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GEOTECHNICAL INVESTIGATION

PROPOSED ALMEDA SIMS
SLUDGE PROCESSING FACILTIY (SPF) IMPROVEMENTS
12319 ½ ALMEDA ROAD, HOUSTON, TEXAS 77045
WBS: R-000298-004-3, FILE NO. WW4903

Reported to

BINKLEY & BARFIELD, INC.

Houston, Texas

by

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REPORT NO. G150-07

October, 2007

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1.0 INTRODUCTION

1.1 Project Description

The study reported herein is a Geotechnical Investigation of the subsurface conditions for the proposed Almeda Sims Sludge Processing Facility (SPF) Improvements. The existing waste water treatment plant is located at 12319 ½ Almeda Road in Houston, Texas (Key Map 572 M). A vicinity map is presented on Plate A-1.

According to the drawings provided to AEC, the proposed new facility include: three sludge holding tanks (each sized 185 feet × 30 feet in plan and about 20-foot high), a new sludge blower building (52 feet × 50 feet), a sludge screening facility, a 100-foot diameter sludge thickener (about 29-foot high: the top of the thickener will be 11 feet above grade and the bottom 18 feet below grade), 14- to 18-inch diameter wastewater drainage lines, as well as access road and site drainage improvements.

1.2 Authorization

The investigation was authorized by Mr. Li C. Chen, P.E., Project Manager with Binkley & Barfield, Inc., on June 11, 2007, upon acceptance of AEC Proposal No. G2007-03-02R5, dated June 6, 2007.

1.3 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil conditions at the project site and develop geotechnical engineering recommendations for foundation support and construction of the proposed facilities. The scope of this geotechnical investigation is summarized as below:

1. Soil drilling for 15 borings to depths of 5 to 65 feet below existing grade;
2. Soil laboratory testing for selected soil samples;
3. Engineering analyses and recommendations of suitable foundations for the proposed sludge holding tanks, the 100-ft diameter sludge thickener and the new sludge blower building;
4. Recommendations for installation of the proposed 14- to 18-inch diameter waster water drainage lines;
4. Pavement section and subgrade preparation recommendations for the proposed access road; and
5. Construction criteria for the structure foundations and pavements.

2.0 SUBSURFACE EXPLORATION

The subsurface conditions at the site were investigated by drilling a total of 15 borings: one 25-foot soil boring (Boring B-1) for the proposed new sludge blower building, four 40-foot soil borings (B-2, B-3, B-5, and B-6) and one 65-foot soil boring (B-4) for the proposed sludge holding tanks, two 50-foot soils borings (B-7 and B-8) for the proposed sludge thickener, three 15-foot soil borings (B-9 through B-11) for the proposed drainage lines, two 5-foot borings (B-13 and B-15) and one 25-foot boring (Boring B-14) for the proposed access road and drainage improvements. The details are shown on the attached Boring Location Plan, Plate A-2. Boring B-7 was converted to a 20-foot piezometer upon completion of drilling; the piezometer installation detail is presented on Plate A-22.

The field drilling was performed using a truck-mounted drilling rig and an all terrain vehicle (buggy) mounted drilling rig. The drilling operations were conducted using dry augering and wet rotary methods. Undisturbed samples of cohesive soils were obtained from the borings by means of 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D-1587 Standard; strength of the cohesive soils was estimated in the field using a hand penetrometer. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586 Standards. Standard Penetration Test resistance values were recorded as "blows-per-foot" and are shown on the Boring Logs. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil. All samples were sealed in plastic bags, placed in core boxes and transported to AEC laboratory.

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 geotechnical technician under supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, minus No. 200 sieve analyses were performed on typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of the cohesive soils were determined by means of triaxial unconsolidated-undrained tests and unconfined compression tests performed on undisturbed samples. The test results are presented on boring logs, Plates A-3 through A-17. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for lab testing are presented on Plates A-18 through A-21. In addition, two consolidation tests were conducted on selected soil samples of Boring B-4 and B-7, the test results are presented on Plates A-23 and A-24.

