otherwise the utilities can settle and break once again, repeating the sinkhole formation cycle. After any leaking/damaged underground utilities are repaired/replaced, then the sinkhole can be backfilled.

The sinkhole area should be over-excavated by a minimum of 2 feet to remove any loose/soft soils that remain. Since the majority of the sinkhole extends beneath the existing asphalt pavement, AEC recommends the sinkhole be properly backfilled with compacted quality select clay fill materials. Once the fill material is compacted, existing asphalt pavement should be restored to its original thickness and grade, where required. Select clay fill recommendations are presented in Section 5.4.1 of this report.

5.2.1 <u>Excavation and Backfilling</u>

Once leaking or damaged underground utilities are repaired or replaced, remove existing debris and broken pipes from the sinkhole. Afterwards, over-excavate a minimum of 2 feet below the bottom of the sinkhole to remove any disturbed/weak/soft/wet soils. If competent soils are still not exposed after the 2 feet of over-excavation, then the excavation depth should be increased incrementally by 6 inches until competent soils are exposed. After over-excavation, the sinkhole can be backfilled with compacted select clay fill. Select clay fill recommendations are presented in Section 5.4.1 of this report.

Cohesive soils in the Houston area contain many secondary features which affect trench stability, including calcareous/ferrous nodules, sand/silt seams/pockets, and slickensides. Slickensides are shiny weak failure planes which are commonly present in high plasticity 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 our boring log.

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

Excavations Less than 20 Feet Deep: 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 should be in accordance with OSHA Safety and Health Regulations, 29 CFR, Part 1926. Stiff to hard clays should be considered OSHA Class "B" soils, while fill soils, granular soils, and very soft to firm clays should be considered OSHA Class "C" soils. Submerged clay soils should also be classified as OSHA Class "C" soils, unless dewatering is conducted to lower the groundwater level below the excavation bottom. OSHA classification of soils encountered in the top 20 feet of Boring B-1 is presented on Plate B-1, in Appendix B.

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. AEC conservatively recommends a factor of safety (FS) of 2 be applied to the determination of critical height; as a result, 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 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 (1). The design soil parameters for trench bracing design are presented on Plate C-1, in Appendix C. AEC recommends that trench bracing design first consider short term soil conditions and then consider long term soil conditions. Whichever soil condition results in a more conservative trench bracing design should then be used, regardless of the actual time the shoring will remain in place during construction.

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

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 for long term soil conditions;
- 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. Struts in mixed soil (i.e. sand and clay) conditions should be based on whichever soil condition (either sand or clay) results in more conservative shoring design.

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.

If the excavation extends below groundwater, and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the excavation bottom. Groundwater control recommendations are presented in Section 6.2 of this report.

Secondary Features: Calcareous/ferrous nodules, silt pockets, and fat clays with slickensides were encountered in our boring. These secondary structures may become sources of localized instability when they are exposed during excavation, especially when they become saturated. AEC notes that soils with secondary features 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.

Backfill Material: After the sinkhole has been over-excavated and competent soils have been exposed, backfill the sinkhole with compacted select clay fill. Select clay fill recommendations are presented in Section 5.4.1 of this report.

5.3 Pavement

Based on drawings provided by Atkins, dated October 11, 2019, the entrance of the Standifer Road will be reconstructed with asphalt pavement after the sinkhole is repaired. AEC understands that the pavement reconstruction will be based on a new design, instead of matching the thickness of the existing roadway pavement section. AEC anticipates that the new pavement will be placed at or near existing grade. Traffic data was not available at the time this report was prepared, but is anticipated to be light to moderate since Standifer Road is a dead end and only appears to service a few industrial properties.

AEC recommends that the thickness for the replacement pavement exceeds the minimum thickness required by Chapter 10 Section 10.04 of the latest edition of the City of Houston Infrastructure Design Manual (COH IDM) and City of Houston Standard Detail 02741-01.

5.3.1 Flexible Pavement

Pavement Design: Flexible pavement design procedure includes determination of the structural number (SN) for the proposed pavement, as well as the thickness of individual components of the surface course, base course, and subgrade. The basic equation developed by the AASHTO Road Test is:

$$SN = a_1(D_1) + a_2(D_2) + a_3(D_3) \qquad \text{...........Equation (2)}$$

where: SN = Structural Number for the total flexible pavement structure.
a_1, a_2, a_3 = layer coefficients for surface, base and subgrade course respectively.
D_1, D_2, D_3 = thickness of surface, base and subgrade course, respectively, in inches.

Layer coefficients used for design are presented on Table 2.

Table 2. Layer Coefficients for Asphalt Pavements

Pavement Layer	Layer Coefficient
Hot Mix Asphaltic Concrete	a1 = 0.44
Asphalt-stabilized Base	a2 = 0.34
Lime-stabilized Subgrade	a3 = 0.11

The parameters that were used in computing the flexible pavement section are as follows:

Roadbed Soil Resilient Modulus (M_R)	4,500 psi
Drainage Coefficient (C_d)	1.0
Overall Standard Deviation (S_0)	0.45
Reliability Level (R)	85%
Initial Serviceability (P_0)	4.2
Terminal Serviceability (P_t)	2.0
Design Life	20 years

The recommended flexible pavement sections are provided on Table 3 below.

Table 3. Recommended Flexible Pavement Section for Standifer Road Entrance

Pavement Layer	Thickness (in)
Hot Mix Asphaltic Concrete Thickness (in)	2
Asphalt-stabilized Base Thickness (in)	6
Lime-stabilized Subgrade Thickness (in)	8[(a)]
Structural Number (SN)	3.8

Pavement Layer	Thickness (in)
Pavement Section 18-kip ESAL Load Capacity	981,123

Note: (a) Minimum thickness required for subgrade where cohesive soils are present, according to Section 10.04.C.2 of the latest edition of the COH IDM.

