ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

CITY OF HOUSTON
GREENRIDGE WWTP IMPROVEMENTS
WBS NO. R-000265-0079-3
6301 W. FUQUA STREET
HOUSTON, TEXAS

Reported to

Infrastructure Associates
Houston, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G132-13R1

October 2014

AVILES ENGINEERING CORP.

October 21, 2014

Mr. Eric Cardwell, P.E.
Infrastructure Associates, Inc.
6117 Richmond Avenue, Suite 200
Houston, Texas 77057

Reference: **Geotechnical Investigation**
Greenridge Wastewater Treatment Plant Improvements
6301 W. Fuqua Street
WBS No. R-000265-0079-3
Houston, Texas
AEC Report No. G132-13

Dear Mr. Cardwell,

Aviles Engineering Corporation (AEC) is pleased to present this report of our geotechnical investigation for the above referenced project. This authorization to proceed for the investigation was provided via email on May 16, 2013 by Mr. Eric Cardwell, P.E. of Infrastructure Associates, Inc., based upon AEC Proposal No. G2013-04-13R1, dated May 2, 2013. Four additional borings (Borings B-7 through B-10) were authorized via email on April 8, 2014, based on AEC Proposal G2013-04-13S, dated November 25, 2013. The contents of this report supersede AEC's previous report for this project, dated September 11, 2014.

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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TABLE OF CONTENTS

APPENDICIES

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EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed City of Houston (COH) Greenridge Wastewater Treatment Plant (WWTP) Improvements, located at 6301 W. Fuqua Street in Houston, Texas (Houston/Harris Key Map: 571X). A vicinity map is presented on Plate A-1 in Appendix A. Based on design drawings (dated August 1, 2014) prepared by Infrastructure Associates, Inc. (IA), the proposed improvements include: (i) a new 63.2 foot long by 23.2 foot wide one-story blower building; (ii) a new 23.3 foot long by 22.7 foot wide by 9.5 foot high scum removal platform; (iii) a new 35.3 foot long by 28.5 foot wide concrete containment slab for two 6,500 gallon sodium bisulphite storage tanks; (iv) 2.2 to 2.5 foot deep detention swales with side slope inclination of H:V = 3:1; and (v) new concrete driveways for the facility entrance, and at the north, central, and southeast portions of the site.

Our findings are summarized below:

- Based on Boring B-1, the subsurface conditions at the blower building generally consist of approximately 16 feet of stiff to hard lean/fat clay (CL/CH) fill at the ground surface, underlain by approximately 9 feet of stiff to very stiff fat/lean clay (CH/CL) to the boring termination depth of 25 feet.

  Based on Boring B-7, the subsurface conditions at the sodium bisulphite storage tanks generally consist of approximately 12 feet of firm to hard fat clay (CH) fill at the ground surface, underlain by approximately 8 feet of stiff to very stiff fat clay (CH) to the boring termination depth of 20 feet.

  Based on Boring B-8, the subsurface conditions at the scum removal structure generally consist of approximately 28 feet of firm to very stiff lean/fat clay (CL/CH), underlain by approximately 4 feet of medium dense silty sand (SM) to the boring termination depth of 32 feet.

- Details of the soils encountered during drilling are presented on the boring logs. The cohesive soils (both fill and natural) encountered in our borings have Liquid Limits (LL) ranging from 30 to 62 and Plasticity Indices (PI) ranging from 13 to 44. This indicates that the cohesive soils have high to very high expansive potential. The cohesive soils encountered are classified as “CL” and “CH” type soils and granular fill soils encountered are classified as “SC” and “SM” type soils in accordance with ASTM D 2487.

- Groundwater was encountered at a depth of 14.3 to 28 feet during drilling and subsequently was observed at a depth of 14.5 to 18.5 feet approximately 15 minutes after the initial encounter in Borings B-2 and B-8. Groundwater was not encountered in the remaining borings.

- We did not detect any visual evidence or odor indicating the presence of hazardous materials in the soil samples. However, AEC notes that the presence of potential hazardous material within the project area cannot be discounted based upon the very small and limited number of samples taken.

- Recommendations for design and construction of the blower building are presented in Section 5.1 of this report. Based on the 16 foot thick layer of fill encountered in Boring B-1, AEC recommends that the blower building should be supported on drilled-and-underreamed footings at a depth of 16 feet below existing grade.

EXECUTIVE SUMMARY (Cont.)

- Recommendations for design and construction of the scum removal structure are presented in Section 5.2 of this report. AEC recommends that the scum removal structure be supported on drilled-and-underreamed footings at a depth of 10 feet below existing grade.

- Recommendations for design and construction of the sodium bisulphite storage tanks are presented in Section 5.3 of this report. Based on the 12 foot thick layer of fill encountered in Boring B-7, AEC recommends that the storage tanks be supported on a mat foundation at 2 feet below existing grade.

- Recommendations for detention swales are presented in Section 5.4 of this report.

- Recommendations for design and construction of concrete driveways are presented in Section 5.5 of this report. AEC recommends that the driveways at the site will be paved with 7 inch concrete pavement, with a 6 inch thick lime-stabilized subgrade.

- This Executive Summary provides an overview of the geotechnical investigation and should not be used without the full text of this report.

AVILES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

CITY OF HOUSTON
GREENRIDGE WWTP IMPROVEMENTS
WBS NO. R-000265-0079-3
HOUSTON, 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 City of Houston (COH) Greenridge Wastewater Treatment Plant (WWTP) Improvements, located at 6301 W. Fuqua Street in Houston, Texas (Houston/Harris Key Map: 571X). A vicinity map is presented on Plate A-1 in Appendix A. Based on design drawings (dated August 1, 2014) prepared by Infrastructure Associates, Inc. (IA), the proposed improvements include: (i) a new 63.2 foot long by 23.2 foot wide one-story blower building; (ii) a new 23.3 foot long by 22.7 foot wide by 9.5 foot high scum removal platform; (iii) a new 35.3 foot long by 28.5 foot wide concrete containment slab for two 6,500 gallon sodium bisulphite storage tanks; (iv) 2.2 to 2.5 foot deep detention swales with side slope inclination of H:V = 3:1; and (v) new concrete driveways for the facility entrance, and at the north, central, and southeast portions of the site. The contents of this report supersede AEC's previous report for this project, dated September 11, 2014.

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 design and construction of the blower building, scum removal platform, sodium bisulphite storage tanks, detention swales, and concrete pavement. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling ten soil borings varying in depth from 10 to 32 feet below existing grade;
2. Performing soil laboratory testing on selected soil samples;
3. Engineering analysis and recommendations for the blower building, scum removal platform, and sodium bisulphite storage tanks, including feasible foundation type and depth, allowable bearing capacity, floor slab, and subgrade preparation;
4. Recommendations for the detention swales, including clay liner (if any) requirements;
5. Engineering analyses and recommendations for concrete pavement, including pavement thickness design and subgrade preparation;
6. Construction recommendations for the blower building, scum structure, storage tanks, detention swales, and concrete pavement.

2.0 SUBSURFACE EXPLORATION

Based on preliminary information provided by IA, subsurface conditions at the site were investigated by drilling six borings (Borings B-1 through B-6) to depths ranging from 10 to 25 feet below existing grade. The total drilling footage for Borings B-1 through B-6 was 95 feet. As requested by the COH Geo-Environmental Services Branch, AEC collected three samples (Samples S-1 through S-3) from the ground surface to a depth of 12 inches from within the perimeter of the proposed drainage swale.

After Borings B-1 through B-6 were drilled, 30 percent complete drawings (dated October 25, 2013) were provided to AEC by IA. The 30 percent plans indicated that: (i) the location of the proposed sodium bisulphite tanks were moved to a new location approximately 150 feet southeast of the original proposed location; (ii) a second detention pond would be added to the west of the existing sludge dewatering building; (iii) a new scum removal platform would be added adjacent to the existing office and lab building; and (iv) new concrete driveways at the facility entrance (between the entrance gate and West Fuqua Drive), to the south of existing Clarifier No. 3, and along the southern property line. Based on the 30 percent complete drawings, AEC recommended that four additional borings be drilled to cover the proposed improvements. Borings B-7 through B-10 were drilled to depths ranging from 10 to 32 feet. The total drilling footage for Borings B-7 through B-10 was 77 feet.

After Borings B-7 through B-10 were drilled, 90 percent complete drawings (dated April 30, 2014) were provided to AEC by IA. The 90 percent plans indicated that: (i) both detention ponds were deleted and replaced with a 2.2 to 2.5 feet deep detention swales; and (ii) an additional concrete driveway (in a northeast-southwest direction) will be constructed in between the existing aerobic digester and aeration basins (on the west side of the driveway) and the thickener and clarifiers (on the east side of the driveway). Based on the 90 percent complete drawings, the current soil borings do not adequately cover the central portion of the proposed driveway between the existing digester/aeration basin and the thickener/clarifiers. According to the COH Geo-Environmental Branch, an additional boring will not be needed for this area. AEC will not be liable for any changed soil or groundwater conditions that may be encountered during construction in this area.

The boring locations are shown on the attached Boring Location Plan on Plate A-2, in Appendix A. After completion of drilling, boring locations were surveyed by Western Group Consultants. Boring survey data is included on the boring logs, and are also summarized in Table 1 below.

Table 1. Summary of Borings

Boring	Purpose	Depth (ft)	Easting (Surface)	Northing (Surface)	Elevation (ft)
B-1	Blower Building and Pavement	25	3,087,798.66	13,785,470.63	63.24
B-2	Scum Removal Platform (moved) and Pavement	20	3,087,857.06	13,785,564.01	62.73
B-3	Underground Utilities (deleted) and Pavement	10	3,087,974.70	13,785,461.59	63.52
B-4	Detention Swale and Pavement	15	3,087,763.74	13,785,013.23	64.09
B-5	Detention Swale and Pavement	15	3,087,844.44	13,784,952.33	63.08
B-6	Pavement	10	3,088,096.87	13,784,812.73	62.35
B-7	Sodium Bisulphite Storage Tanks and Pavement	20	3,087,929.84	13,785,411.31	64.20
B-8	Scum Removal Platform and Pavement	32	3,087,962.83	13,785,022.82	62.94
B-9	Detention Pond (deleted) and Pavement	15	3,087,926.91	13,784,900.47	63.31
B-10	Pavement	10	3,087,782.75	13,785,199.82	63.98

Note: (a) Texas State Plane Coordinate System, South Central Zone No. 4204 (NAD 83).
(b) Elevations based on NAVD 1988, 2001 adjustment.

The field drilling was performed with a truck-mounted drilling rig using dry auger method to advance the borings. 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. 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. The borings were backfilled with bentonite chips after completion of drilling. Details of the soils encountered in the borings are presented on Plates A-3 through A-12, 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, 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-13 through A-16, in Appendix A.

4.0 SITE CONDITIONS

4.1 Subsurface Conditions

Soil strata encountered in our borings are summarized below.

Boring	Depth	Description of Stratum
B-1	0' - 8'	Fill: hard, Lean Clay w/Sand (CL), with siltstone fragments
	8' - 12'	Fill: very stiff to hard, Fat Clay (CH), with siltstone fragments
	12' - 16'	Fill: stiff to very stiff, Lean Clay (CL), with sand pockets and siltstone fragments
	16' - 21'	Stiff to very stiff, Fat Clay (CH), with slickensides and silt partings
	21' - 25'	Very stiff, Lean Clay (CL), with abundant silt seams and siltstone fragments
B-2	0' - 14'	Fill: firm to hard, Fat Clay (CH), with shell
	14' - 16'	Fill: Sandy Fat Clay (CH), with abundant sand seams
	16' - 20'	Stiff to very stiff, Fat Clay w/Sand (CH), with slickensides
B-3	0' - 2'	Fill: very stiff, Fat Clay (CH), with shell and sand seams
	2' - 10'	Very stiff, Fat Clay (CH), with slickensides
B-4	0' - 2'	Fill: hard, Fat Clay (CH), with shell, sand clay seams, roots, and siltstone fragments
	2' - 10'	Hard, Fat Clay w/Sand (CH)
	10' - 14'	Stiff to very stiff, Lean Clay (CL), with abundant silt partings
	14' - 15'	Very stiff, Fat Clay (CH), with silty clay seams and silt pockets
B-5	0' - 2'	Fill: Clayey Sand (SC), with shell and siltstone fragments
	2' - 10'	Very stiff to hard, Fat Clay w/Sand (CH)
	10' - 14'	Stiff to very stiff, Lean Clay (CL), with abundant silt partings and siltstone fragments
	14' - 15'	Stiff to very stiff, Fat Clay (CH), with silty clay seams and siltstone fragments

Boring	Depth	Description of Stratum
B-6	0' - 0.33'	Pavement: 4" asphalt
	0.33' - 0.92'	Base: 7" asphalt stabilized gravel
	0.92' - 2'	Fill: Fat Clay (CH), with asphalt pieces and gravel
	2' - 10'	Very stiff to hard, Fat Clay w/Sand (CH)
B-7	0' - 12'	Fill: firm to hard, Fat Clay (CH)
	12' - 20'	Stiff to very stiff, Fat Clay (CH), with slickensides
B-8	0' - 0.2'	Pavement: 2" asphalt
	0.2' - 0.8'	Base: 8" crushed gravel with clayey sand
	0.8' - 2'	Very stiff, Lean Clay w/Sand (CL), with silty sand pockets
	2' - 10'	Firm to very stiff, Fat Clay w/Sand (CH)
	10' - 14'	Firm to very stiff, Lean Clay w/Sand (CL), with silt partings
	14' - 28'	Firm to very stiff, Fat Clay (CH), with slickensides
	28 - 32'	Medium dense, Silty Sand (SM)
B-9	0' - 2'	Fill: hard, Sandy Lean Clay (CL), with sand seams and gravel
	2' - 8'	Fill: stiff to very stiff, Lean Clay w/Sand (CL), with sand pockets
	8' - 15'	Soft to very stiff, Lean Clay (CL), with slickensides
B-10	0' - 1'	Base: 12' silty sand with gravel, roots, and clay pockets
	1' - 2'	Fill: very stiff, Lean Clay w/Sand (CL), with gravel, silty sand partings, and fat clay pockets
	2' - 10'	Stiff to very stiff, Fat Clay (CH)

Details of the soils encountered during drilling are presented on the boring logs. The cohesive soils (both fill and natural) encountered in our borings have Liquid Limits (LL) ranging from 30 to 62 and Plasticity Indices (PI) ranging from 13 to 44. This indicates that the cohesive soils have high to very high expansive potential. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular fill soils encountered are classified as "SC" and "SM" type soils in accordance with ASTM D 2487. "CH" soils can undergo significant volume changes due to seasonal changes in 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. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes.

Groundwater: Groundwater was encountered at a depth of 14.3 to 28 feet during drilling and subsequently was observed at a depth of 14.5 to 18.5 feet approximately 15 minutes after the initial encounter in Borings B-2 and B-8. Groundwater was not encountered in the remaining borings. Groundwater level measurements encountered during drilling are summarized in Table 2.

Table 2. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth Encountered during Drilling (ft)	Groundwater Depth 15 min. after Initial Encounter (ft)
B-1	6/5/13	25	Dry	Dry
B-2	6/5/13	20	14.3	14.5
B-3	6/5/13	10	Dry	Dry
B-4	6/5/13	15	Dry	Dry
B-5	6/5/13	15	Dry	Dry
B-6	6/5/13	10	Dry	Dry
B-7	4/21/14	20	Dry	Dry
B-8	4/21/14	32	28	18.5
B-9	4/21/14	15	Dry	Dry
B-10	4/21/14	10	Dry	Dry

The information in this report summarizes conditions found on the dates the borings were drilled. However, it should be noted that our ground water observations are short term; ground water 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

We did not detect any visual evidence or odor indicating the presence of hazardous materials in the soil samples. However, AEC notes that the presence of potential hazardous material within the project area cannot be discounted based upon the very small and limited number of samples taken.

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 away from and in between borings.

Clay soils in the Houston area typically have secondary features such as slickensides 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 which were generally continuously obtained at intervals of 2 feet from the ground surface to a depth of 20 feet, then at 5 foot intervals 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 some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

Based on design drawings (dated August 1, 2014) prepared by IA, the proposed improvements include: (i) a new 63.2 foot long by 23.2 foot wide one-story blower building; (ii) a new 23.3 foot long by 22.7 foot wide by 9.5 foot high scum removal platform; (iii) a new 35.3 foot long by 28.5 foot wide concrete containment slab for two 6,500 gallon sodium bisulphite storage tanks; (iv) 2.2 to 2.5 foot deep detention swales with side slope inclination of H:V = 3:1; and (v) new concrete driveways for the facility entrance, and at the north, central, and southeast portions of the site.

5.1 Blower Building

Based on the design drawings, the existing blower building at the site will be demolished, and a new 63.2 foot long by 23.2 foot wide blower building will be constructed approximately 90 feet to the north of the existing location. The footprint of the new building will not overlap with the existing building footprint. The finished floor elevation of the blower building will be at 63.75 feet above Mean Sea Level (MSL). According to Infrastructure Associates, the long term loads for the blower building is 700 kips, and the short term load is 730 kips. The given loads are the total loads from the superstructure (minus slab on grade loads) on the foundation system, and are not individual pier loads.

Based on Boring B-1 (at a surface elevation of 63.24 feet above MSL), the soil conditions encountered at the proposed blower building consist of approximately 16 feet of stiff to hard lean/fat clay (CL/CH) fill, underlain by stiff to very stiff fat/lean clay (CH/CL) to the boring termination depth of 25 feet below grade. In addition, numerous siltstone fragments, gravel, roots, and sand pockets were encountered in the fill materials encountered in Boring B-1.