AEC used the AASHTO Darwin v3.0 computer program to perform flexible pavement design. Given the above design parameters, the asphalt pavement section proposed in Table 3 should sustain 981,123 repetitions of 18-kip ESALs (see Table 3). The design engineer should verify whether the proposed pavement section will provide enough ESALs for the anticipated amount of site traffic. AEC should be notified immediately if different standards or constants are required for pavement design at the site, so that our recommendations can be updated accordingly.

Asphalt Pavement: Hot Mix Asphaltic Concrete (HMAC) pavement should be constructed in accordance with Section 02741 of the latest edition of the COHSCS. Tack coat should be in accordance with Section 02743 of the latest edition of the COHSCS.

Hot Mix Asphalt Base Course: Hot mix asphalt base (i.e. "Black Base") course shall be in accordance with Section 02711 of the latest edition of the COHSCS.

Prime Coat: The surface of the compacted base should be primed in accordance with Section 02742 of the latest edition of the COHSCS.

5.3.2 Pavement Subgrade Preparation

AEC assumes that new pavement will be at or near existing grade. Based on Boring B-1, surficial soils in the project area primarily consist of lean clay (CL) soils with medium expansive potential. Based on the subgrade conditions, AEC recommends that the pavement subgrade be stabilized with hydrated lime.

Existing pavement and base should be demolished in accordance with Section 02221 of the latest edition of the COHSCS. Where possible, subgrade preparation should extend a minimum of 2 feet beyond the paved area perimeters. After demolition of existing pavement and base, we recommend that a competent soil technician inspect the exposed subgrade to determine if there are any unsuitable soils or other deleterious materials. Excavate and dispose of unsuitable soils and other deleterious materials which will not consolidate; the excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths. Unsuitable soil is defined in Section 02319 of the latest edition of the COHSCS. The exposed soils should be proof-rolled to identify and remove any weak, compressible, or other unsuitable materials; such over-excavations should be backfilled in general accordance with Section 02315 of the latest edition of the COHSCS. Proof rolling should be performed with a pneumatic tire roller (or using equivalent compaction equipment), with a loaded weight between 25 and 50 tons. At least two coverages should be made with the proof-roller, and offset each trip of the roller by at most 1 tire width. Rollers should make passes at a speed between 2 and 6 miles per hour.

After proof rolling, scarify the exposed subgrade to a depth of 8 inches and stabilize with a minimum of 5 percent hydrated lime (by dry soil weight). Lime stabilization shall be performed in accordance with Section 02336 of the latest edition of the COHSCS. The stabilized soils should be compacted to 95 percent of their ASTM D 698 (Standard Proctor) dry density at a moisture content ranging from optimum to 3 percent above optimum.

5.4 Fill Requirements

5.4.1 Select Clay Fill

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

Precautions: Prior to construction, the Contractor should determine if they can obtain qualified select clay fill meeting the below select clay fill criteria. The closest sand and clay pit to the project site may not be able to deliver fill material that meets the requirements below. The Contractor should also be aware that testing of select clay fill (see below) typically takes a minimum of 1.5 days to complete and they should accommodate testing in their fill placement in their project schedule. In addition, imported fill that is delivered to the project site may vary from day to day; material delivered to the site may pass one day but fail the next.

Select Clay Fill Requirements: Select clay fill (whether imported from offsite or excavated onsite) should consist of *uniform*, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Any clay soil intended for use as select clay fill (whether imported from offsite or excavated onsite) shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Sand and clay mixtures/blends are unacceptable for use as select clay fill. Sand/silt with clay clods is unacceptable for use as select clay fill. Mixing sand into clay or mixing clay into sand/silt is also unacceptable for use as select clay fill. **The testing lab shall *reject* any material intended for use as select clay fill that does not meet the PI, sieve, and clay clod requirements above, without exceptions.**

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

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

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

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation and Grading

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 site, requiring further evaluation and consideration of the excess hydrostatic pressures. Groundwater control should be in general accordance with Section 01578 of the latest edition of the City of Houston Standard General Requirement (COHSGR).

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.

Groundwater control methods typically can be classified into three categories: (i) open pumping, where water is allowed to flow into an excavation and is collected in ditches or sumps and pumped away; (ii) predrainage, where the water table is lowered before excavation using wellpoints, ejector/eductor systems, deep wells, etc.; and (iii) cut off or exclusion, where the groundwater is prevented from entering the excavation by an impermeable barrier, such as by sheet piling, grouting, deep soil mixing, ground freezing, slurry shields, etc.

<u>Cohesive Soils:</u> Groundwater control in cohesive soils can typically be performed using open pumping methods. Seepage rates are lower than in granular soils and groundwater is usually collected in sumps and/or channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

<u>Granular Soils:</u> Groundwater control in granular soils will typically require predrainage methods or cutoff/exclusion methods. For excavations that are less than 15 feet deep that will occur within saturated sands, a predrainage method such as wellpoints can be considered. For excavations that are greater than 15 feet deep,

other predrainage methods that can be considered include multiple staged wellpoints, ejectors/eductors (primarily for use when silty soils are present), or deep wells with submersible pumps. Generally with predrainage methods, the groundwater depth should 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.

If predrainage methods cannot be used, then a cutoff/exclusion method such as interlocking water-tight sheet piles, drilled shaft/secant pile wall (with grout between the shafts/piles), or jet grouting of the granular strata may be necessary.

Extended Dewatering: Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity of the dewatering operations; 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 them in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

Bottom Heave or Boiling: For excavation 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. If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the excavation.

6.3 Construction Monitoring

Site preparation (including clearing and proof-rolling) and earthwork operations should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered.

6.4 Monitoring of Existing Structures

Existing structures in the vicinity of the project site 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 project site. We therefore recommend that the Contractor be required to survey and adequately document the condition of existing structures in the vicinity of the project site.

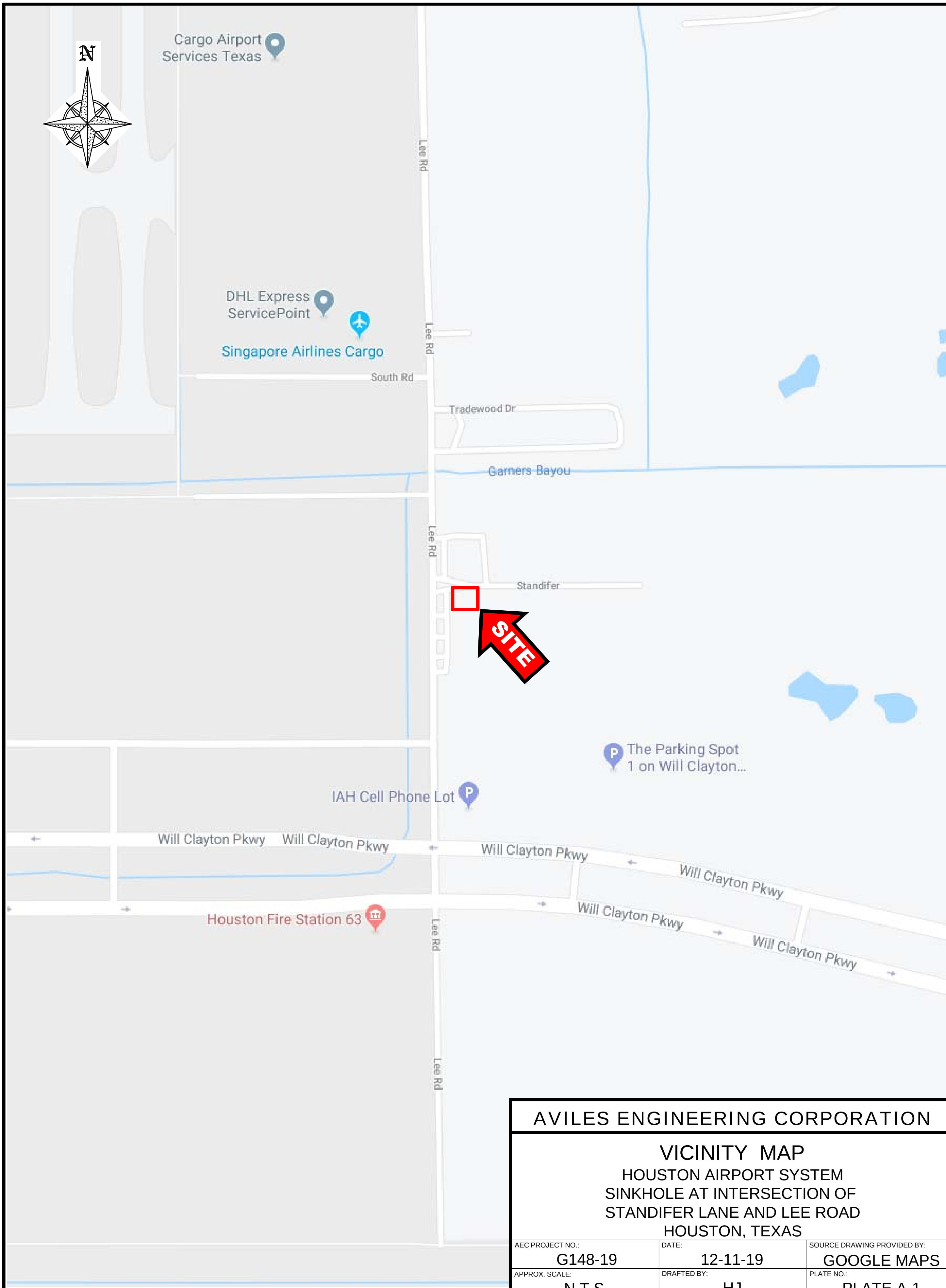
7.0 GENERAL

The information contained in this report summarizes conditions found on the date the boring was drilled. The attached boring log is a true representation of the soils encountered at the specific boring location on the date of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. AEC should be notified immediately when conditions encountered during construction are significantly different from those presented in this report.

8.0 LIMITATIONS

The 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. The report has been prepared exclusively for the project and location described in this report, and is intended to be used in its entirety. 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 scope of services does not include a fault investigation. The recommendations presented in this report should not be used for other structures located at this site or similar structures located at other sites, without additional evaluation and/or investigation.

APPENDIX A



AVILES ENGINEERING CORPORATION

VICINITY MAP

HOUSTON AIRPORT SYSTEM
SINKHOLE AT INTERSECTION OF
STANDIFER LANE AND LEE ROAD
HOUSTON, TEXAS

AEC PROJECT NO.: G148-19	DATE: 12-11-19	SOURCE DRAWING PROVIDED BY: GOOGLE MAPS
APPROX. SCALE: N.T.S.	DRAFTED BY: HJ	PLATE NO.: PLATE A-1

Notes:

1) As directed by Atkins, Boring B-1 was performed on Standifer Lane. Additional borings will be performed in future for demolition of building adjacent to sinkhole.

2) Boring location has been surveyed by Landtech, Inc.

LEGEND

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

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

HOUSTON AIRPORT SYSTEM
SINKHOLE AT INTERSECTION OF
STANDIFER LANE AND LEE ROAD
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G148-19	12-10-19	GOOGLE EARTH
APPROX. SCALE: 1" = 60'	DRAFTED BY: WLW/HJ	PLATE NO.: PLATE A-2

PROJECT: **Sinkhole at Lee Road and Standifer Drive**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-1**

DATE **08/21/2019** TYPE **4" Dry Auger/Wet Rotary** LOCATION **See Boring Location Plan**