Considering the non-uniform nature of the fill and the potential for detrimental long term vertical and differential settlement on the blower building, AEC recommends that the building foundations extend through the existing uncontrolled fill material and bear on the natural clay soils beneath the fill. The blower building should be supported on drilled-and-underreamed footings founded at least 16 feet below grade. If the footings are constructed at a depth of less than 16 feet and are instead terminated in the fill itself, the footings can experience significant differential settlement as the uncontrolled fill consolidates at varying rates from the foundation loads over time.

5.1.1 Drilled-and-Underreamed Footings

Drilled-and-Underreamed Footings: AEC recommends that drilled-and-underreamed footings be founded at a depth of 16 feet below existing grade (i.e. at an elevation of approximately 47.2 feet above MSL), and be designed for a net allowable bearing capacity of 3,000 pounds per square foot (psf) for sustained loads and 4,500 psf for total loads, based on a minimum factor of safety (FS) of 3 for sustained loads and 2 for total loads, whichever is critical should be used for design.

Downdrag Force: The fill will be subjected to consolidation settlement which can result in negative skin friction on drilled-and-underreamed footings. Considering the potential downdrag force on the footings, we recommend that the average negative skin friction resulting from fill be calculated using the following equation (based on Tomlinson's "Foundation Design and Construction", 1995, and the US Naval Facilities Engineering Command's "Foundations and Earth Structures Design Manual 7.2", 1982):

$$f_n = \beta p_0 \qquad \text{............Equation (1)}$$

Where, f_n = unit negative skin friction (to be multiplied by area of shaft in zone of subsiding soil relative to shaft),
p_0 = effective vertical stress at the middle depth of uncontrolled fill, $p_0 = \gamma h_c$,
γ = 120 pcf, unit weight of uncontrolled fill
h_c = middle depth of uncontrolled fill, ft
β = empirical factor from full scale tests, we recommend the use of $\beta = 0.20$ for uncontrolled fill.

The total downdrag force imposed on a footing due to negative skin friction in the uncontrolled fill can be calculated by Equation (2):

$$Q_n = f_n(3.14dh) \qquad \text{............Equation (2)}$$

Where, Q_n = total downdrag force imposed on a drilled shaft due to negative skin friction in the uncontrolled fill,
d = drilled shaft diameter, ft
h = shaft length within the uncontrolled fill, ft

Vertical Reinforcement: To withstand uplift forces resulting from the shrink/swell movements of clay soils in the moisture active zone, each footing should contain reinforcing steel throughout its full length to sustain an uplift load of at least 62d kips, where "d" is the diameter of the shaft in feet.

Footing Spacing: To reduce stress overlap from adjacent footings and potential construction problems, the minimum edge-to-edge clear spacing between the underreams should not be less than 0.6 x diameter of the larger underream.

Footing Settlements: Assuming that the footings extend through the uncontrolled fill and bear into the natural soils beneath the uncontrolled fill, we estimate that drilled-and-underreamed footings designed and constructed as recommended will experience total settlements within 1 inch. If the footing depth is reduced and bears instead within the uncontrolled fill, estimated settlements can exceed 1 inch. However, the exact amount of settlement is difficult to estimate, given the non-uniform nature of the uncontrolled fill.

Drilled-and-Underreamed Footing Construction: Drilled-and-underreamed footings should be constructed in accordance with Section 02465 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS). A qualified geotechnical technician should check each footing excavation prior to placing concrete to insure that:

1) The footing has been constructed to the specified dimensions at the recommended depth and founded in the correct formation as indicated in this report;
2) The column is concentric with the pier cap/grade beam and drilled footing; and
3) Excessive cuttings, any soft or compressible materials, and ponded water are removed from the bottom of the excavation.

There is a possibility that slickensides and/or pockets/seams of sands/silts within the clay soils may make underreaming (belling) difficult, and result in potential sloughing or caving-in of the shaft excavation sidewalls during construction, particularly for underreams over 6 feet in diameter. We recommend that a maximum diameter ratio of bell to shaft not exceed 2.5 to 1. Based on Boring B-1, the top 16 feet of soils at the proposed blower building consist of clay fill with abundant silt partings and siltstone fragments, which have a potential for sidewall sloughing and caving during shaft excavation. If significant sloughing or caving occurs during shaft excavation, further excavation should be stopped and a reduced bell/shaft ratio or even straight-sided shafts (matching the bell diameter) in combination with bentonite slurry and/or temporary casing may be necessary. Although groundwater was not encountered in Boring B-1, the site's groundwater level will fluctuate with seasonal rainfall and other climatic events, and may be higher at the time of construction. If ground water is encountered within the cohesive soils during construction, sump pumps may be used to pump water out from the excavations and soft sediments should be removed.

Placement of concrete should be accomplished immediately after excavation is completed to reduce potential for sloughing of the foundation soils. Footing excavations should not be left open overnight. No concrete should be placed without the prior approval of the Owner's Representative. New drilled footings should not be excavated within 2 bell diameters (edge to edge) of an open footing excavation, or one in which concrete has been placed in the preceding 24 hours, to prevent movement of fresh concrete from the recently filled footing to an adjacent unfilled footing.

Construction Monitoring: AEC notes that the 63.2 foot long by 23.2 foot wide footprint of the proposed blower building indicated on the design drawings is larger than what was indicated during the preliminary design phase of the project (when only one boring was proposed for the blower building). As a result, the thickness and strength of the existing fill material can vary across the building footprint. AEC should be retained to monitor the construction of the building foundations and determine if the fill materials encountered during construction are similar to those encountered in Boring B-1. If the thickness and strength of the fill materials encountered during foundation construction vary considerably from those encountered in Boring B-1, AEC will revise the drilled-and-underreamed footing recommendations presented in this report accordingly.

5.1.2 Floor Slab

Based on Boring B-1, the Client should be aware that the uncontrolled fill contains siltstone fragments, gravel, sand pockets, and silty clay seams to a depth of 16 feet. Variations from our borings in type and strength of the subgrade soils within the proposed building footprint should be expected. Significant long term settlement/differential settlement of the floor slab of the blower building can occur, which can cause distress such as cracks and unevenness in the floor slab.

Estimated Shrink/Swell Soil Movements: Expansive clays exhibit a potential to shrink and swell with changes in their moisture contents. The changes in the soil moisture content are usually caused by variations in the seasonal amount of rainfall and evaporation rates or other localized factors like the moisture withdrawal by nearby trees. The seasonal moisture active zone generally extends to about 10 feet below ground in the Houston area, and will be deeper if trees with deep root zones exist adjacent to the structure.

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Potential Vertical Rise (PVR) is an estimate of the potential of an expansive soil to swell from its current state. For the top 10 feet of the existing soils encountered in Boring B-1, the PVR at the blower building is estimated to be approximately 2.6 inches based on in-situ moisture conditions. PVR was computed using the Texas Department of Transportation (TxDOT) test method Tex-124-E.

Additional movements can occur in areas if water is allowed to pond during or after construction on soils with high plasticity, or if highly plastic soils are allowed to dry out prior to fill or concrete placement. High plasticity clay may also experience shrinkage during periods of dry weather as moisture evaporation occurs at the ground surface and the groundwater table drops. The actual PVR of the site will be highly dependent upon the actual PI and moisture regime of the clayey soils at the time of construction. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reducing potential shrink-swell movement of the floor slab.

Table 3. Estimated PVR vs. Thickness of Replacement Fill (Based on Boring B-1)

Thickness of Replacement Fill Beneath the Existing Ground Surface (ft)	PVR (in)
0 (Approx. EL = 63.2' MSL)	2.6
1 (Approx. EL = 62.2' MSL)	2.3
2 (Approx. EL = 61.2' MSL)	2.0
3 (Approx. EL = 60.2' MSL)	1.6
4 (Approx. EL = 59.2' MSL)	1.3
5 (Approx. EL = 58.2' MSL)	1.0

Floor Slab: In general, the tolerable differential vertical movement for a common building slab is about 1 inch. To limit the PVR to 1 inch, the following approaches can be used: (1) a drilled-and-underreamed footing supported structural floor slab in combination with 6 inch carton forms between the bottom of the slab and the subgrade soil; or (2) excavating approximately 5 feet of existing soils (i.e. to elevation 58.2 feet above MSL) and replacing them with a low-expansive select fill material (in accordance with Table 3).

5.1.2.1 Option 1 - Structural Floor Slab

To mitigate the effects of shrink/swell movements of expansive clays, as well as long term settlement of the uncontrolled fill, it is AEC's opinion that the most effective floor slab option is to use a structural floor slab that is supported by drilled-and-underreamed footings, with a minimum 6 inch space maintained between the bottom of the slab and the top of the subgrade soils.

Subgrade Preparation: Subgrade preparation should extend a minimum of 5 feet beyond the floor slab perimeter. Thereafter, a minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials shall be removed and wasted in accordance with Section 02233 of the latest edition of the COHSCS. The excavation depth should be increased when inspection indicates the presence of organics or deleterious materials to greater depths.

The exposed subgrade should then be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with clean onsite clay soils. Afterwards, general fill can be placed and compacted to achieve the design FFE of the building (i.e. the bottom of the carton forms). Fill should be in accordance with Section 02316 of the latest edition of the COHSCS.

Grade Beams: We recommend that foundation grade beams be founded at least 24 inches below the lowest final grade. The grade beams should be constructed on 6 inch carton forms. Care should be taken so that the carton forms do not collapse during concrete placement and will not be exposed to water in the grade beam excavations. Surface water should not be allowed to seep into and remain in the carton form space during the life of the structures. The drilled shafts and beams should be tied together.

Moisture Barrier: To prevent mildew or mold growth on the underside of the structural floor slab, we recommend that a horizontal moisture barrier (minimum 10-mil thick) be placed below the concrete slab (on top of the carton forms).

5.1.2.2 Option 2 - Subgrade Supported Floor Slab

A less expensive alternative to a structural slab or full-depth (5 feet, see Table 3 in Section 5.1.2 of this report) expansive soil replacement is a reinforced on-grade floor slab with limited expansive soil replacement. A concrete slab-on-grade in conjunction with limited fill replacement can be considered, if the Owner is willing to take some risk of floor slab movement. **As stated above, there is a high risk of long term settlement/differential settlement of the new floor slab due to the uncontrolled fill soils. Due to the non-uniform nature of the uncontrolled fill, AEC will not be liable for the future performance or distress of a floor slab-on grade.**

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Subgrade Preparation: Subgrade preparation should extend a minimum of 5 feet beyond the floor slab perimeter. A minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials shall be removed and wasted in accordance with Section 02233 of the latest edition of the COHSCS. The excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths.

Afterwards, an additional 3.5 feet [total depth of 4 feet (i.e. an elevation of approximately 58.2 feet above MSL), which includes the 6 inches of surface removal] of existing soils should be removed. The exposed subgrade should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill or clean stabilized soils. After proof-rolling, compacted select fill or clean, stabilized soils should then be used to achieve the design FFE of the building (at an elevation of 63.75 feet above MSL). Select fill should be in accordance with Section 02320 of the latest edition of the COHSCS.

According to Table 3, the PVR for 4 feet of soil replacement is 1.3 inches, which is still greater than 1 inch. The Owner should be aware that the risk of floor slab movement is still present if this floor slab option is selected. If conditions which exacerbate moisture variations such as the presence of trees, poor drainage, excessive drying/wetting of subsurface soils, or leaking underground utilities are located nearby, the floor slab total vertical movements and net differential vertical movements could be higher than estimated.

Grade Beams: We recommend that foundation grade beams be founded at least 24 inches below the lowest final grade. The grade beams can be constructed on 6 inch carton forms. If carton forms are used, care should be taken so that the carton forms do not collapse during concrete placement and will not be exposed to water in the grade beam excavations. Surface water should not be allowed to seep into and remain in the carton form space during the life of the structures. If no carton forms will be used, we recommend that tensile reinforcement be placed in both top and bottom of the beams. The drilled-and-underreamed footings and beams should be tied together.

Floor slabs are typically structurally tied to the grade beams. Alternatively, isolating the floor slabs from grade beams with a flexible impervious compound will be beneficial to reduce the potential for slab cracking due to differential soil movement; however, its use will not mitigate the total and differential PVR movements and the floor slabs are expected to move corresponding to the subgrade soils.

Moisture Barrier: We recommend that a horizontal moisture barrier (minimum 10-mil thick) be placed below the concrete slab to move edge effects away from the slab and mitigate seasonal fluctuations of water content directly below the structure.

5.2 Scum Removal Platform

Based on the design drawings, the scum removal platform will be an elevated structure that is 9.5 feet high with a footprint that is 23.3 foot long by 22.7 foot wide. The ground surface beneath the elevated structure will be paved with concrete. According to Infrastructure Associates, the long term loads for the scum removal structure is 70 kips, and the short term load is 170 kips. The given loads are the total loads from the superstructure (minus slab on grade loads) on the foundation system, and are not individual pier loads. Lateral loading on the scum structure platform will be 3.8 kips.

Based on Boring B-8 (at a surface elevation of 62.94 feet above MSL), the soil conditions encountered at the proposed scum removal platform consist of approximately 28 feet of firm to very stiff lean/fat clay (CL/CH). AEC recommends that the scum removal platform be supported on drilled-and-underreamed footings, founded at least 10 feet below grade, i.e. below the zone of seasonal moisture variation (typically 10 feet deep in the Houston area).

5.2.1 Drilled-and-Underreamed Footings

Drilled-and-Underreamed Footings: AEC recommends that drilled-and-underreamed footings be founded at a depth of 10 feet below existing grade (i.e. at an elevation of approximately 52.9 feet above MSL), and be designed for a net allowable bearing capacity of 1,900 pounds per square foot (psf) for sustained loads and 2,850 psf for total loads, based on a minimum FS of 3 for sustained loads and 2 for total loads, whichever is critical should be used for design.

Vertical Reinforcement: To withstand uplift forces resulting from the shrink/swell movements of clay soils in the moisture active zone, each footing should contain reinforcing steel throughout its full length to sustain an uplift load of at least 28d kips, where “d” is the diameter of the shaft in feet.

Footing Spacing: To reduce stress overlap from adjacent footings and potential construction problems, the minimum edge-to-edge clear spacing between the underreams should not be less than 0.6 x diameter of the larger underream. New foundations for the scum removal platform should be spaced to reduce the potential of new foundations affecting building foundations supporting the adjacent office and lab building (and vice versa) by placing the new foundations outside the bearing (stress) zone of existing foundations. The bearing (stress) zone can be defined by a line drawn downward from the outer edge of the existing foundation and inclined at an angle of 45 degrees to the vertical.

Footing Settlements: Based on the soil conditions encountered and the anticipated structural loads, we estimate that drilled-and-underreamed footings, designed and constructed as recommended in this report, will experience total settlements on the order of 1 inch.

Drilled-and-Underreamed Footing Construction: General drilled-and-underreamed footing construction recommendations are presented in Section 5.1.1 of this report. We recommend that a maximum diameter ratio of bell to shaft not exceed 2.5 to 1. Based on Boring B-8, the top 10 feet of soils at the proposed scum removal structure consist of fat clay with abundant silty sand pockets and silt partings, which have a potential for sidewall sloughing and caving during shaft excavation. If significant sloughing or caving occurs during shaft excavation, further excavation should be stopped and a reduced bell/shaft ratio or even straight-sided shafts (matching the bell diameter) in combination with bentonite slurry and/or temporary casing may be necessary. Although the groundwater level encountered in Boring B-8 is below the anticipated footing depth, the site's groundwater level will fluctuate with seasonal rainfall and other climatic events, and may be higher at the time of construction. If ground water is encountered within the cohesive soils during construction, sump pumps may be used to pump water out from the excavations and soft sediments should be removed.

5.3 Sodium Bisulphite Tanks

Based on the design drawings, the two 6,500 gallon sodium bisulphite storage tanks will be supported on a 35.3 foot long by 27.5 foot wide by 12 inch thick mat foundation bearing at 1 foot below grade. According to Infrastructure Associates, the long term loads for the sodium bisulphite tanks is 420 kips, and the short term load is 440 kips.

Based on Boring B-7 (at a surface elevation of 64.20 feet above MSL), the soil conditions encountered at the proposed tanks consist of approximately 12 feet of firm to hard fat clay (CH) fill, underlain by stiff to very stiff fat clay (CH) to the boring termination depth of 20 feet below grade. In addition, gravel pockets were encountered in the fill materials encountered in Boring B-7.

Considering the non-uniform nature of the fill and the potential for detrimental long term vertical and differential settlement on the storage tanks, AEC recommends that the mat foundation bear at a depth of 2 feet below grade, and that an additional 2 feet of existing soils below the mat foundation be excavated and replaced with compacted select fill or lime-stabilized onsite clay.

5.3.1 Mat Foundation

Mat Foundation: A mat foundation founded at 2 feet below existing grade (i.e. at an elevation of approximately 62.2 feet above MSL) can be designed for a net allowable bearing capacity of 1,500 psf for sustained loads and 2,250 psf for total loads, based on a minimum FS of 3 for sustained loads and 2 for total loads; whichever is critical should be used for design.

Modulus of Subgrade Reaction: The modulus of subgrade reaction (k) is frequently used in the structural analysis of mat foundations. Based on the soil conditions encountered and the size of the mat foundation, we recommend using k = 50 pounds per cubic inch (pci) for a mat foundation founded at a depth of 2 feet below the existing ground surface.

Mat Settlement: Given the non-uniform nature of the existing fill at the tanks location, AEC recommends that the structural engineer design the mat to be rigid enough to tolerate differential settlements and also design the mat to bridge over any soft/weak soils beneath the mat footing. Given the non-uniform nature of the fill materials present, the amount of long term vertical and differential settlement will be difficult to predict.

Estimated Shrink/Swell Soil Movements General recommendations regarding shrink/swell movements of expansive soils and PVR are presented in Section 5.1.2 of this report. Considering that the mat foundation will be placed at 2 feet below existing grade, for the 8 feet of soils beneath the mat foundation, the PVR at the sodium bisulphite tanks (based on Boring B-7) is estimated to be approximately 1.9 inches based on in-situ moisture conditions.

Additional movements can occur in areas if water is allowed to pond during or after construction on soils with high plasticity, or if highly plastic soils are allowed to dry out prior to fill or concrete placement. High plasticity clay may also experience shrinkage during periods of dry weather as moisture evaporation occurs at the ground surface and the groundwater table drops. The actual PVR of the site will be highly dependent upon the actual PI and moisture regime of the clayey soils at the time of construction. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reducing potential shrink-swell movement of the floor slab.

Table 4. Estimated PVR vs. Thickness of Replacement Fill (Based on Boring B-7)

Thickness of Replacement Fill Beneath Mat Foundation at 2 feet Below Grade* (ft)	PVR (in)
0 (Approx. EL = 62.2' MSL) - Mat Bearing Depth	1.9
1 (Approx. EL = 61.2' MSL)	1.6
2 (Approx. EL = 60.2' MSL)	1.2
3 (Approx. EL = 59.2' MSL)	1.1
4 (Approx. EL = 58.2' MSL)	0.9

Note: (*) Depth of fill replacement corresponds to depth below the mat footing, which is at 2 feet below existing grade (at an elevation of 62.2 feet above MSL).

Subgrade Preparation: Subgrade preparation should extend a minimum of 5 feet beyond the mat foundation perimeter. A minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials shall be removed and wasted in accordance with Section 02233 of the latest edition of the COHSCS. The excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths.

Afterwards, an additional 3.5 feet [total depth of 4 feet (to an elevation of approximately 60.2 feet above MSL, which is 2 feet below the mat foundation bearing depth), which includes the 6 inches of surface removal] of existing soils should be removed. The exposed subgrade should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill or clean stabilized soils. After proof-rolling, compacted select fill or clean, stabilized soils should then be used to achieve the bottom of the mat foundation (at an elevation of 62.2 feet above MSL). Select fill should be in accordance with Section 02320 of the latest edition of the COHSCS.

According to Table 4, the PVR for 4 feet of soil replacement is 0.9 inches, which is less than 1 inch. The Owner should be aware that the risk of mat foundation movement is still present if the subgrade soils are not uniform. If conditions which exacerbate moisture variations such as the presence of trees, poor drainage, excessive drying/wetting of subsurface soils, or leaking underground utilities are located nearby, the mat foundation total vertical movements and net differential vertical movements could be higher than estimated.

Moisture Barrier: We recommend that a horizontal moisture barrier (minimum 10-mil thick) be placed below the mat foundation to move edge effects away from the mat and mitigate seasonal fluctuations of water content directly below the structure.

5.3.2 Mat Excavation

We recommend that the exposed walls of mat foundation excavations be covered by a polyethylene membrane during construction. The excavation bottom must also be protected to prevent loss of moisture. We recommend that the exposed subgrade of the mat foundation excavation be covered by a minimum 2-inch thick lean concrete seal slab if the foundation will not be poured within 24 hours. Central to this recommendation is the importance of preserving the moisture regime that exists in the fat clays at the site. Maintaining a stable moisture condition is essential in minimizing swelling of the high plasticity fat clays at the site.

Excavations may be shored or laid back to a stable slope for the safety of workers, public, and adjacent structures. In addition, design, construction, and maintenance of shoring should be performed by qualified personnel under experienced supervision. The recommendations presented herein are intended to guide the Contractor in his design; the Contractor should be responsible for designing, installing and maintaining safe excavations.

In areas where the foundations are to be installed by open cut, excavations should be adequately sloped, shored or braced according to the OSHA's excavation safety standards, 29 CFR Part 1926, Subpart P (Excavation and Trenches) and applicable local regulations. Based on Boring B-7, in general, the top 10 feet of subsurface soils at the sodium bisulphite tank location can be classified as OSHA Type "C" for fat clay fill material.

5.4 Detention Swale

Based on the design drawings, the detention swales will be 2.2 to 2.5 foot deep with side slope inclination of H:V = 3:1. Top of bank of the swales is at an elevation of 62.0 feet MSL, and the bottom varies from an elevation of 59.8 to 58.5 feet MSL.

As requested by the COH Geo-Environmental Services Branch, AEC collected three samples (Samples S-1 through S-3) from the ground surface to a depth of 12 inches from within the perimeter of the proposed drainage swale. The locations of the samples are presented on Plate A-2, in Appendix A. Based on a visual classification the samples, Samples S-1 and S-2 consisted of lean/fat clay (CL/CH) fill, with abundant calcareous nodules and gravel. Sample S-3 consisted of lean clay (CL), with abundant silt partings. Based on the visual classification of surficial soils from within the perimeter of the drainage swale, a clay liner is not necessary. However, AEC recommends the swale slopes and bottom be lined with sodding, seeding, or hydro-mulching.

5.4.1 Swale Excavation

Since ground water was not encountered in Borings B-4 or B-5 during drilling, excavation of the detention swales will probably not require dewatering. However, ground water 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. Dewatering and ground water control, if needed, should be the Contractor's responsibility.

Seepage in the clay will probably be low. Seepage influx will be primarily from sand/silt seams, layers, and fissures. Gravity drainage with sumps can be effective in removing water seeping into excavations from these clayey soil zones.

5.5 Pavement

AEC understands that some of the existing gravel and asphalt driveways at the site will be reconstructed with concrete pavement. Based on the design drawings, the concrete pavement for the driveways will be 7 inches thick. The pavement design recommendations developed herein are in accordance with the "AASHTO Guide for Design of Pavement Structures," 1993 edition. AEC assumes that the traffic at the site will consist of passenger cars and pickup trucks, with occasional chemical and tanker trucks, although traffic loading and volume was not available to AEC at the time this report was prepared. Based on the provided drawings, the pavement will be at or near existing grade.

5.5.1 Rigid Pavement

Rigid pavement design is based on the anticipated design number of 18-kip Equivalent Single Axle Loads (ESALs) the pavement is subjected to during its design life. The parameters that were used in computing the rigid pavement section are as follows:

Overall Standard Deviation (S_0)	0.35
Initial Serviceability (P_0)	4.5

Terminal Serviceability (P_t) 2.5
Reliability Level (R) 85%
Overall Drainage Coefficient (C_d) 1.0
Load Transfer Coefficient (J) 3.2
Loss of Support Category (LS) 1.0
Roadbed Soil Resilient Modulus (M_R) 3,000 psi
Elastic Modulus (E_{sb}) of Stabilized Soils 20,000 psi
Composite Effective Modulus of Subgrade Reaction (k) 65 pci
Mean Concrete Modulus of Rupture (S'_c) 600 psi (at 28 days)
Concrete Elastic Modulus (E_c) 3.37×10^6 psi

Recommended rigid pavement sections are provided on Table 5 below.

Table 5. Recommended Rigid Pavement Section

Pavement Layer	Thickness (in)
Portland Cement Concrete	7
Lime-Stabilized Subgrade[1]	6

Note: Stabilized subgrade recommendations are presented in Section 5.5.2 of this report.

Given the above design parameters, the 7-inch thick concrete driveway section should sustain 833,910 repetitions of 18-kip ESALs. The design engineer should verify whether the proposed pavement sections will provide enough ESALs for the anticipated amount of site traffic. AEC should be notified if different standards or constants are required for pavement design at the site, so that our recommendations can be updated accordingly.

Concrete Pavement: Portland Cement Concrete (PCC) pavement should be constructed in accordance with Section 02751 of the latest edition of the COHSCS. According to the COHSCS, concrete mix design has a required flexural strength of 600 psi at 28 days, and field testing shall confirm a minimum concrete compressive strength of 3,500 psi at 28 days. The Contractor shall be responsible for ensuring that a concrete mix design based on concrete compressive strength of 3,500 psi at 28 days also meets a minimum concrete flexural strength of 500 psi at 7 days and 600 psi at 28 days.

Reinforcing Steel: Reinforcing steel is required to control pavement cracks, deflections across pavement joints and resist warping stresses in rigid pavements. The cross-sectional area of steel (A_s) required per foot of slab width can be calculated as follows (for both longitudinal and transverse steel).

$A_s = FLW/(2f_s)$ ....Equation (3)

where:	A_s	=	Required cross-sectional area of reinforcing steel per foot width of pavement, in^2
	F	=	Coefficient of resistance between slab and subgrade, F = 1.8 for stabilized soil
	L	=	Distance between free transverse joints or between free longitudinal edges, ft.
	W	=	Weight of pavement slab per foot of width, lbs/ft
	f_s	=	Allowable working stress in steel, 0.75 x (yield strength), psi i.e. f_s = 45,000 psi for Grade 60 steel.

5.5.2 Pavement Subgrade

Subgrade Preparation: Subgrade preparation should extend a minimum of 2 feet beyond the paved area perimeters. Existing asphalt, concrete, or gravel pavement (if any) along the proposed driveway alignments should be demolished and removed. After pavement demolition, we recommend that a minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials be removed and wasted. The excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths. The exposed soils should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill. Select fill should be in accordance with Section 02320 of the latest edition of the COHSCS.

Scarify the top 6 inches of the exposed subgrade and stabilize the underlying soils with at least 7 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 percentage of lime required for stabilization are preliminary estimates for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. 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.

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 Construction Monitoring

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

6.3 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 alignments.

7.0 GENERAL

AEC should be allowed to review construction documents and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

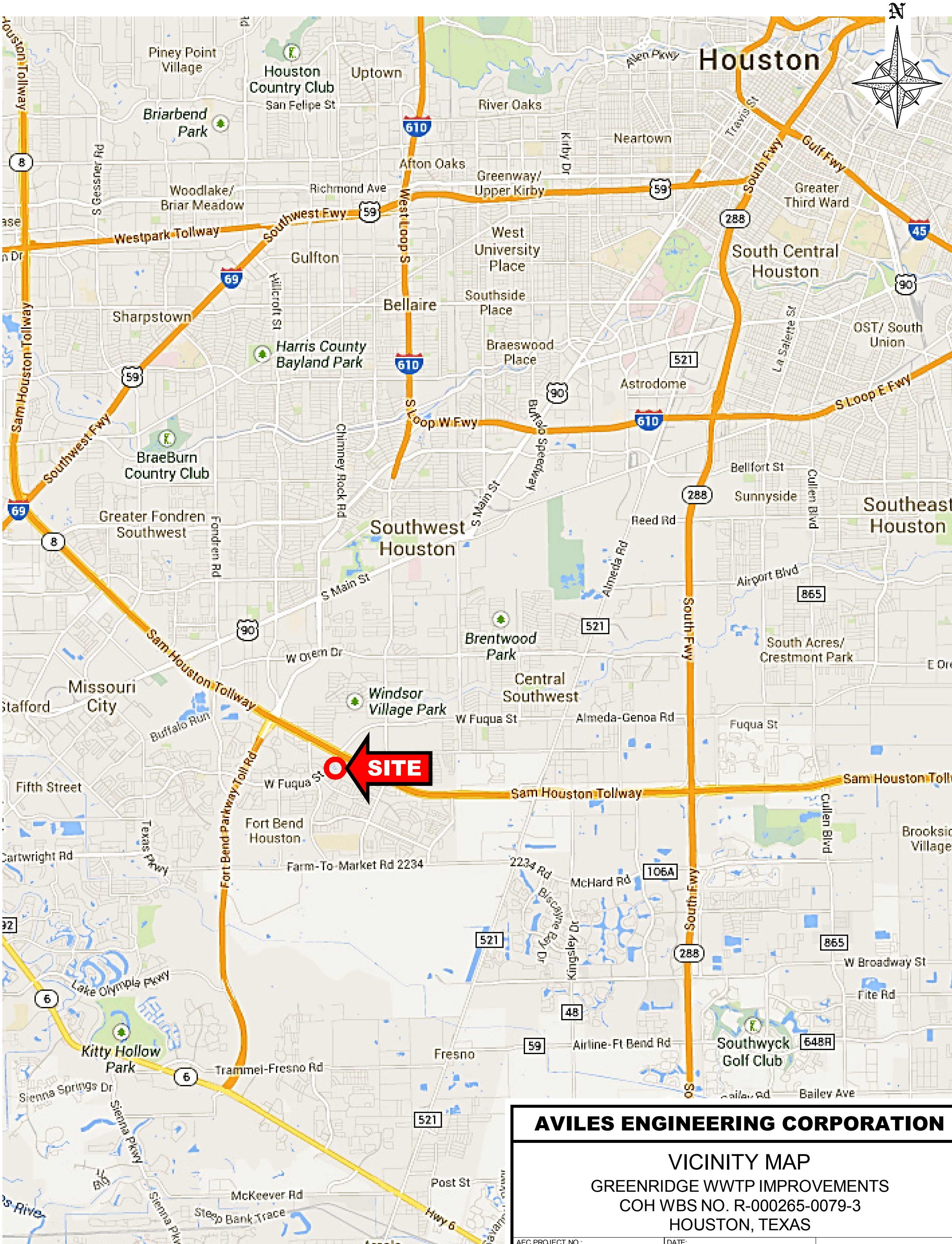
The information contained in this report summarizes conditions found on the date the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the date of drilling. Due to variations encountered in the subsurface conditions across the site, changes in soil conditions from those 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

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 to A-12	Boring Logs
Plate A-13	Key to Symbols
Plate A-14	Classification of Soils for Engineering Purposes
Plate A-15	Terms Used on Boring Logs
Plate A-16	ASTM & TXDOT Designation for Soil Laboratory Tests



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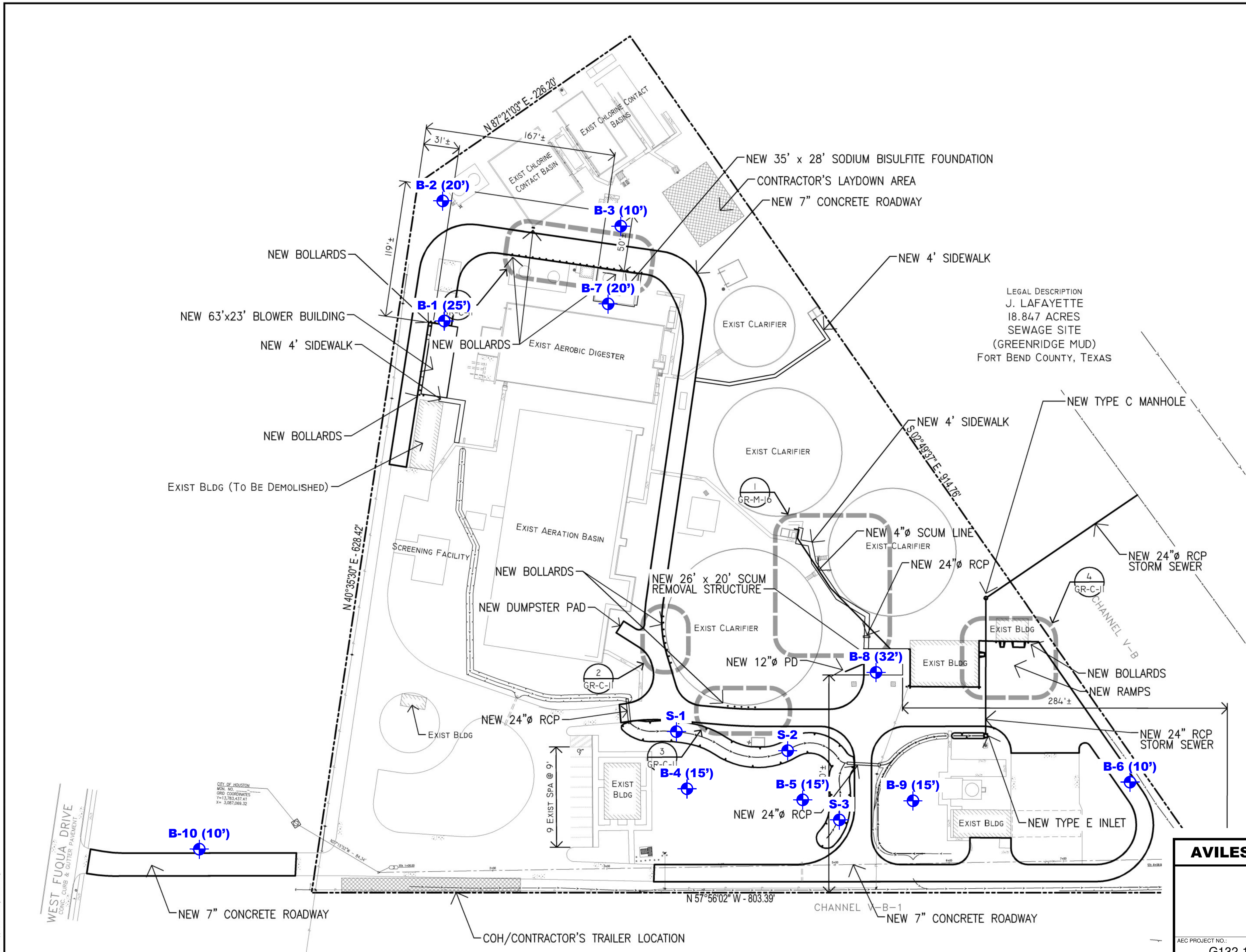
VICINITY MAP