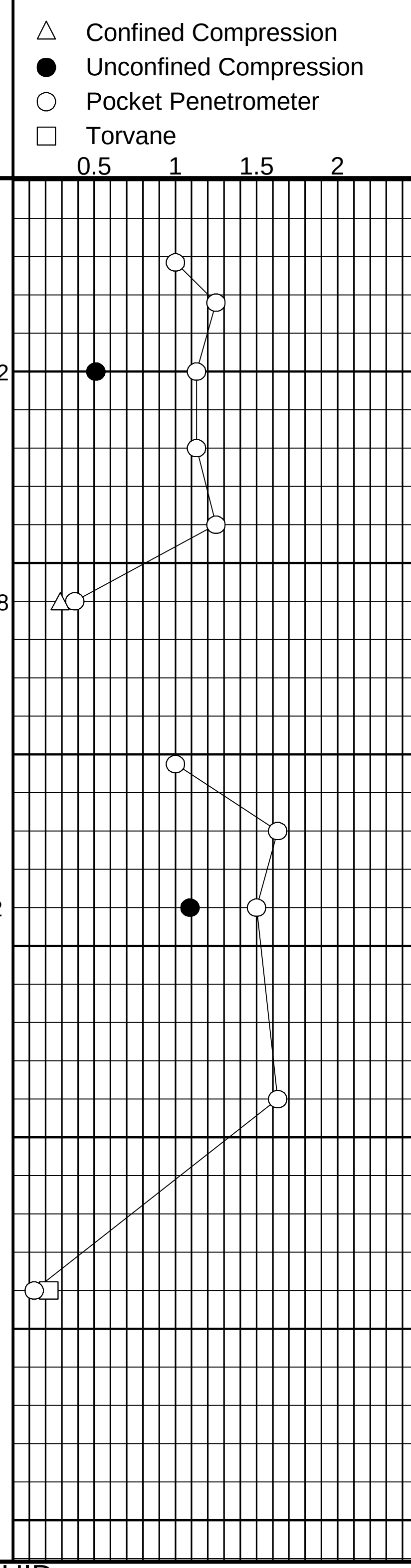
DESCRIPTION

GRID Coordinates (US Survey ft):
Texas State Plane Zone: 4204
Easting: 3137015.43
Northing: 13925501.23
Elevation: 81.39

DEPTH IN FEET	DESCRIPTION
0	Pavement: 8" asphalt
	Base: 14.8" cement-stabilized sand with gravel
	Stiff to very stiff, grayish tan Sandy Lean Clay (CL)
5	Firm to very stiff, gray and tan Lean Clay with Sand (CL), with silty sand partings -with ferrous stains and nodules 4'-8' -with ferrous seams 8'-10' and silty clayey sand pockets 8'-12' -groundwater at 8.1' approximately 30 minutes after completion of drilling -groundwater at 9.1' approximately 15 minutes after initial encounter -gray 10'-12' -boring cave-in at 10.8' during drilling
	Medium dense, tannish gray Silty Sand (SM), with lean clay pockets, wet
15	Soft to very stiff, tan and light gray Fat Clay (CH), with slickensides -with calcareous nodules 16'-18' -with silt pockets 18'-20' -tan and reddish tan, with silt seams 23'-25' -boring cave-in at 26.8' after completion of drilling
	Soft, tan and light gray Lean Clay with Sand (CL), with silty clay partings
30	Silty Sand (SM), wet
	Termination depth = 30 feet

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	25					
	18					
	17		69	29	17	12
	17	113.2				
	18					
			71	27	15	12
	16					
	19	109.8				
22	23					
			99			
4	37					
			100	74	27	47
	30					
	31	96.2				
	23					
			79	28	19	9
	24					

SHEAR STRENGTH, TSF



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

WATER ENCOUNTERED AT **14** FEET WHILE DRILLING

WATER LEVEL AT **5.2** FEET AFTER **COMPLETE**

DRILLED BY **GDT** DRAFTED BY **HJ** LOGGED BY **BTC**

KEY TO SYMBOLS

Symbol Description

Strata symbols

Paving

Fill

Low plasticity clay

Silty sand

High plasticity clay

Misc. Symbols

Water table depth during drilling

Subsequent water table depth

Torvane

Pocket Penetrometer

Unconfined Compression

Confined Compression

Soil Samplers

Auger

Undisturbed thin wall Shelby tube

Standard penetration test

AMLES ENGINEERING CORP.

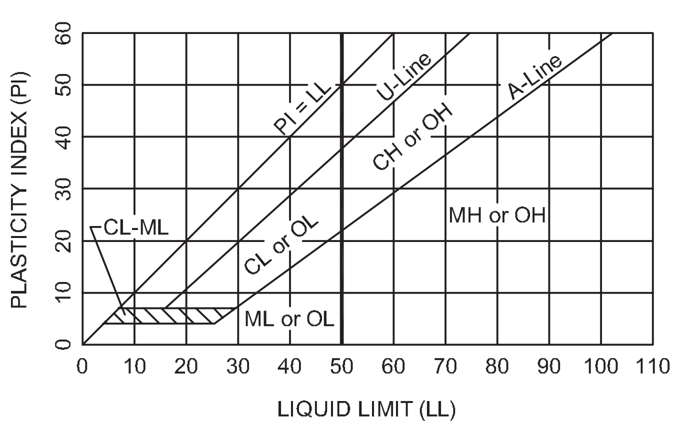
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

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

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

PLASTICITY CHART



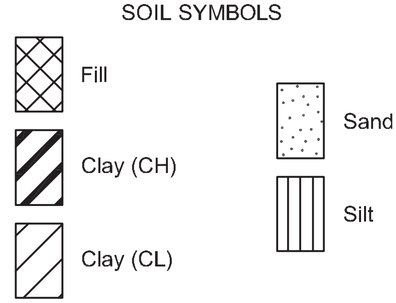
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

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

DEGREE OF PLASTICITY OF COHESIVE SOILS

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

SOIL SYMBOLS



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TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6"	3"	3/4"	#4	#10	#40	#200		
BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		
152	76.2	19.1	4.76	2.00	0.420	0.074	0.002	