GREENRIDGE WWTP IMPROVEMENTS
COH WBS NO. R-000265-0079-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	
G132-13	09-26-13	
APPROX. SCALE: N.T.S.	DRAFTED BY: BpJ	PLATE NO.: PLATE A-1



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F.B. No.: P-5861

CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS AND ENGINEERING

KEEGANS BAYOU & GREENRIDGE WWTP IMPROVEMENTS

GREENRIDGE WWTP SITE PLAN

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN
GREENRIDGE WWTP IMPROVEMENTS
WBS No. R-000265-0079-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G132-13	10-21-14	INFRASTRUCTURE ASSOC.
APPROX. SCALE: N.T.S.	DRAFTED BY: WIW	PLATE NO.: PLATE A-2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Greenridge WWTP Improvements** BORING **B-1**

DATE **6/5/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DESCRIPTION

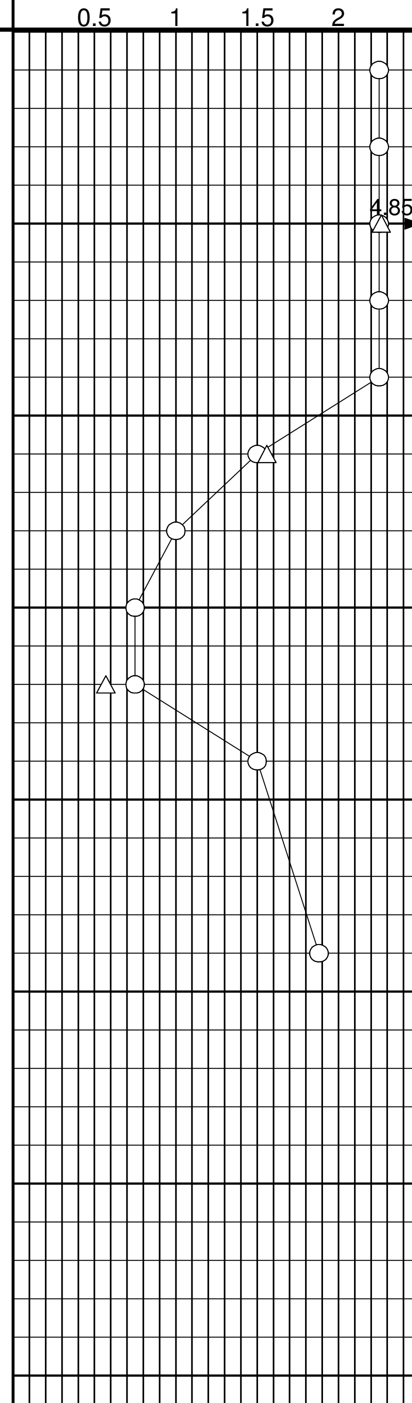
Survey Coordinates (ft):
Easting: 3,087,798.66
Northing: 13,785,470.63
Elevation: 63.24

DEPTH IN FEET	DESCRIPTION
0 – 8	Fill: hard, dark gray and tan Lean Clay w/ Sand (CL), with siltstone fragments, calcareous nodules, and ferrous stains -with roots 0'-2' -with gravel 2'-4'
8 – 12	Fill: very stiff to hard, dark gray and reddish brown Fat Clay (CH), with siltstone fragments, calcareous nodules, and ferrous stains -with sand layers 10'-12'
12 – 16	Fill: stiff to very stiff, brown, gray, and tan Lean Clay (CL), with sand pockets, siltstone fragments, and calcareous nodules -with silty clay seams 14'-16'
16 – 21	Stiff to very stiff, brown and light gray Fat Clay (CH), with slickensides, silt partings, calcareous nodules, and ferrous stains -with siltstone fragments 18'-20'
21 – 25	Very stiff, tan and light gray Lean Clay (CL), with abundant silt seams, siltstone fragments, and calcareous nodules
25	Termination Depth = 25 feet

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	10					
			85	48	16	32
	13					
	14	116				
	16					
			90	56	17	39
	19					
	19	105				
	24					
			85	42	14	28
	23					
	26	97				
	23					
	17					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **WLW** LOGGED BY **RJM**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Greenridge WWTP Improvements**

BORING **B-2**

DATE **6/5/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

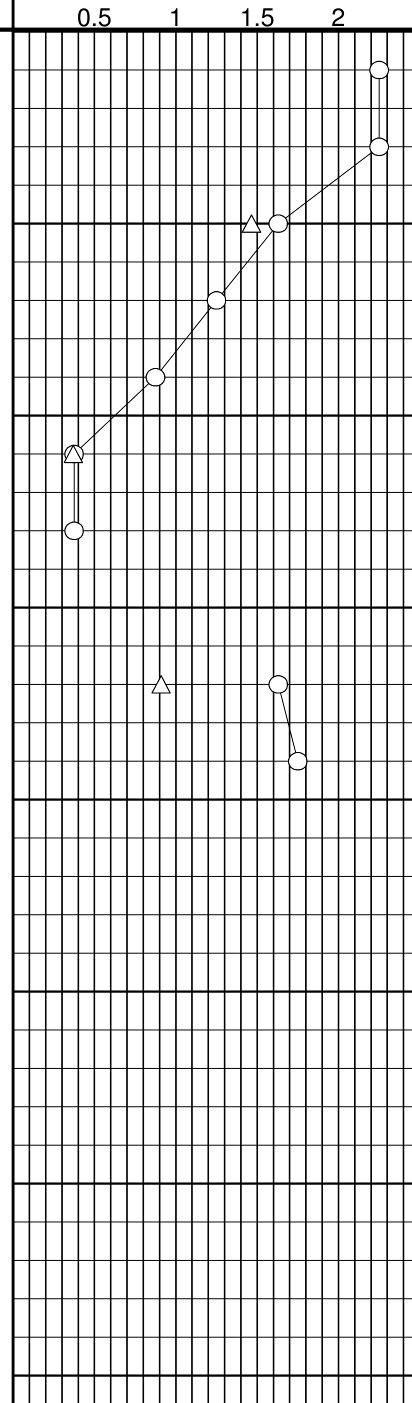
Survey Coordinates (ft):
Easting: 3,087,857.06
Northing: 13,785,564.01
Elevation: 62.73

DEPTH IN FEET	DESCRIPTION
0	Fill: firm to hard, dark gray Fat Clay (CH), with shells, calcareous nodules, and ferrous stains -with roots 0'-2' -dark gray, tan, and brown 2'-4', with siltstone fragments and sand seams 2'-10'
5	-tan and dark gray 6'-10'
10	-gray and tan, with sand pockets 10'-14'
15	Fill: gray and tan Sandy Fat Clay (CH), with abundant sand seams Stiff to very stiff, tan and gray Fat Clay w/ Sand (CH), with slickensides
20	Termination Depth = 20 feet
25	
30	
35	

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	10					
			85	53	14	39
	17					
	17	111				
	21					
			86	59	18	41
	24					
	26	100				
	23					
			52			
	34					
			83	62	18	44
	24	95				
	23					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **14.3** FEET WHILE DRILLING
WATER LEVEL AT **14.5** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **WLW** LOGGED BY **RJM**

PROJECT NO. G132-13 **PLATE A-4**

PROJECT: Greenridge WWTP Improvements

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING B-3

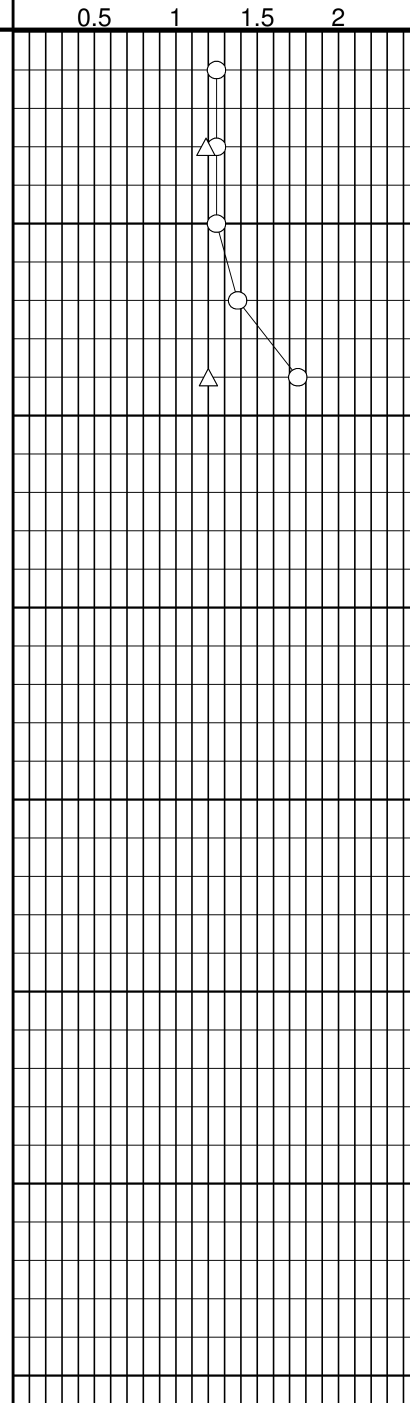
DATE 6/5/13 **TYPE** 4" Dry Auger **LOCATION** See Boring Location Plan

Survey Coordinates (ft):
Easting: 3,087,974.70
Northing: 13,785,461.59
Elevation: 63.52

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: very stiff, dark gray Fat Clay (CH), with shell and sand seams		26					
	Very stiff, dark gray Fat Clay (CH), with slickensides and ferrous stains		20	109				
					88	55	18	37
5			21					
	-light gray and tan, with siltstone fragments and calcareous nodules 6'-10'		20					
			20	111				
10	Termination Depth = 10 feet							

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 10 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT n/a FEET WHILE DRILLING
WATER LEVEL AT n/a FEET AFTER COMPLETE
DRILLED BY V&S CHECKED BY WLW LOGGED BY RJM

PROJECT: **Greenridge WWTP Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-4**

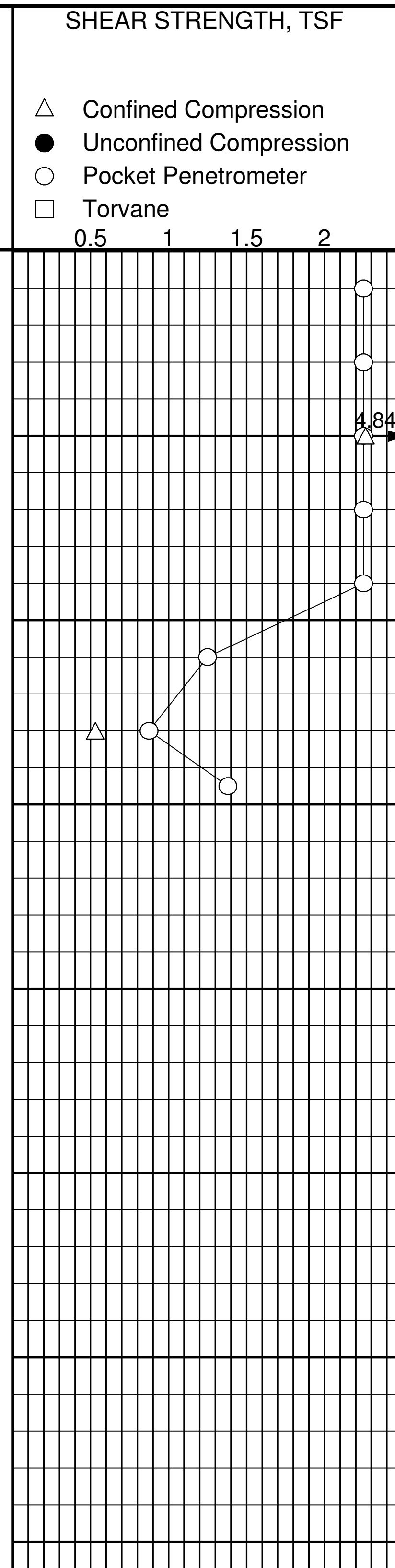
DATE **6/5/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (ft):
Easting: 3,087,763.74
Northing: 13,785,013.23
Elevation: 64.09

DEPTH IN FEET	DESCRIPTION
0 – 2	Fill: hard, dark gray and tan Fat Clay (CH), with shell, sandy clay seams, roots, siltstone fragments, and calcareous nodules
2 – 10	Hard, dark gray Fat Clay w/Sand (CH), with ferrous stains -with calcareous nodules 6'-10' -light gray and tan, with siltstone fragments and calcareous nodules 8'-10'
10 – 14	Stiff to very stiff, tan and light gray Lean Clay (CL), with abundant silt partings, calcareous nodules, and ferrous stains -with siltstone fragments 10'-12'
14 – 15	Very stiff, reddish brown and light gray Fat Clay (CH), with silty clay seams, silt pockets, and calcareous nodules
	Termination Depth = 15 feet

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	12					
	13					
	15	121	79	54	15	39
	15					
	16					
	23		95	39	17	22
	24	102				
	27					



BORING DRILLED TO **15** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **WLW** LOGGED BY **RJM**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Greenridge WWTP Improvements** BORING **B-5**

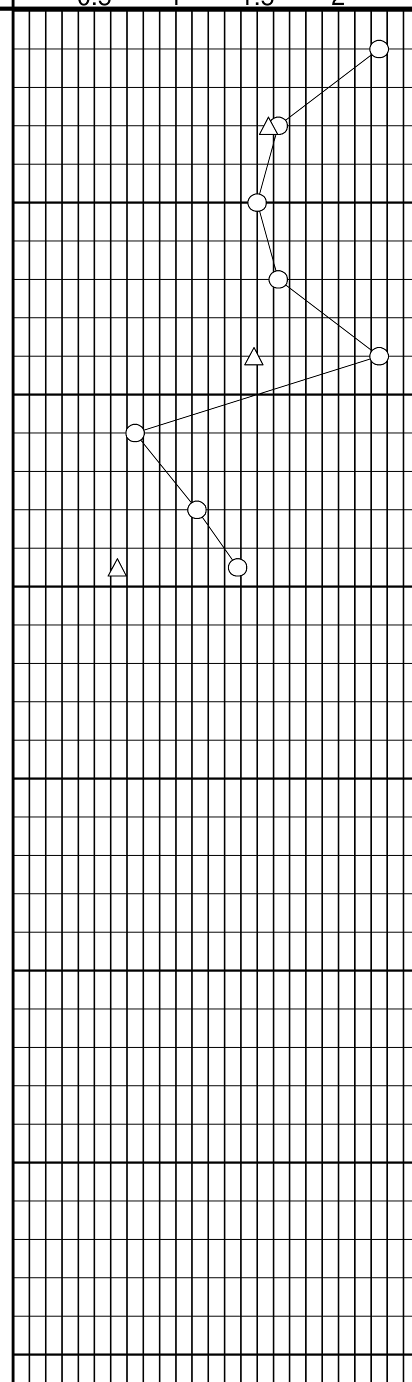
DATE **6/5/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3,087,844.44
Northing: 13,784,952.33
Elevation: 63.08

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: dark gray Clayey Sand (SC), with shell, siltstone fragments, and calcareous nodules		14		17	42	15	27
2	Very stiff to hard, dark gray Fat Clay w/Sand (CH), with ferrous stains		20	104				
5			22					
6	-tan and light gray 6'-8', with siltstone fragments and calcareous nodules 6'-10'				81	61	17	44
7			19					
8	-tan, red, and light gray 8'-10'							
9			16	117				
10	Stiff to very stiff, red, tan, and light gray Lean Clay (CL), with abundant silt partings, siltstone fragments, and calcareous nodules				95	41	16	25
11			25					
12	-with ferrous stains 12'-14'							
13			26					
14	Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with silty clay seams, siltstone fragments, and calcareous nodules		23	102				
15	Termination Depth = 15 feet							

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **15** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **WLW** LOGGED BY **RJM**

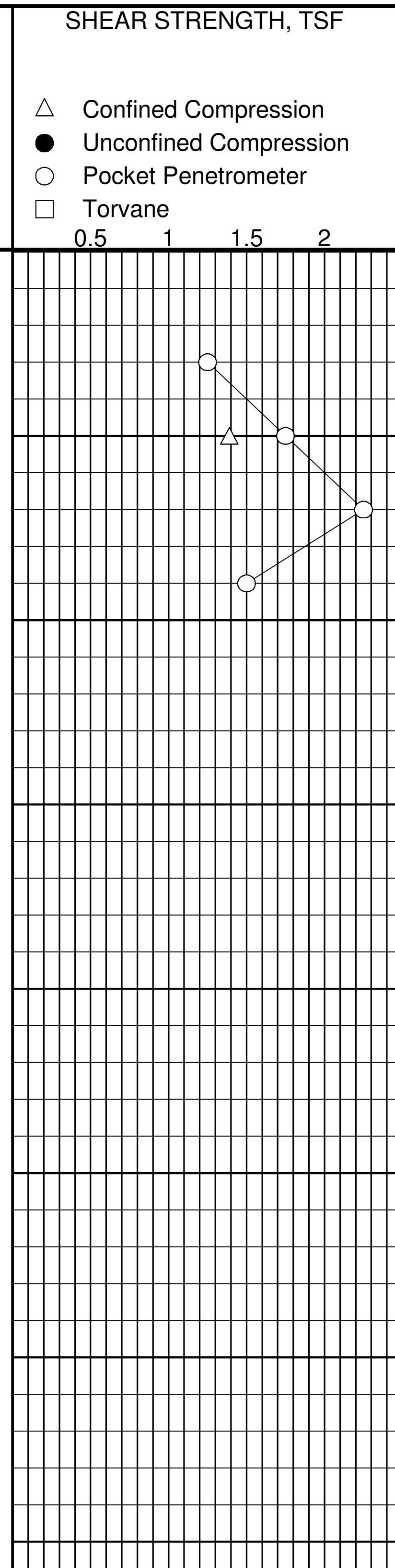
PROJECT: **Greenridge WWTP Improvements** — **AVILES ENGINEERING CORP.** GEOTECHNICAL ENGINEERS — BORING **B-6**