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

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

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

DRY STRENGTH ASTM D2488

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

MOISTURE CONDITION ASTM D2488

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

SOIL STRUCTURE

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

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

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

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APPENDIX B

Plate B-1	Geotechnical Design Parameters for Excavation Shoring Design

G148-19 SINKHOLE AT INTERSECTION OF STANDIFER LANE AND LEE ROAD, HOUSTON, TEXAS
SOIL PARAMETERS FOR EXCAVATION SHORING

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	0-5	Stiff to very stiff CL	125	63	B	1000	0	1.00	1.00	1.00	100	18	0.53	0.69	1.89
	5-10	Very stiff CL	132	70	C*	1000	0	1.00	1.00	1.00	100	18	0.53	0.69	1.89
	10-12	Firm CL	131	69	C	550	0	1.00	1.00	1.00	50	18	0.53	0.69	1.89
	12-14	Medium dense SM	120	58	C	0	30	0.33	0.50	3.00	0	30	0.33	0.50	3.00
	14-16	Soft to very stiff CH	125	63	C*	1000	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	16-25	Very stiff CH	126	64	C* (16'-20')	2150	0	1.00	1.00	1.00	200	16	0.57	0.72	1.76

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

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

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

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

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

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

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

B: cohesive soils with qu = 0.5 tsf or greater

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

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

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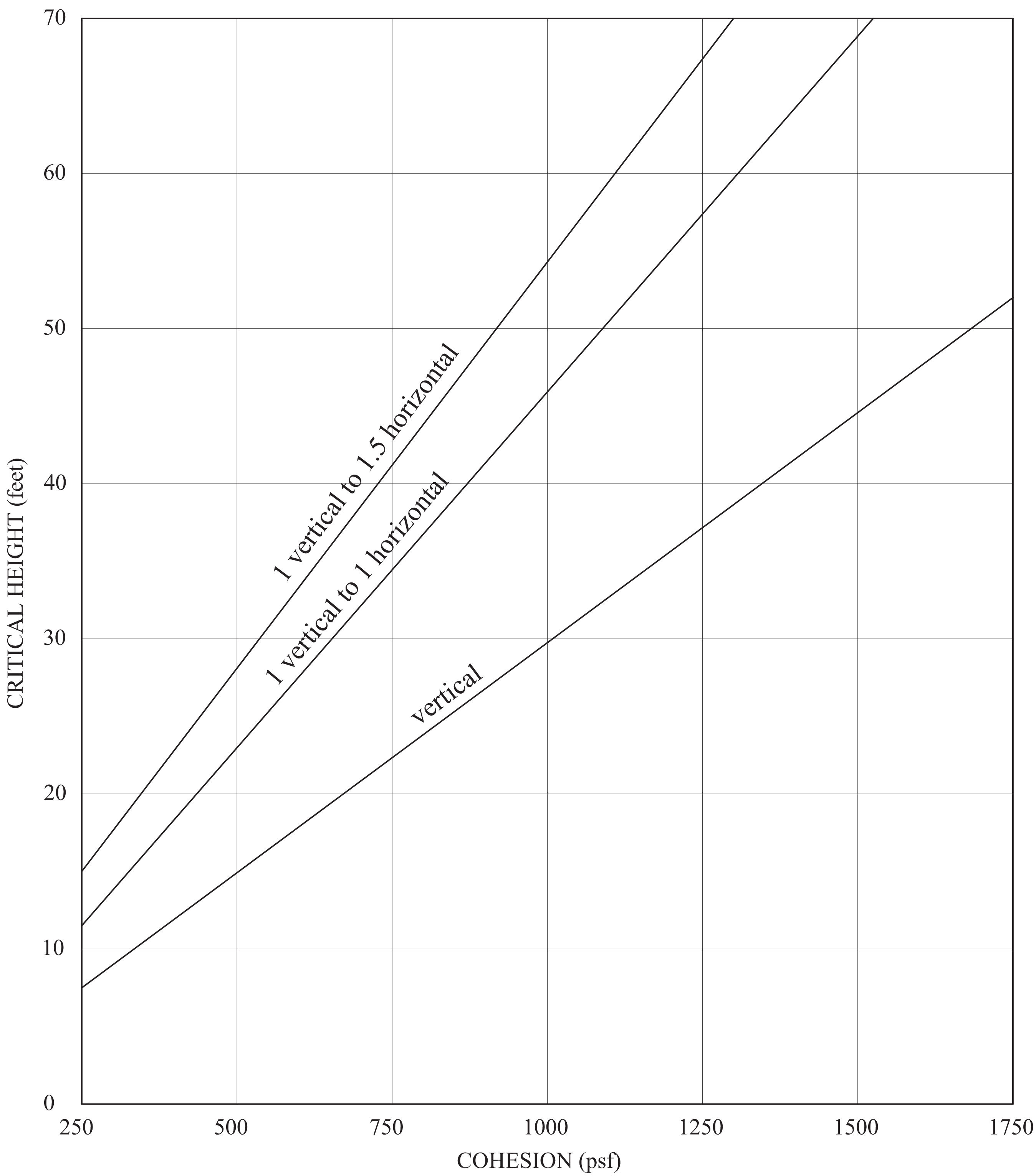
APPENDIX C

AVILES
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.

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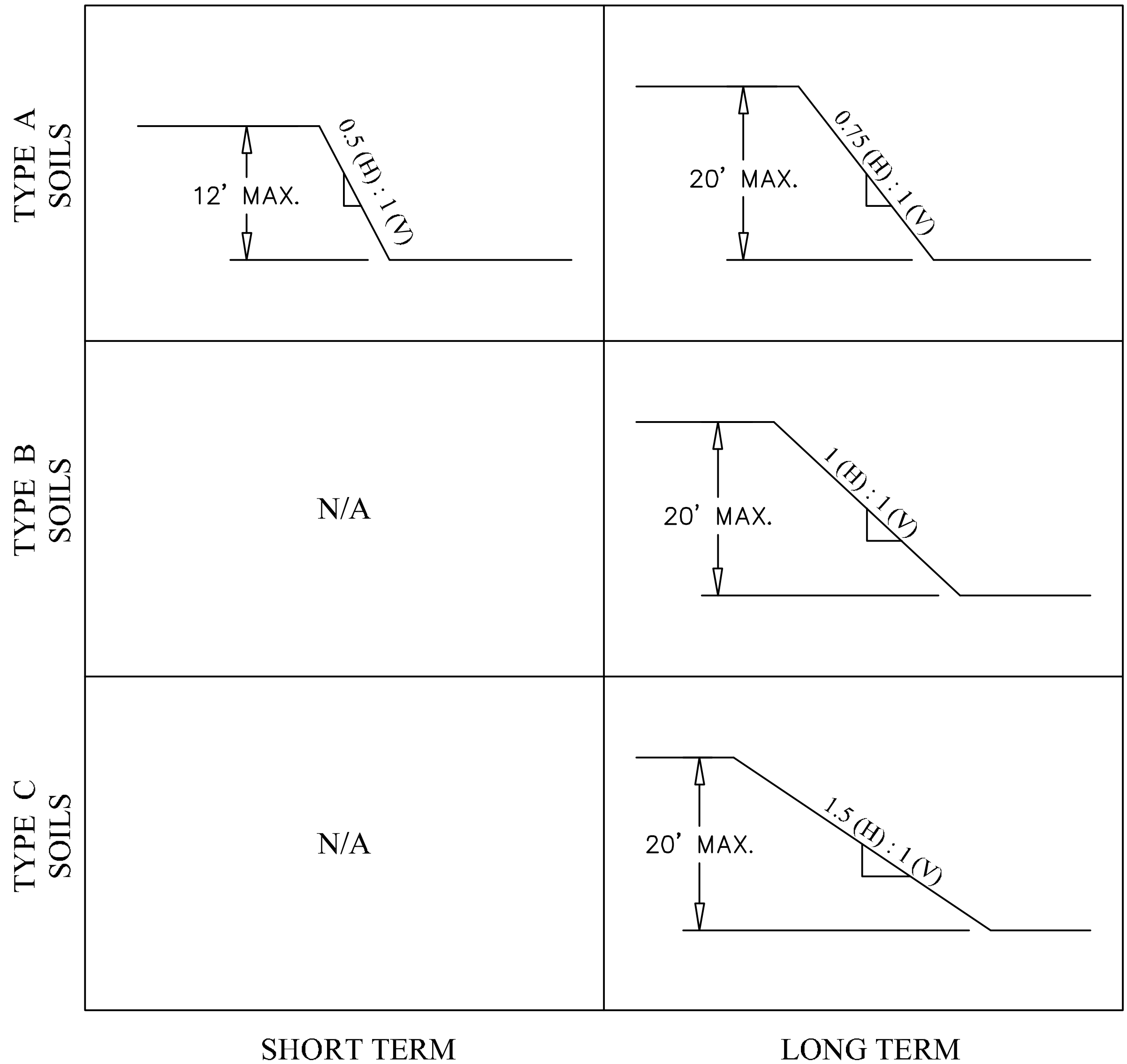
MAXIMUM ALLOWABLE SLOPES



NOTES:

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

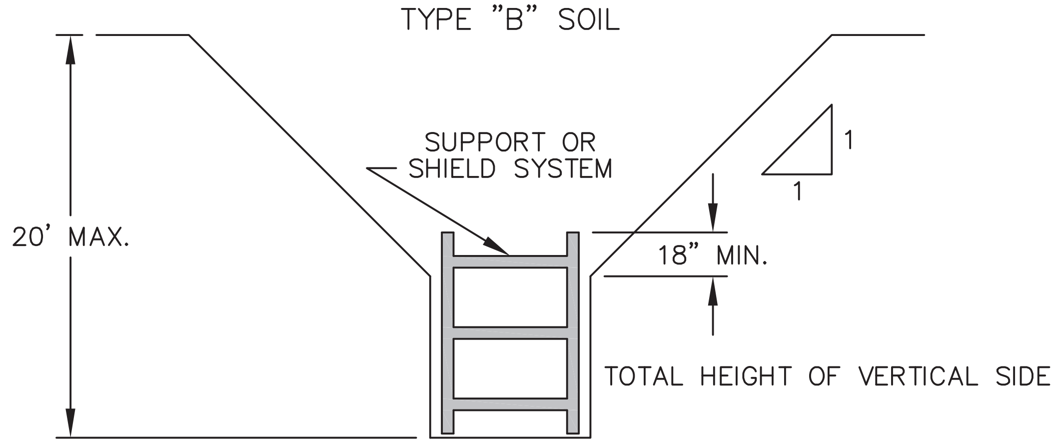
(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

(3) When surcharge loads from stored material or equipment, operating equipment, or traffic are present, a competent person shall determine the degree to which the actual slope must be reduced below the maximum allowable slope.

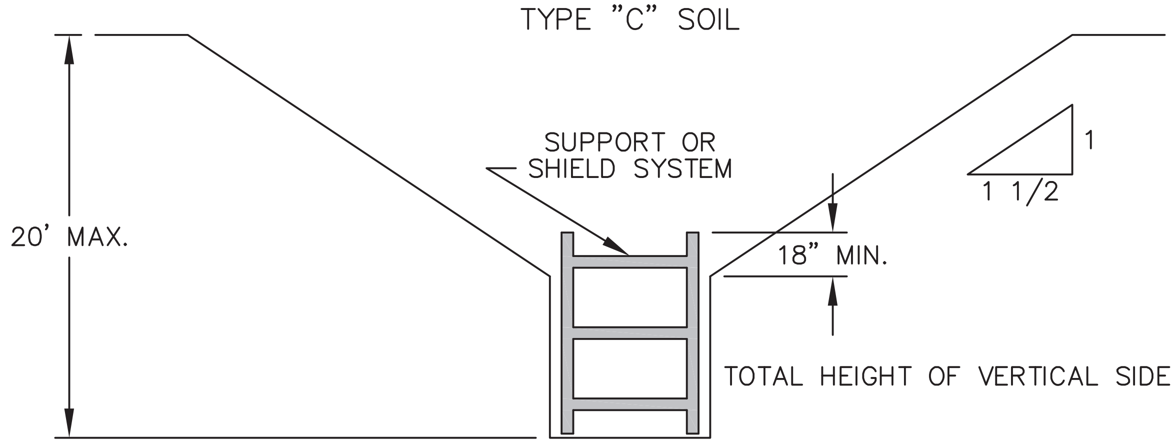
Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