DATE **6/5/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (ft):
Easting: 3,088,096.87
Northing: 13,784,812.73
Elevation: 62.35

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 4" asphalt							
	Base: 7" asphalt stabilized gravel		3					
	Fill: light gray Fat Clay (CH), with asphalt pieces, gravel, and calcareous nodules				82	56	17	39
	Very stiff to hard, dark gray Fat Clay w/Sand (CH), with ferrous stains		23					
5			21	103				
	-tan and light gray, with siltstone fragments and calcareous nodules 6'-10'		16					
			19					
10	Termination Depth = 10 feet							



BORING DRILLED TO **10** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **WLW** LOGGED BY **RJM**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Greenridge WWTP Improvements** BORING **B-7**

DATE **4/21/14** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

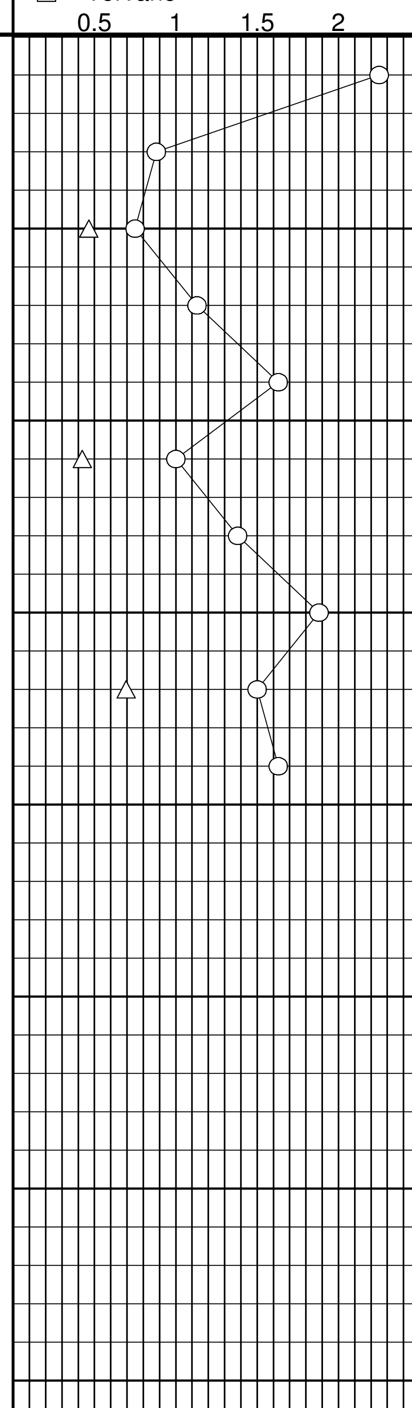
DESCRIPTION

Survey Coordinates (ft):
Easting: 3,087,929.84
Northing: 13,785,411.31
Elevation: 64.20

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: firm to hard, dark brown Fat Clay (CH) -with calcareous nodules 0'-6' -with gravel 0'-2' -dark brown and red 2'-6'		16					
			22		86	51	16	35
5			21	102				
	-dark brown and gray 6'-10'		20					
	-with calcareous nodules 8'-12'		20		89	55	16	39
10	-with gravel 10'-12'		24	101				
	Stiff to very stiff, red, tan, and gray Fat Clay (CH), with slickensides -with ferrous and calcareous nodules 12'-14'		26		98	51	22	29
15			24					
	-tan and gray 16'-18'		25	102				
	-with calcareous nodules 18'-20'		24					
20	Termination depth = 20 feet.							
25								
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **JOHNSON** CHECKED BY **WLW** LOGGED BY **CHL**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Greenridge WWTP Improvements** BORING **B-8**

DATE **4/21/14** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

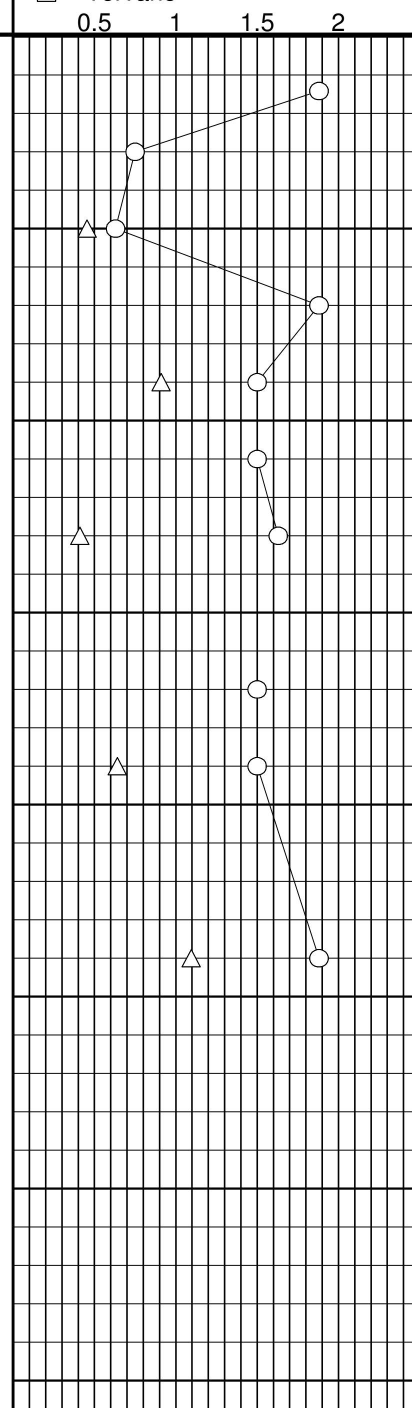
DESCRIPTION

Survey Coordinates (ft):
Easting: 3,087,962.83
Northing: 13,785,022.82
Elevation: 62.94

Depth in Feet	Description	S.P.T. Blows / Ft.	Moisture Content, %	Dry Density, PCF	-200 Mesh	Liquid Limit	Plastic Limit	Plasticity Index
0	Pavement: 2" asphalt		5					
	Base: 8" crushed gravel with clayey sand		21		80	42	17	25
	Very stiff, dark brown Lean Clay w/Sand (CL), with silty sand pockets		24					
5	Firm to very stiff, dark brown Fat Clay w/Sand (CH) -with silty sand pockets 2'-4' -with ferrous nodules 4'-6' -gray and tan, with calcareous nodules 6'-10', with silty clay pockets 6'-8'		31	89				
			22		80	50	17	33
			16	114				
10	Firm to very stiff, tan and gray Lean Clay w/ Sand (CL), with calcareous nodules and silt partings -with fat clay pockets 10'-12'		20		77	30	17	13
			22	109				
15	Firm to very stiff, red and tan Fat Clay (CH), with slickensides -with calcareous nodules 14'-20' -red and brown, with silty sand pockets 16'-18' -dark tan 18'-20'	7	22					
			27		97	56	23	33
			27	100				
20	-red and gray, with ferrous nodules 23'-25'		21	107				
25								
	Medium dense, tan Silty Sand (SM), wet		23		21	24	22	2
30		13	24					
	Termination depth = 32 feet.							
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



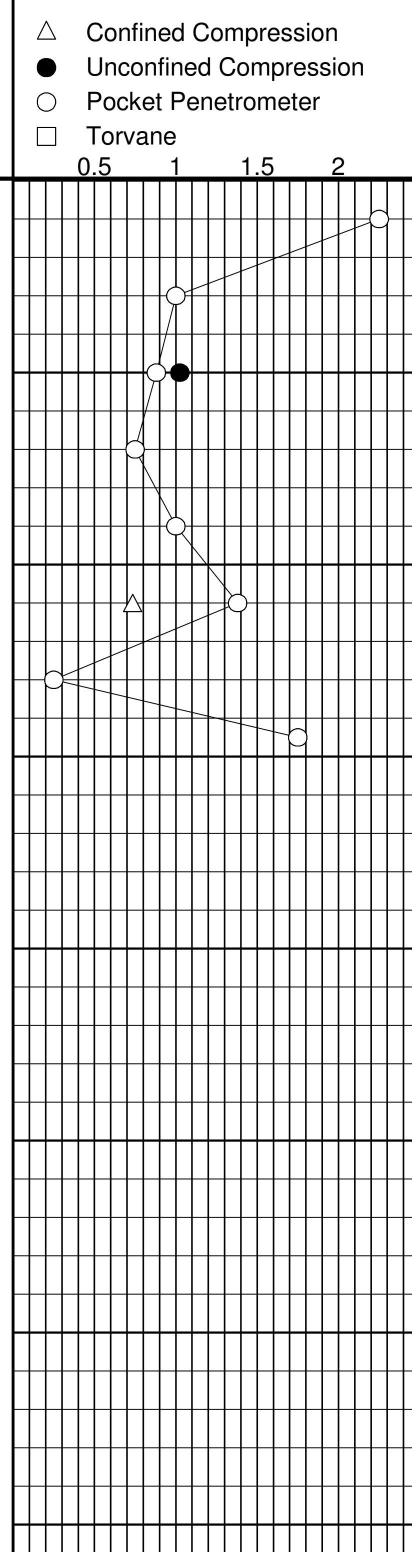
BORING DRILLED TO **32** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **28** FEET WHILE DRILLING
WATER LEVEL AT **18.5** FEET AFTER **COMPLETE**
DRILLED BY **JOHNSON** CHECKED BY **WLW** LOGGED BY **CHL**

PROJECT: **Greenridge WWTP Improvements**　BORING **B-9**

DATE **4/21/14**　TYPE **4" Dry Auger**　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
			Survey Coordinates (ft): *Easting:* 3,087,926.91 *Northing:* 13,784,900.47 *Elevation:* 63.31							
0			Fill: hard, brown and dark brown Sandy Lean Clay (CL), with sand seams, calcareous nodules, and gravel		11					
							65	41	17	24
			Fill: stiff to very stiff, dark brown Lean Clay w/ Sand (CL), with sand pockets		14					
5			-with roots and gravel 2'-4' -with ferrous nodules and shells 4'-6'		20	108				
							77	48	18	30
			-with sand seams and calcareous nodules 6'-8'		23					
			Soft to very stiff, gray and tan Lean Clay (CL), with slickensides		23					
10			-with fat clay partings and ferrous nodules 8'-10' -with silt partings 10'-14'		21	107				
							99	36	18	18
			-with fat clay pockets and calcareous nodules 12'-15'		25					
			-red and brown, with silt pockets 14'-15'		25					
15			Termination depth = 15 feet.							
20										
25										
30										
35										

SHEAR STRENGTH, TSF



BORING DRILLED TO **15** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **JOHNSON**　CHECKED BY **WLW**　LOGGED BY **CHL**

PROJECT: **Greenridge WWTP Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-10**

DATE **4/21/14** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3,087,782.75
Northing: 13,785,199.82
Elevation: 63.98

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	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Base: 12" silty sand with gravel, roots, and clay pockets		4						
	Fill: very stiff, dark brown and gray Lean Clay w/Sand (CL), with gravel, silty sand partings, and fat clay pockets		15		○ 2	77	35	15	20
			19	109	● 1.1; ○ 1.5				
5	Stiff to very stiff, gray and light olive gray Fat Clay (CH), with calcareous and ferrous nodules -gray and tan 4'-10'		19		○ 2				
			19						
			23		○ 1.5				
10	Termination depth = 10 feet.								

BORING DRILLED TO **10** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **n/a** FEET WHILE DRILLING
WATER LEVEL AT **n/a** FEET AFTER **COMPLETE**
DRILLED BY **JOHNSON** CHECKED BY **WLW** LOGGED BY **CHL**

KEY TO SYMBOLS

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

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

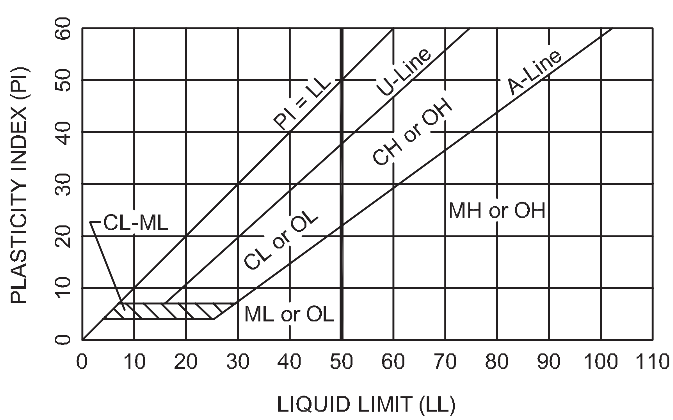
AVILES ENGINEERING CORP.

ASTM Designation D-2487

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

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

PLASTICITY CHART



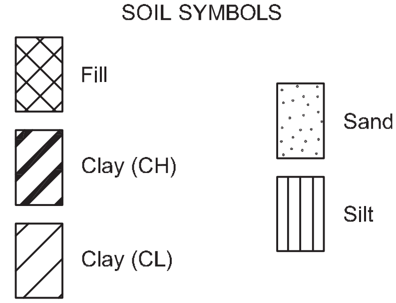
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

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

DEGREE OF PLASTICITY OF COHESIVE SOILS

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

SOIL SYMBOLS



AVILES ENGINEERING CORP.

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 | 76.2 | 19.1 | 4.76 | 2.00 | 0.420 | 0.074 | 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

DRY STRENGTH ASTM D2488

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

MOISTURE CONDITION ASTM D2488

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

SOIL STRUCTURE

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

AVILES ENGINEERING CORP.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	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
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Direct Shear	D 3080	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Pinhole Test	D 4647	-
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

AVILES
ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

KEEGANS BAYOU WWTP IMPROVEMENTS
WBS NO. R-000265-0079-3
HOUSTON, TEXAS

Reported to:

Infrastructure Associates
Houston, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G131-13R2

October 2014

October 8, 2014

Mr. Eric Cardwell, P.E.
Infrastructure Associates, Inc.
6117 Richmond Avenue, Suite 200
Houston, Texas 77057

Reference: Geotechnical Investigation
Keegans Bayou Wastewater Treatment Plant Improvements
9500 White Chapel Lane
Houston, Texas
AEC Report No. G131-13

Dear Mr. Cardwell,

Aviles Engineering Corporation (AEC) is pleased to present this report of our geotechnical investigation for the above referenced project. This investigation was authorized via email by Mr. Eric Cardwell, P.E. of Infrastructure Associates, Inc., on May 16, 2013, based upon AEC Proposal No. G2013-04-14R1, dated May 2, 2013. The contents of this report supersede our previous geotechnical report for this project dated September 17, 2014.

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)

STATE OF TEXAS
WILBER L. WANG
99226
LICENSED
PROFESSIONAL ENGINEER

Wilber L. Wang, M.Eng., P.E.
Project Engineer

Shou Ting Hu, M.S.C.E., P.E.
Principal Engineer

Reports Submitted: 3 Infrastructure Associates, Inc.
1 File (electronic)

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TABLE OF CONTENTS

APPENDICIES

ENGINEERING CORP.

EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed City of Houston (COH) Keegans Bayou Wastewater Treatment Plant (WWTP) Improvements, located at 9500 White Chapel Lane, in Houston, Texas (Houston/Harris County Key Map: 530S). According to Infrastructure Associates, the proposed improvements include: (i) a new 20 foot inner diameter grit chamber with perimeter wall that is 15 feet above grade plus a 6.5 foot deep pit; (ii) a new elevated structural slab adjacent to the grit chamber; and (iii) two new elevated concrete channels supported on piers.

Our findings are summarized below:

- Based on Boring B-1, the subsurface soil conditions at the proposed grit chamber generally consist of approximately 2 feet of hard fat clay (CH) fill at the ground surface, underlain by stiff to hard fat/lean clay (CH/CL) to the boring termination depth of 40 feet below grade.

- Details of the soils encountered during drilling are presented on the representative boring log. The cohesive soils encountered in Boring B-1 have Liquid Limits (LL) ranging from 47 to 58 and Plasticity Indices (PI) ranging from 29 to 40. This indicates that the cohesive soils have high expansive potential. The cohesive soils encountered are classified as "CL" and "CH" type soils in accordance with ASTM D 2487.

- Groundwater was encountered at a depth of 30 feet during drilling and subsequently was observed at a depth of 21.8 feet approximately 15 minutes after the initial encounter in Boring B-1. Groundwater at the grit chamber site could be pressurized.

- We did not detect any visual evidence or odor indicating the presence of hazardous materials in the soil samples. However, AEC notes that the presence of potential hazardous material within the project area cannot be discounted based upon the very small and limited number of samples taken.

- Recommendations for design and construction of the grit chamber are presented in Section 5.1 of this report.

- Recommendations for design and construction of the elevated structural slab and elevated concrete channels are presented in Section 5.2 of this report.

- Recommendations for controlling ground water during construction are presented in Section 6.2 of this report.

- This Executive Summary provides an overview of the geotechnical investigation and should not be used without the full text of this report.

ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

KEEGANS BAYOU WWTP IMPROVEMENTS
WBS NO. R-000265-0079-3
HOUSTON, 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 City of Houston (COH) Keegans Bayou Wastewater Treatment Plant (WWTP) Improvements, located at 9500 White Chapel Lane, in Houston, Texas (Houston/Harris County Key Map: 530S). A vicinity map is presented on Plate A-1 in Appendix A. According to Infrastructure Associates, the proposed improvements include: (i) a new 20 foot inner diameter grit chamber with perimeter wall that is 15 feet above grade plus a 6.5 foot deep pit; (ii) a new elevated structural slab adjacent to the grit chamber; and (iii) two new elevated concrete channels supported on piers.

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 design and construction of the grit chamber, elevated structural slab, and elevated concrete channels. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling one soil boring to 40 feet below existing grade;
2. Performing soil laboratory testing on selected soil samples;
3. Engineering analysis and recommendations for the grit chamber, including foundation type and depth, allowable bearing capacity, and lateral earth pressure parameters for pit wall design;
4. Engineering analyses and recommendations for the elevated structural slab and elevated concrete channels, including foundation type and depth, and allowable bearing capacity;
5. Construction recommendations for the grit chamber, elevated structural slab, and elevated concrete channels.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling one boring to a depth of 40 feet below existing grade at the proposed grit chamber location. Boring survey data is presented on the representative boring log.

The boring location is shown on the attached Boring Location Plan on Plate A-2, in Appendix A. The boring was drilled using a truck-mounted drill rig and advanced initially by dry auger method, 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. 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. After completion of drilling, the boring was backfilled with bentonite chips. Details of the soils encountered in the 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 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 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-4 through A-7, in Appendix A.

4.0 SITE CONDITIONS

4.1 Subsurface Conditions

Soil Conditions: Based on Boring B-1, the subsurface soil conditions at the proposed grit chamber generally consist of approximately 2 feet of hard fat clay (CH) fill at the ground surface, underlain by stiff to hard fat/lean clay (CH/CL) to the boring termination depth of 40 feet below grade.

Soil Properties: Details of the soils encountered during drilling are presented on the representative boring log. The cohesive soils encountered in Boring B-1 have Liquid Limits (LL) ranging from 47 to 58 and Plasticity Indices (PI) ranging from 29 to 40. This indicates that the cohesive soils have high expansive potential. The cohesive soils encountered are classified as "CL" and "CH" type soils in accordance with ASTM D 2487. "CH" soils can undergo significant volume changes due to seasonal changes in 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. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes.

Groundwater: Groundwater was encountered at a depth of 30 feet during drilling and subsequently was observed at a depth of 21.8 feet approximately 15 minutes after the initial encounter in Boring B-1. Groundwater at the grit chamber site could be pressurized. The information in this report summarizes conditions found on the date the boring was drilled. However, it should be noted that our ground water observations are short term; ground water 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

We did not detect any visual evidence or odor indicating the presence of hazardous materials in the soil samples. However, AEC notes that the presence of potential hazardous material within the project area cannot be discounted based upon the very small and limited number of samples taken.

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 away from and in between borings.

Clay soils in the Houston area typically have secondary features such as slickensides 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 which were generally continuously obtained at intervals of 2 feet from the ground surface to a depth of 20 feet, then at 5 foot intervals thereafter until the boring termination depth of 40 feet was reacted.

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 some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

According to Infrastructure Associates, the proposed improvements include: (i) a new 20 foot inner diameter grit chamber with perimeter wall that is 15 feet above grade plus a 6.5 foot deep pit; (ii) a new elevated structural slab adjacent to the grit chamber; and (iii) two new elevated concrete channels supported on piers.

5.1 Grit Chamber

Based on design drawings (dated August 1, 2014) provided by Infrastructure Associates, the new grit chamber will have an inner diameter of 20 feet, with a 1 foot thick perimeter concrete wall (i.e. outer diameter of 22 feet). The majority of the grit chamber is above ground, although there will also be a pit that extends approximately 6 feet below grade. Cement-stabilized sand will be used to partially support the upper chamber and also as backfill against the pit walls. The top of wall elevation of the grit chamber is at elevation +84.09 feet Mean Sea Level (MSL), the top of floor slab elevation of the upper chamber is at elevation +71.07 feet MSL, existing grade is at approximately elevation +68.00 feet MSL, and the top of the pit floor slab is at elevation +63.07 feet MSL. Boring B-1 is at elevation +68.20 feet MSL.

Structural loads for the grit chamber were provided by Infrastructure Associates. The total empty dead weight of the grit chamber is 380 kips. The total long term load is 655 kips (including 25 percent sustained live loads plus normal water level) and the total short term load is 800 kips (including 100 percent sustained live loads plus normal water level).

5.1.1 Mat Foundation

According to the design drawings, the top of the pit floor will be at elevation +63.07 feet MSL and will be supported on a 2 foot thick concrete mat foundation bearing at elevation +61.07 feet MSL.

Allowable Bearing Capacity: Based on Boring B-1 (surface elevation +68.20 feet MSL), a mat foundation founded at elevation +61.07 feet MSL can be designed for a net allowable bearing capacity of 2,700 psf for sustained loads and 4,000 psf for total loads, based on a minimum factor of safety (FS) of 3 for sustained loads and 2 for total loads, whichever is critical should be used. The modulus of subgrade reaction for soils beneath the mat foundation can be taken as 75 pci.

Mat Settlement: A detailed settlement analysis of the grit chamber is beyond the scope of service of this report. However, since the grit chamber is partially below grade, the long-term consolidation settlement of the structure will be small, since the weight of the soils removed for the pit excavation will partially compensate the weight of the grit chamber structure and equipment.

5.1.2 Grit Chamber Pit

Lateral Earth Pressures: The magnitudes of the lateral earth pressures on the pit walls will depend on the type and density of the backfill, surcharge on the backfill, and hydrostatic pressure, if any. If the backfill is over-compacted or if highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure. Based on the drawings, cement-stabilized sand will be used to backfill the pit excavation. Cement-stabilized sand should be in accordance with Section 02321 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS).

Lateral pressure resulting from construction equipment, structural loads, or other surcharge on the top of the pit walls should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure should also be included in the design (while assuming the pit is drained). We recommend that at least 240 psf uniform surcharge pressure be considered for design of the walls.

According to the drawings, the pit walls will be cast-in-place reinforced concrete. As a result, the pit walls can be designed based on at-rest earth pressure. The at-rest earth pressure at depth z can be determined by Equation (1). The walls should consider short term and long term conditions, whichever condition is critical should be used for design. Lateral earth pressure parameters for the pit wall design are presented on Table 1. Based on the drawings, approximately 1 to 3 feet of cement-stabilized sand backfill will be placed around the pit perimeter. Since the amount of cement-stabilized sand backfill surrounding the pit walls is relatively small, AEC recommends that the in-situ soil parameters be used for pit wall design.

$$p_0 = (q_s+\gamma\ h_1+\gamma'\ h_2)K_0 + \gamma_w h_2 \qquad \text{...........Equation (1)}$$

where, p_0 = at-rest earth pressure, psf.
q_s = uniform surcharge pressure, minimum 240 psf.
γ, γ' = wet and buoyant unit weights of soil, pcf.
h_1 = depth from ground surface to groundwater table, feet.
h_2 = $z\text{-}h_1$, depth from groundwater table to point under consideration, feet.
Z = depth below ground surface, feet.
K_0 = coefficient of at-rest earth pressure.
γ_w = unit weight of water, 62.4 pcf.

Table 1. Design Soil Parameters for Pit Walls (Based on Boring B-1)

Elevation (ft)	Soil Type	γ (pcf)	γ' (pcf)	Short-Term					Long-Term				
				C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p
N/A	Cement Stabilized Sand	120	58	0	30	0.33	0.70	3.00	0	30	0.33	0.70	3.00
68 to 66	Fill: hard CH	120	58	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
66 to 58	Very stiff to hard CH	131	69	3000	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76

Notes: (1) γ = unit weight for soil above water level, γ' = buoyant unit weight for soil below water level.
(2) C = ultimate cohesion, φ = ultimate angle of internal friction.
(3) K_a = coefficient of active earth pressure, K_0 = coefficient of at-rest earth pressure, K_p = coefficient of passive earth pressure, for level backfill.
(4) AEC recommends the use of FS = 2 for passive earth pressure if it is to be used in the design.

Hydrostatic Uplift Resistance: The pit should be designed to resist hydrostatic uplift. For uplift design of underground structures, we recommend that the water level be assumed to be at the ground surface (while assuming the pit is drained and empty). If the dead weights of the structure plus the skin friction resistance of the subgrade soils are inadequate to resist uplift forces, toe extensions of the mat foundation may be constructed so that the effective weight of the soil above the extended mat can be utilized to resist the uplift forces. The unit buoyant weight of concrete can be taken as 90 pcf. The minimum recommended factors of safety against uplift should be 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. Design soil parameters for uplift design are included on Table 1 above. Recommended design criteria for uplift resistance are shown on Plate B-8, in Appendix B.

Pit Wall Backfill: If the pit excavation will be laid/stepped back, we recommend use of select fill as backfill behind the pit walls. The excavation area should extend a minimum of 2 feet horizontally beyond the mat foundation perimeter, then slope upwards at a H:V = 1:1 slope or flatter. Select fill should be in accordance with Section 02316 of the latest edition of the COHSCS.

5.1.3 Pit Excavation

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

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

Excavations 20 feet and Deeper: 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 Occupational Safety and Health Administration (OSHA), Safety and Health Regulations, 29 CFR, Part 1926. Firm to hard clays should be considered OSHA Class “B” soils, while fill soils and granular soils should be considered OSHA Class “C” soils. Submerged soils should be classified as OSHA Class “C” soils, unless dewatering is conducted to lower the ground water level below the excavation bottom. Based on Boring B-1, the 2 feet of hard fat clay fill at the ground surface can be classified as OSHA Class “C”, and below the fill, the very stiff to hard fat clay to a depth of 10 feet below grade can can be classified as OSHA Class “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 B-1, in Appendix B. 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 and shoring should be designed and constructed by qualified professionals in accordance with OSHA requirements.

Plate B-2, in Appendix B, presents the maximum (steepest) allowable slopes for OSHA Soil Types for excavations less than 20 feet.

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

<u>Computation of Bracing Pressures:</u> 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 (2) below, the design soil parameters are presented on Table 1 in Section 5.1.2 of this report.

$$p_a = (q_s + rh_1 + r' h_2)K_a - 2c\sqrt{K_a} + r_w h_2 \qquad \text{............Equation (2)}$$

where,

p_a	=	active earth pressure, psf.
K_a	=	coefficient of active earth pressure, see Table 1.
c	=	cohesion of clayey soils, see Table 1, c can be omitted for design.
q_s	=	uniform surcharge pressure, minimum 300 psf.
γ, γ	=	wet and buoyant unit weights of soil, pcf, see Table 1.
h_1	=	depth from ground surface to groundwater table, feet.
h_2	=	$z-h_1$, depth from groundwater table to point under consideration, feet.
z	=	depth below ground surface, feet.
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 B-4 through B-6, in Appendix B.

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 ground water is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate B-7, in Appendix B.

If the excavation extends below ground water, 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 ground water is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized ground water, the ground water table should be lowered at least 3 feet below the excavation bottom.

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

Protection of Excavation Walls and Bottom: We recommend that the exposed walls of the pit foundation excavations be covered by a polyethylene membrane. The excavation bottom must also be protected to prevent loss of moisture. We recommend that the exposed subgrade of the foundation excavation be covered by a minimum 2-inch thick lean concrete seal slab if the mat foundation will not be poured within 24 hours. Central to this recommendation is the importance of preserving the moisture regime of the subsurface soils, in order to minimize the shrink/swell potential of the high plasticity fat clays at the site.

5.2 Elevated Structural Slab and Influent/Effluent Channels

Infrastructure Associates's drawings indicate that the existing elevated structural slab and elevated influent/effluent channels are supported on drilled-and-underreamed footings, placed at approximately 10 feet below typical grade. The drawings indicate that typical grade is at an approximate elevation of +68.00 feet MSL, and that the footings are founded approximately at an elevation of +56.59 feet MSL. The existing elevated structural slab and elevated channels are at an elevation of approximately +75.65 to +76.65 feet MSL.

Structural loads for the influent and effluent channels were provided by Infrastructure Associates. The total empty dead weight is 26 kips per footing. The total long term load is 40 kips per footing (including 25 percent sustained live loads plus normal water level) and the total short term load is 50 kips per footing (including 100 percent sustained live loads plus normal water level).

Based on the highly expansive clay soils encountered in Boring B-1, AEC recommends that the new elevated structural slab and elevated concrete channels also be supported on drilled-and-underreamed footings, founded at a depth below the zone of seasonal moisture variation (typically 10 feet below grade in the Houston area). AEC recommends that the new drilled footings be founded at an elevation that matches the existing footing elevation.

5.2.1 Drilled-and-Underreamed Footings

Allowable Bearing Capacity: Based on Boring B-1 (surface elevation +68.20 feet MSL), drilled-and-underreamed footings founded at approximately 11.6 feet below existing grade (at an elevation of +56.60 feet MSL) should be designed for a net allowable bearing capacity of 3,800 psf for sustained loads and 5,700 psf for total loads, based on a minimum FS of 3 for sustained loads and 2 for total loads; whichever is critical should be used for design.

Vertical Reinforcement: To withstand uplift forces resulting from the shrink/swell movements of clay soils in the moisture active zone, each footing should contain reinforcing steel throughout its full length to sustain an uplift load of at least 60d kips, where "d" is the diameter of the shaft in feet.

Footing Spacing: To reduce stress overlap from adjacent footings and potential construction problems, the minimum edge-to-edge clear spacing between the underreams should not be less than 0.6 x diameter of the larger underream. New foundations should be spaced to reduce the potential of new foundations affecting existing foundations or the pit foundation (and vice versa) by placing the new foundations outside the bearing (stress) zone of existing foundations. The bearing (stress) zone can be defined by a line drawn downward from the outer edge of the existing foundation and inclined at an angle of 45 degrees to the vertical.

Footing Settlements: Based on the soil conditions encountered, we estimate that drilled-and-underreamed footings, designed and constructed as recommended in this report, will experience total settlements on the order of 1 inch.

Drilled-and-Underreamed Footing Construction: Drilled-and-underreamed footings should be constructed in accordance with Section 02465 of the latest edition of the COHSCS. A qualified geotechnical technician should check each footing excavation prior to placing concrete to insure that:

1) The footing has been constructed to the specified dimensions at the recommended depth and founded in the correct formation as indicated in this report;
2) The column is concentric with the pier cap/grade beam and drilled footing; and
3) Excessive cuttings, any soft or compressible materials, and ponded water are removed from the bottom of the excavation.

There is a possibility that slickensides and/or pockets/seams of sands/silts within the clay soils may make underreaming (belling) difficult, and result in potential sloughing or caving-in of the shaft excavation sidewalls during construction, particularly for underreams over 6 feet in diameter. We recommend that a maximum diameter ratio of bell to shaft not exceed 2.5 to 1. Although the groundwater level encountered in Boring B-1 are below the anticipated footing depth, the site's groundwater level will fluctuate with seasonal rainfall and other climatic events, and may be higher at the time of construction. If ground water is encountered within the cohesive soils during construction, sump pumps may be used to pump water out from the excavations and soft sediments should be removed. However, if significant sloughing or caving occurs during shaft excavation, further excavation should be stopped and a reduced bell/shaft ratio or even straight-sided shafts (matching the bell diameter) in combination with bentonite slurry and/or temporary casing may be necessary.

Placement of concrete should be accomplished immediately after excavation is completed to reduce potential for sloughing of the foundation soils. Footing excavations should not be left open overnight. No concrete should be placed without the prior approval of the Owner's Representative. New drilled footings should not be excavated within 2 bell diameters (edge to edge) of an open footing excavation, or one in which concrete has been placed in the preceding 24 hours, to prevent movement of fresh concrete from the recently filled footing to an adjacent unfilled footing.

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.

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 of this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, multiple staged wellpoint or deep wells with submersible pumps have generally proved successful. Generally, 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.

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 5.1.3 of this report.

6.3 Construction Monitoring

Site preparation (including clearing and proof-rolling), earthwork operations, foundation construction, and subgrade preparation 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 area should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience, and supervision) may impact ground movement in the vicinity of the 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 area.

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7.0 GENERAL

AEC should be allowed to review construction documents and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted. The information contained in this report summarizes conditions found on the date the boring was drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the date of drilling. Due to variations encountered in the subsurface conditions across the site, changes in soil conditions from those 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.