A COMBINATION OF BRACING AND OPEN CUTS

TYPE "B" SOIL



TYPE "C" SOIL



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

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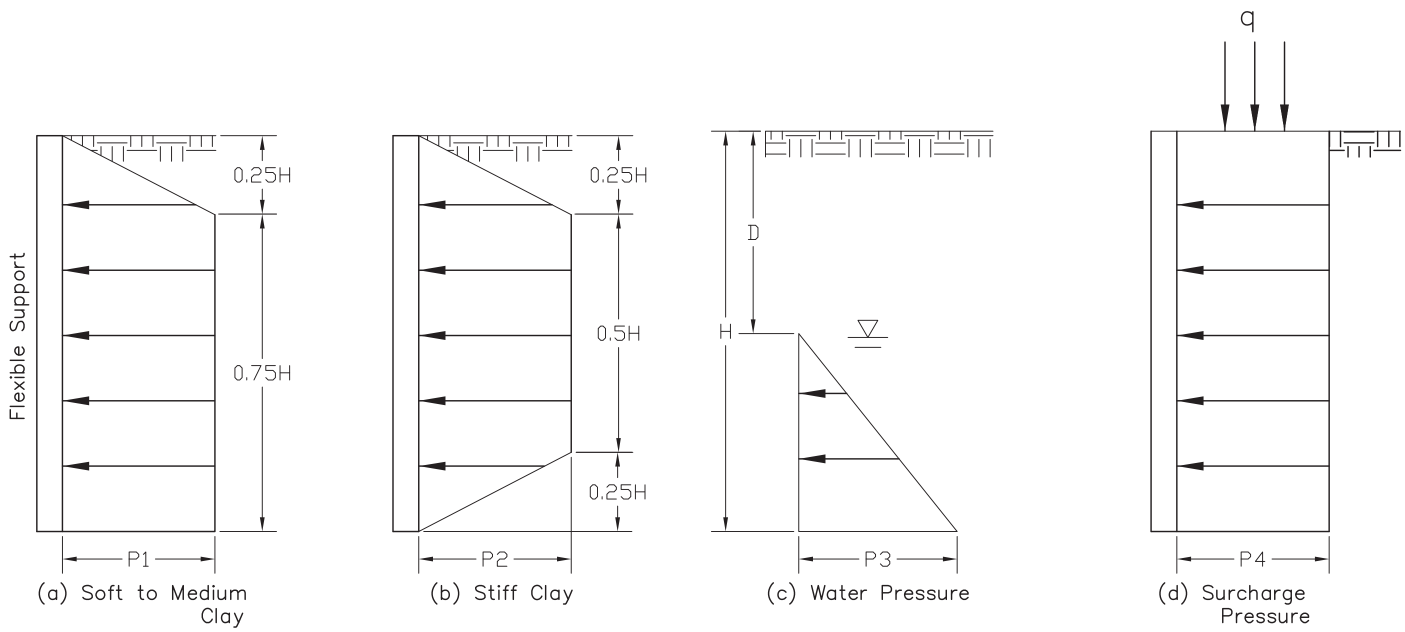
LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



Empirical Pressure Distributions

Where:

H = Total excavation depth, feet

D = Depth to water table, feet

P1 = Lateral earth pressure = γH-4C, psf

P2 = Lateral earth pressure = 0.4γH, psf

P3 = Water pressure = γ_w (H-D), psf

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

γ = Effective unit weight of soil, pcf

γ_w = Unit weight of water, pcf

C = Drained shear strength or cohesion, psf

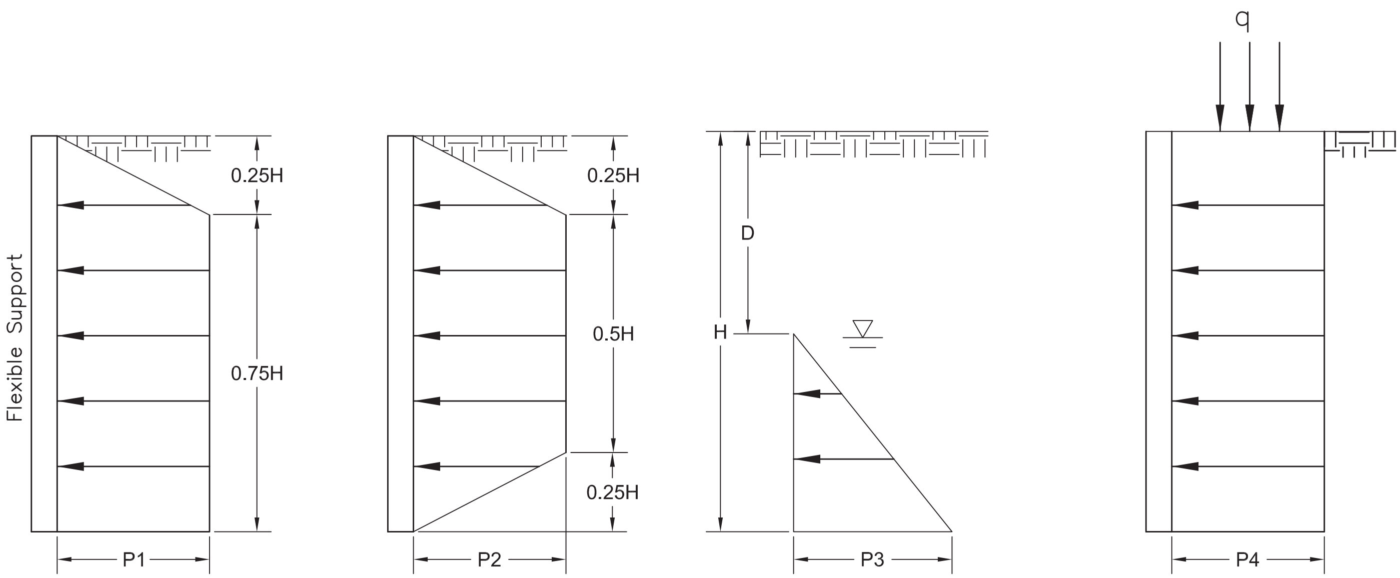
K_a = Coefficient of active earth pressure