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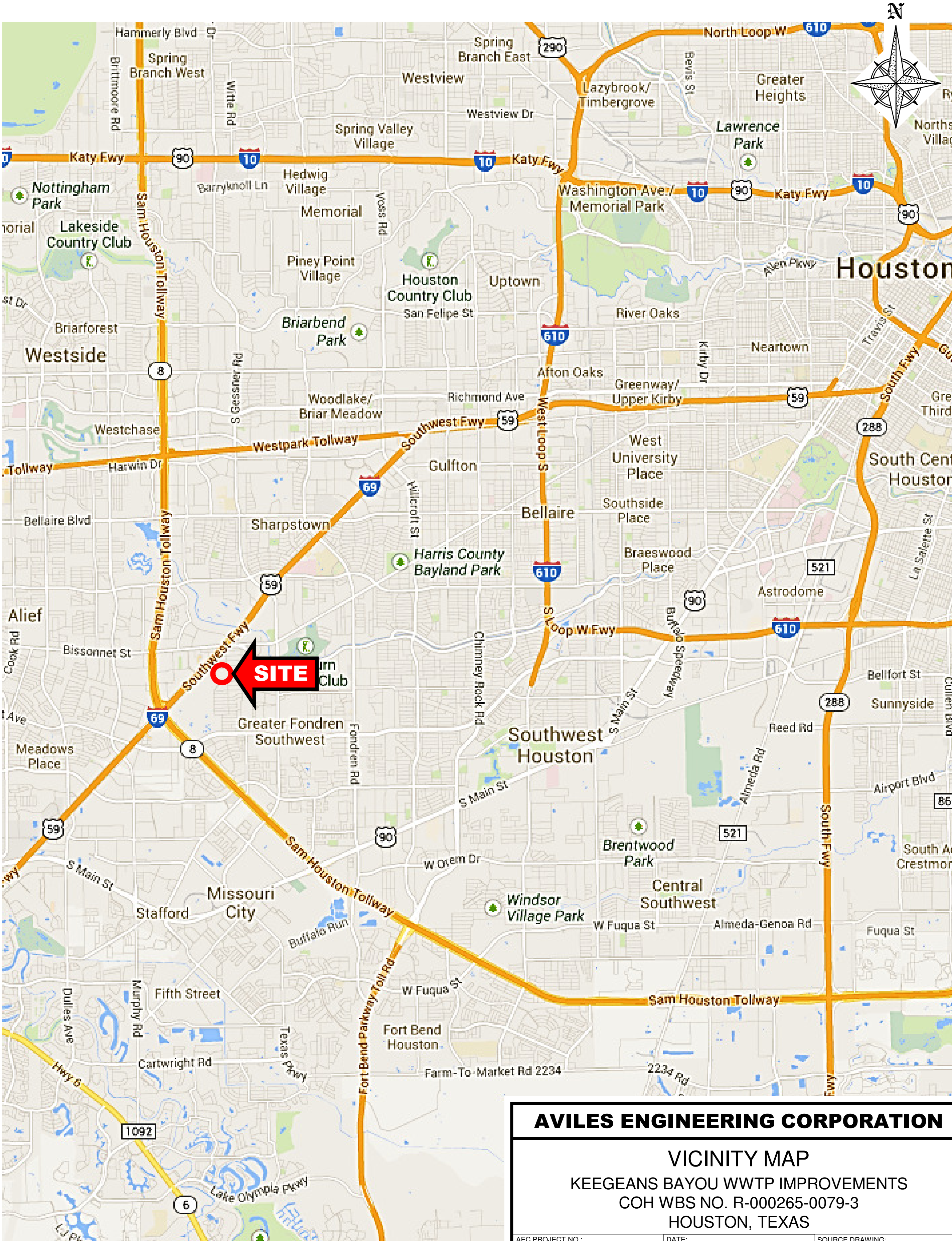
APPENDIX A



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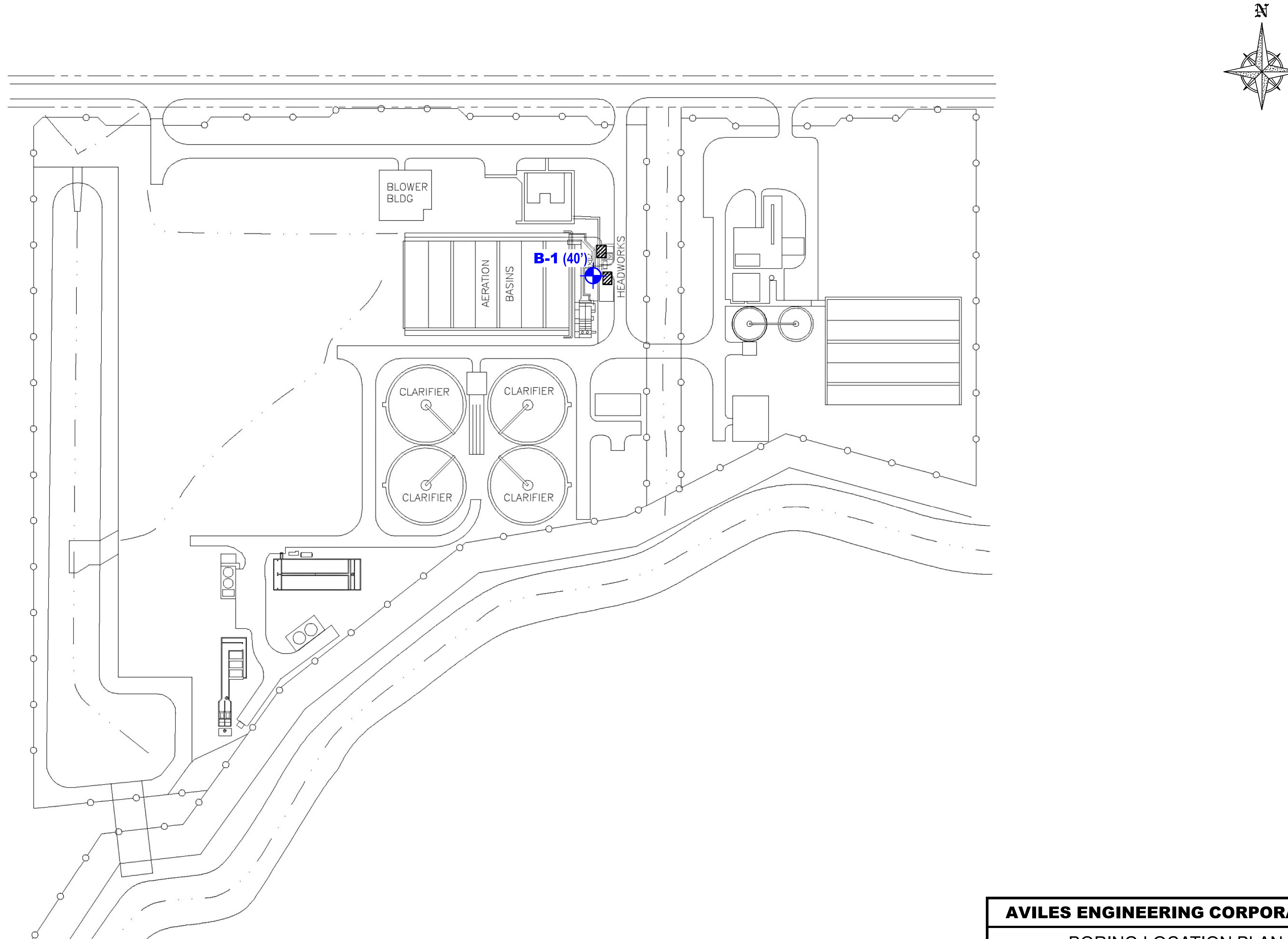
VICINITY MAP

KEEGEANS BAYOU WWTP IMPROVEMENTS
COH WBS NO. R-000265-0079-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING:
G131-13	09-26-13	COH GIMS
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
N.T.S.	BpJ	PLATE A-1



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

KEEGEANS BAYOU WWTP IMPROVEMENTS
COH WBS NO. R-000265-0079-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G131-13	09-26-13	INFRASTRUCTURE ASSOC.
APPROX. SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2

PROJECT: **Keegans Bayou WWTP Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-1**

DATE **6/7/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (TSPC, Surface):

Easting: 3066643.29

Northing: 13808992.43

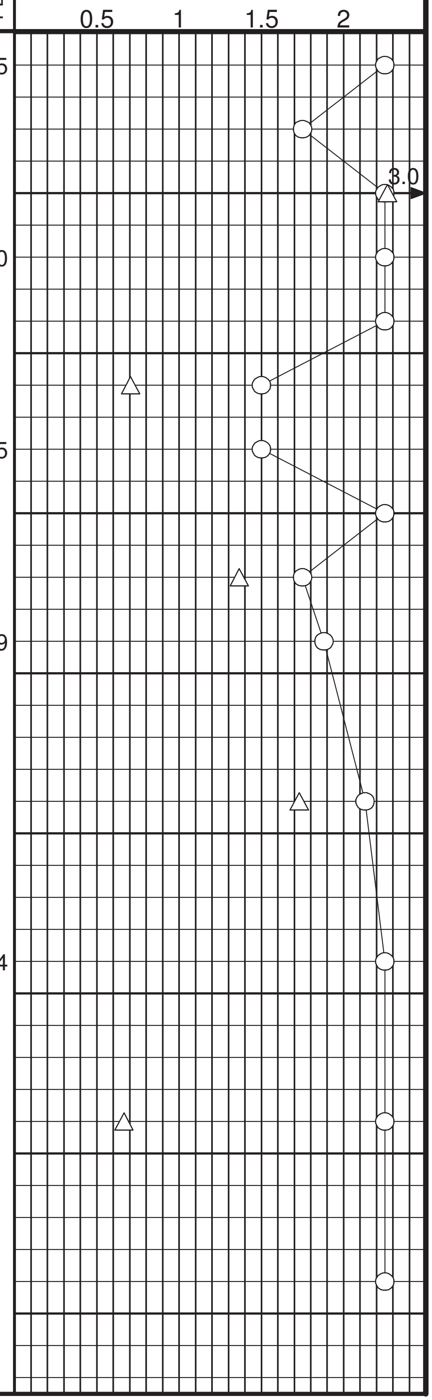
Elevation: 68.20

WBS No. R-000265-0079-3

ELEVATION IN FEET	DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	0	Fill: hard, dark gray Fat Clay w/Sand (CH), with calcareous nodules, sand pockets, shell, roots, and ferrous stains		71		12	51	16	35
66		Very stiff, dark gray Fat Clay (CH)				21			
		Hard, dark gray and gray Fat Clay w/Sand (CH), with calcareous nodules, siltstone fragments, and ferrous stains			113	16			
	6	-with sand pockets 6'-8'		76		17	58	18	40
60		-tan and light gray, with abundant calcareous nodules 6'-10'				21			
		Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with slickensides, silt pockets, and calcareous nodules			94	27			
	12	Very stiff, light gray and tan Fat Clay (CH), with sand pockets and ferrous stains		86		21	52	17	35
54		Very stiff to hard, light gray and tan Lean Clay w/Sand (CL), with sand partings, calcareous nodules, and ferrous stains				17			
					109	18			
	18			82		16	47	18	29
48									
	24				123	14			
42		Hard, light gray and tan Lean Clay (CL), with slickensides and sand partings		91		16	48	14	34
	30								
36		Stiff to hard, reddish brown and light gray Fat Clay (CH), with slickensides			105	24			
	36								
30		-with calcareous nodules and siltstone fragments 38'-40'							
		Termination depth = 40 feet.							
	42								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **30** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **30** FEET WHILE DRILLING

WATER LEVEL AT **21.8** FEET AFTER **15 MIN.**

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

PROJECT NO. G131-13

KEY TO SYMBOLS

Symbol Description

Strata symbols

Fill

High plasticity clay

Low plasticity clay

Misc. Symbols

Water table depth during drilling

Subsequent water table depth

Pocket Penetrometer

Confined Compression

Soil Samplers

Undisturbed thin wall Shelby tube

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

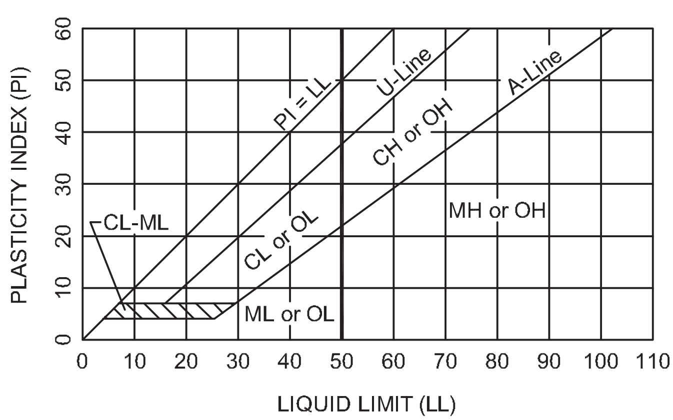
AYLES ENGINEERING CORP.

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

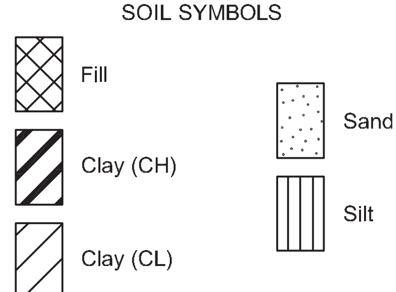


PLATE A-5

AVILES ENGINEERING CORP.

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

Sieve boundaries: 6" 3" 3/4" #4 #10 #40 #200

Grain size boundaries (mm): 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.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

DRY STRENGTH ASTM D2488

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

MOISTURE CONDITION ASTM D2488

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

SOIL STRUCTURE

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

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	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
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Direct Shear	D 3080	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Pinhole Test	D 4647	-
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

AVILES
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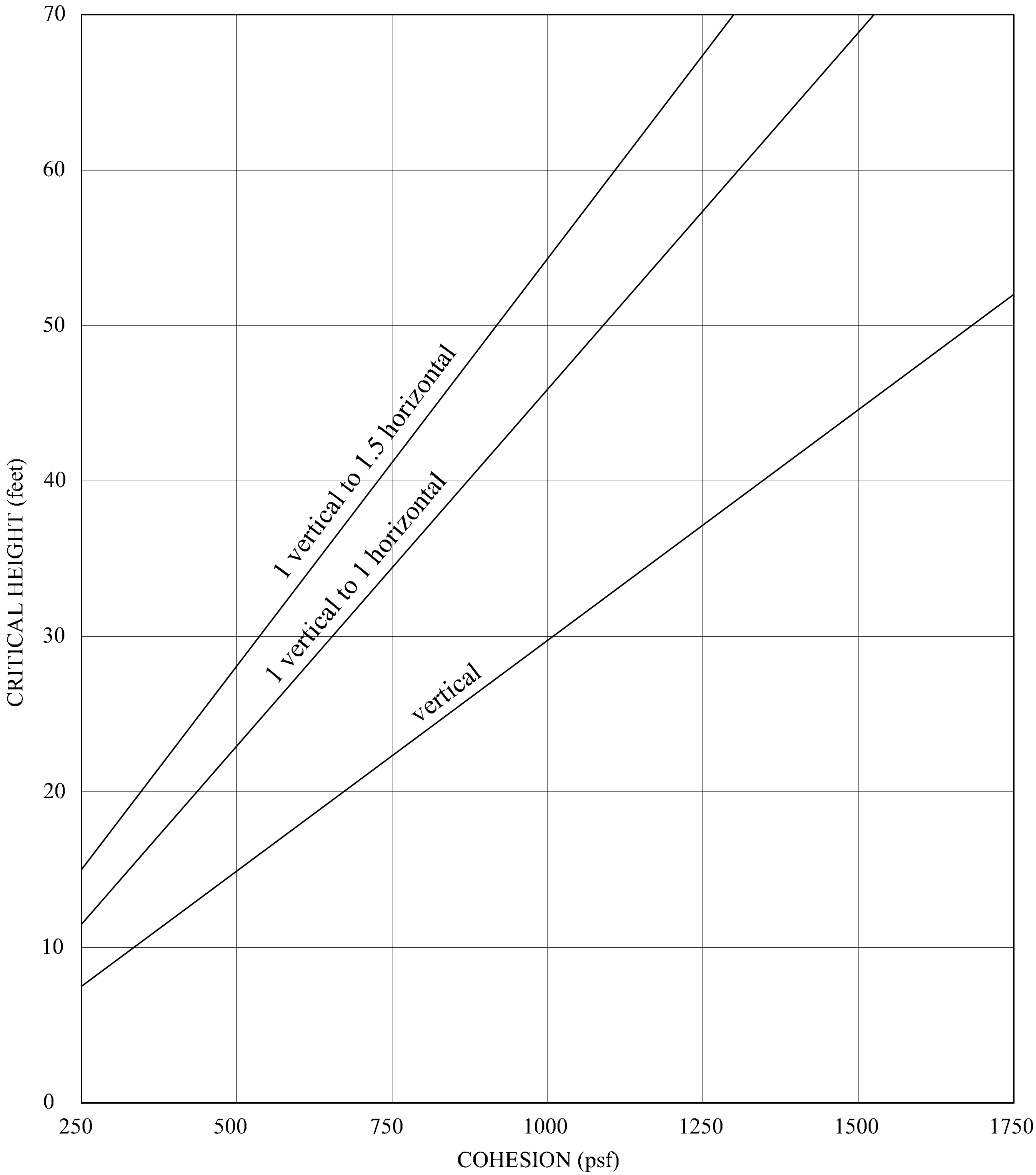
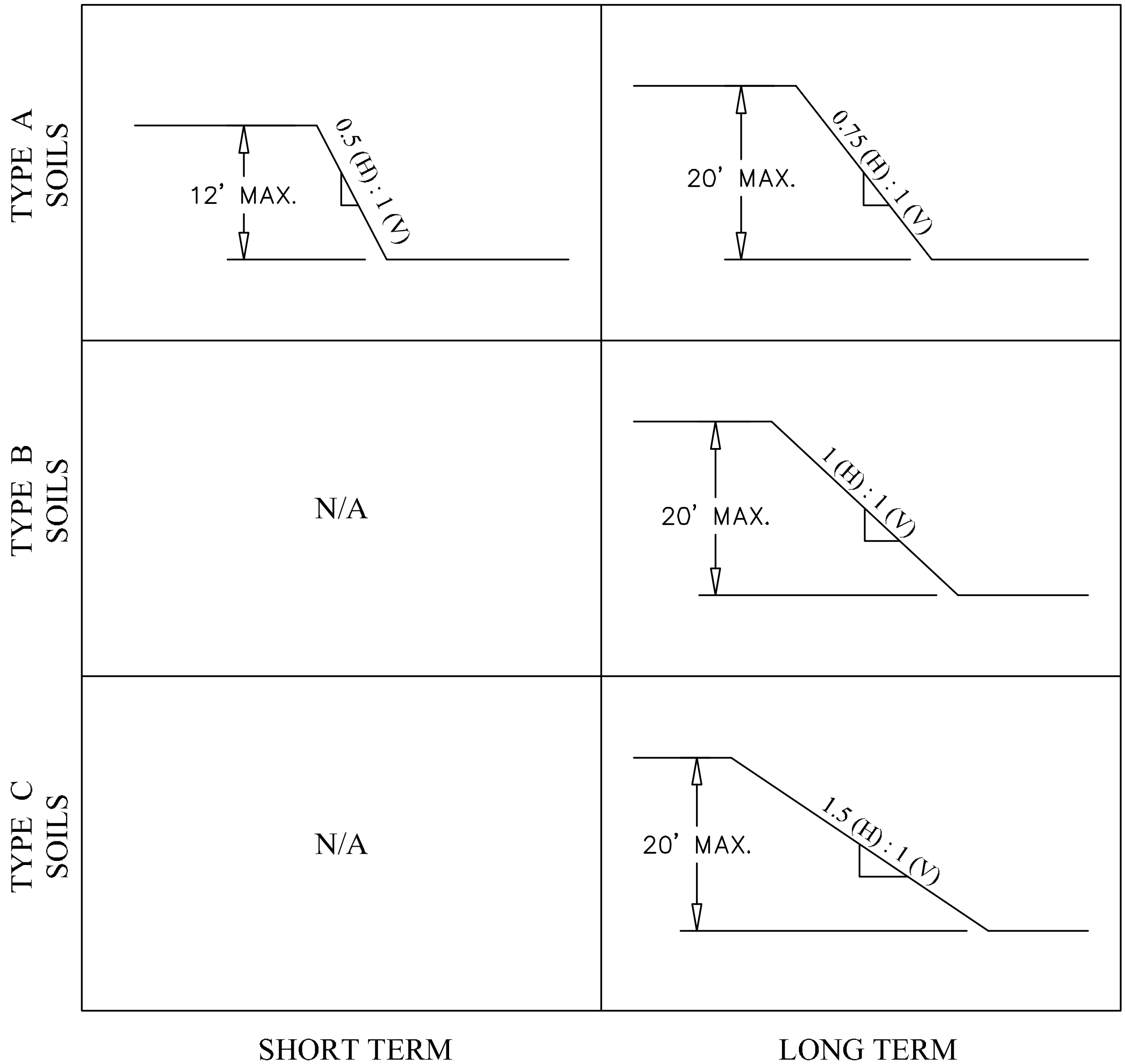
APPENDIX B

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



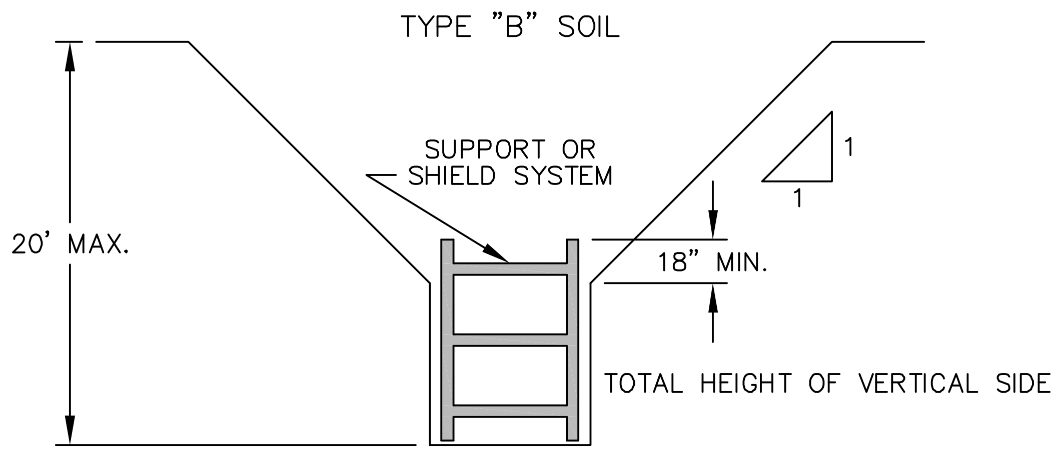
NOTES:

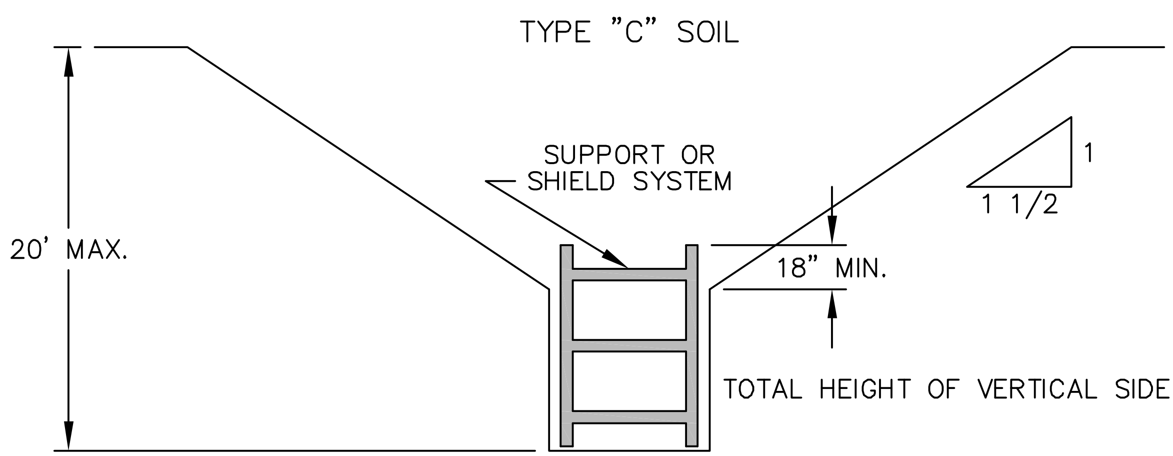
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(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.

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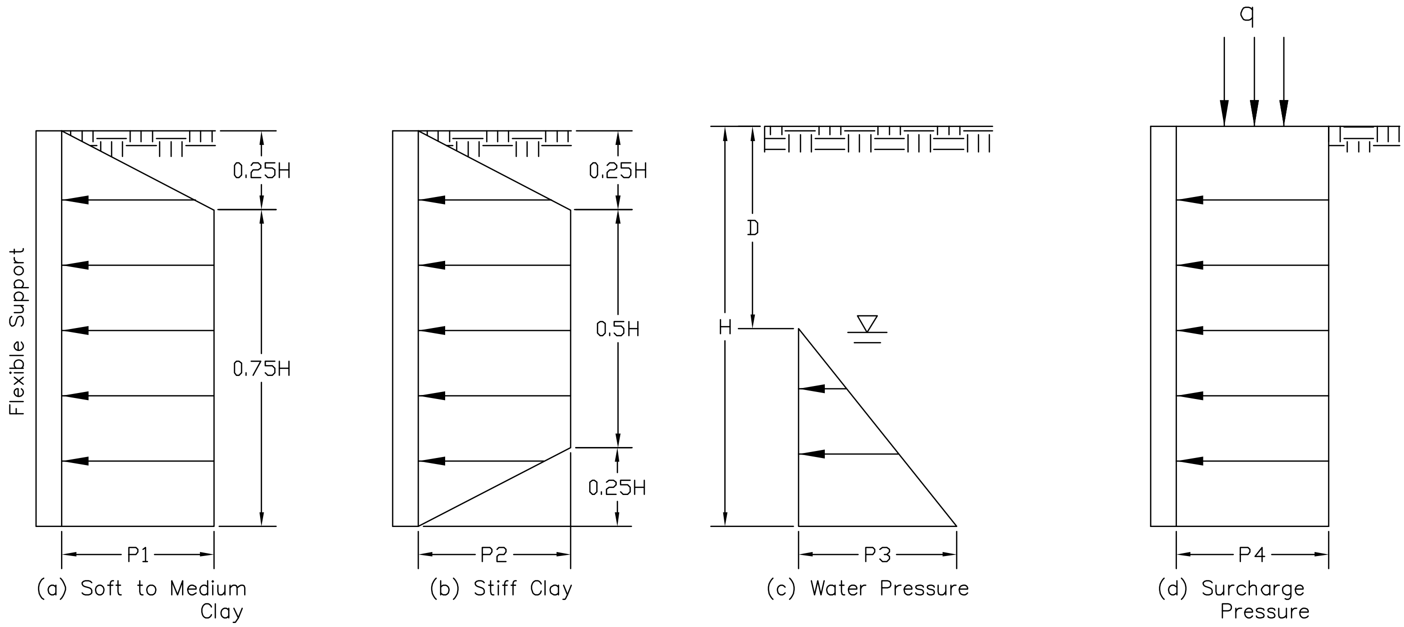
LATERAL PRESSURE DIAGRAMS

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

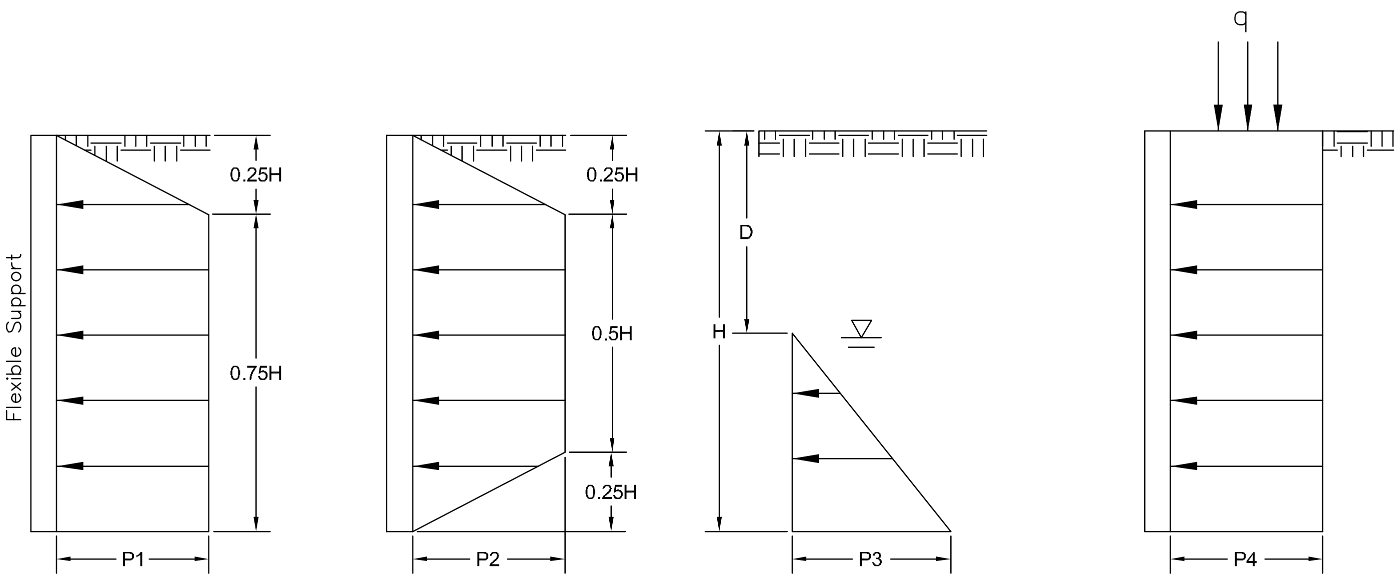
Notes:

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.

ENGINEERING CORP.

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



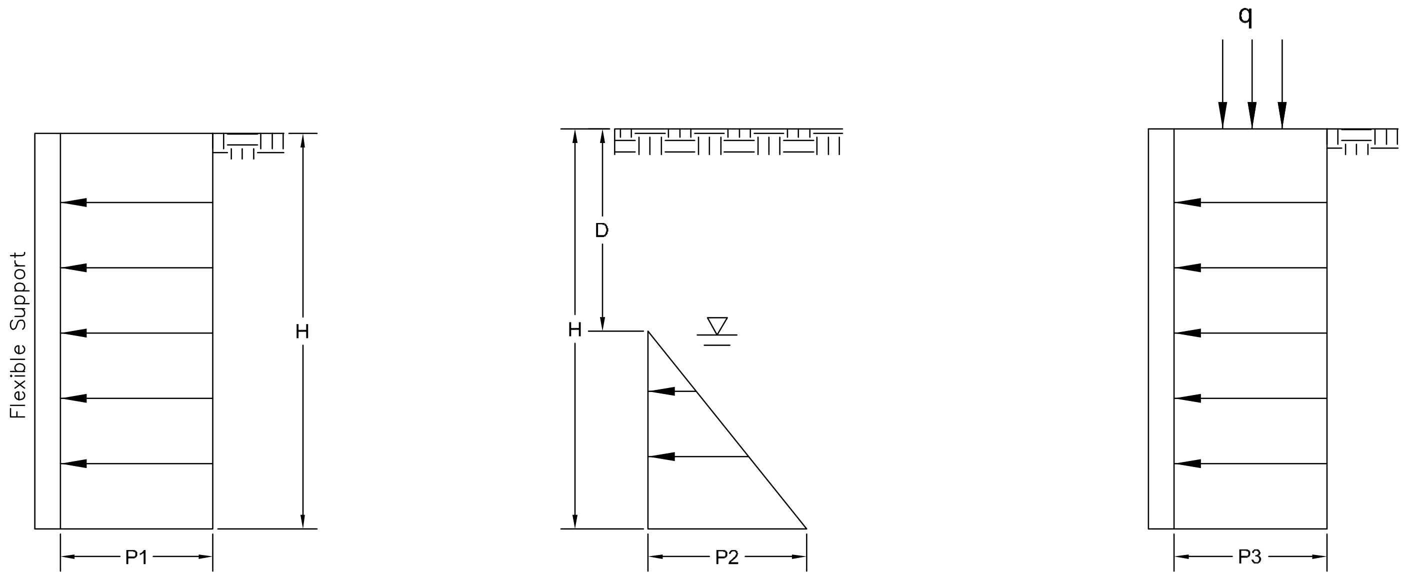
Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H - 4S_u$, 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:

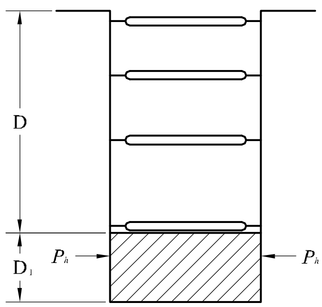
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma H K_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient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

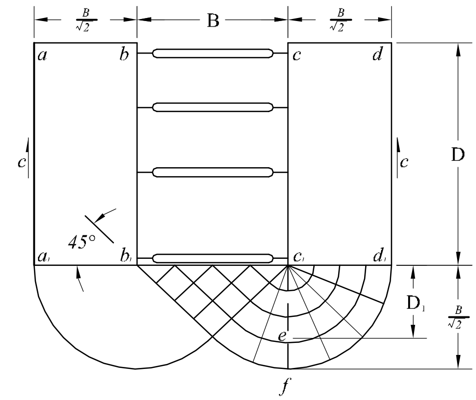
Notes:

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

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

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

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

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

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

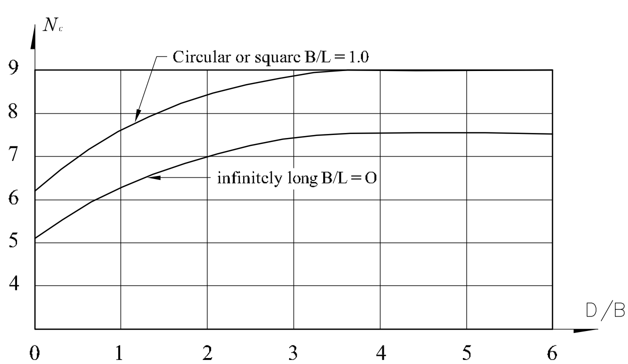
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \; ; D_1 \geqslant 5 \text{ ft.}$$

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).

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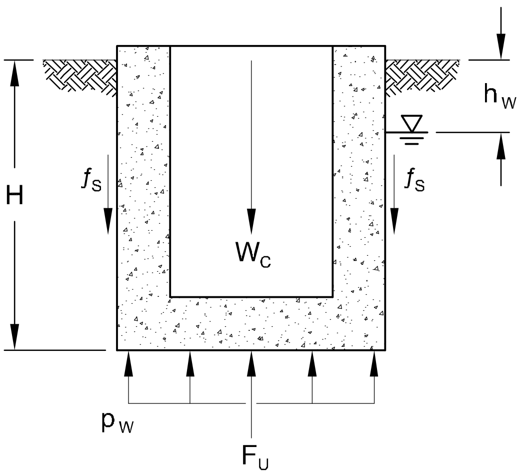
BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

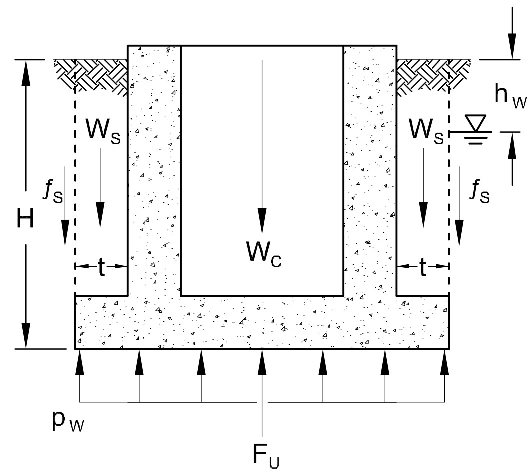


SOIL LAYER 1

SOIL LAYER 2

SOIL LAYER " j "

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)

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

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

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

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

(b)

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

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

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

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

Where:

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

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

Reference:

1) American Concrete Pipe Association, (1996), Manhole Floatation

2) O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA-IF-99-025