Notes:

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

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

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 = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
S_u = Undrained shear strength = $q_u/2$, psf
q_u = Unconfined compressive strength, psf
K_a = Coefficient of active earth pressure

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

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

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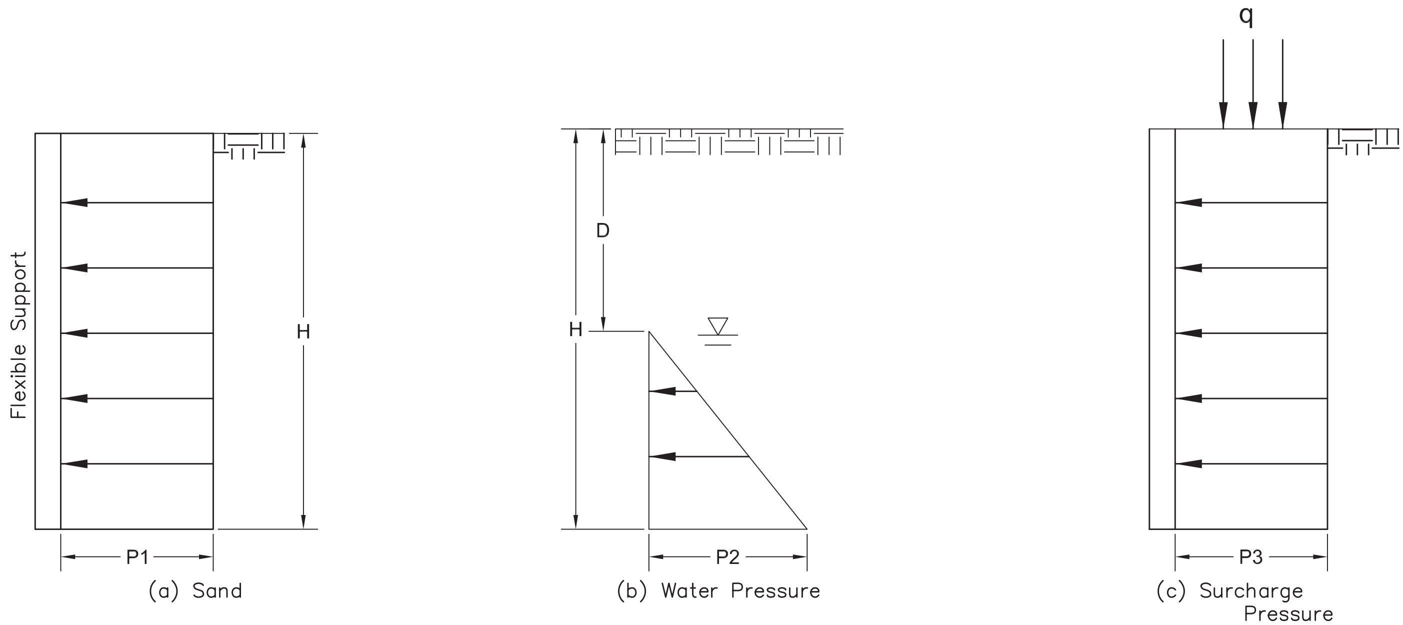
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



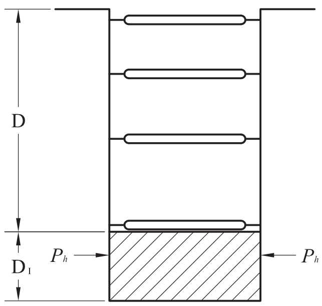
Empirical Pressure Distributions

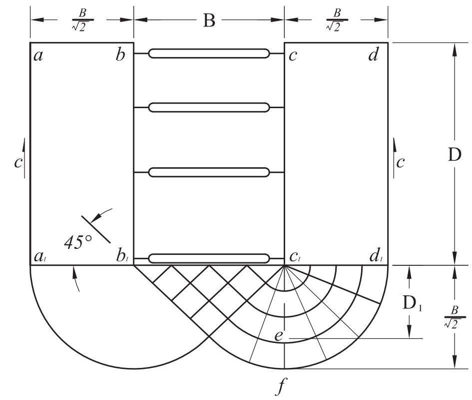
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65 * \gamma H K_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = (1-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,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

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

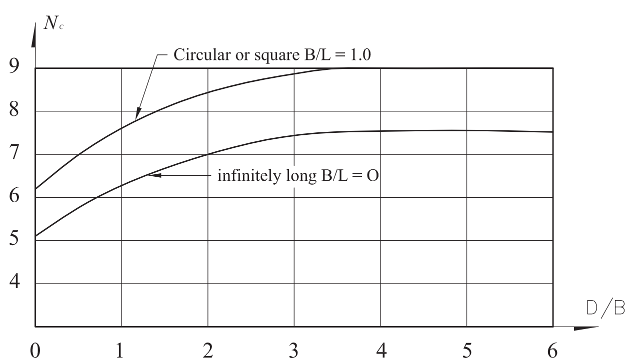
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \;;\; D_1 \geqslant 5 \text{ ft.}$$

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).