4.0 SITE CONDITIONS

4.1 Subsurface Conditions

Based on our soil borings, the subsurface soils are variable across the site, and fill was encountered up to 8 feet deep below existing ground surface. Soils encountered in the borings are generally summarized below:

Borings	Depth (ft)	Description of Stratum
B-1	0 - 4	Fill: very stiff, dark gray Fat Clay (CH)
	4 - 25	Stiff to very stiff, tan and gray Fat Clay (CH)
B-2	0 - 2	Fill: dark gray Clayey Sand (SC)
	2 - 4	Fill: very stiff, dark gray Sandy Lean Clay (CL)
	4 - 27	Firm to hard, light gray and tan Fat Clay (CH)
	27 - 32	Medium dense, tan Silty Sand (SM)
	32 - 40	Very stiff, brown and light gray Lean Clay (CL)
B-3	0 - 2	Fill: stabilized gray Sand (SP)
	2 - 8	Fill: stiff, dark gray Fat Clay (CH)
	8 - 33	Firm to hard, gray, tan, reddish brown, and light gray Fat Clay (CH)
	33 - 40	Stiff to very stiff, brown Lean Clay (CL)
B-4	0 - 2	Fill: stiff, dark gray Sand (SP)
	2 - 4	Fill: very stiff, dark gray Fat Clay (CH)
	4 - 27	Frim to very stiff, dark gray, reddish brown, and light gray Fat Clay (CH)
	27 - 32	Medium dense, brown Sandy Silt (ML)
	32 - 47	Medium dense to dense, brown Silt (ML)
	47 - 63	Dense, brown and light gray Silty Sand (SM)
	63 - 65	Stiff, dark gray Fat Clay (CH)
B-5	0 - 2	Fill: stiff, dark gray Fat Clay (CH)
	2 - 22	Firm to hard, light gray and tan Fat Clay (CH)
	22 - 32	Medium dense, tan Silty Sand (SM)
	32 - 37	Medium dense, brown Sandy Silt (ML)
	37 - 40	Hard, brown Lean Clay (CL)
B-6	0 - 2	Fill: stiff, dark gray Fat Clay (CH)
	2 - 26	Stiff to very stiff, dark gray, tan, and light gray Fat Clay (CH)
	26- 40	Medium dense to dense, brown tan Silty Silt (ML)

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B-7	0 - 2	Fill: stiff to very stiff, brown and light gray Lean Clay (CL)
	2 - 4	Fill: firm, dark gray and tan Fat Clay (CH)
	4 - 28	Stiff to very stiff, dark gray, reddish brown, and light gray Fat Clay (CH)
	28 - 33	Brown Silt with Sand (ML)
	33 - 38	Medium dense, brown Silty Sand (SM)
	38 - 43	Stiff, brown Fat Clay (CH)
	43 - 50	Medium dense, brown Silty Sand (SM)
B-8	0 - 8	Fill: firm to hard, brown and light gray Fat Clay (CH)
	8 - 23	Stiff to very stiff, dark gray, reddish brown, and light gray Fat Clay (CH)
	23 -28	Firm to very stiff, reddish brown Lean Clay (CL)
	28 - 33	Brown Sandy Silt (ML)
	33 - 48	Loose to medium dense, brown Silty Sand (SM)
	48 - 50	Stiff to very stiff, gray and tan Lean Clay (CL)
B-9	0 - 2	Fill: dark gray Sand (SP)
	2 - 15	Firm to very stiff, dark gray Fat Clay (CH)
B-10	0 - 2	Fill: brown and light gray Sandy Lean Clay (CL)
	2 - 6	Stiff to very stiff, gray and tan Fat Clay (CH)
	6 - 15	Stiff to very stiff, dark gray Fat Clay (CH)
B-11	0 - 8	Fill: stiff to very stiff, gray and tan Fat Clay (CH)
	6 - 15	Stiff to very stiff, dark gray Fat Clay (CH)
B-12	0 - 8	Fill: very stiff, gray and tan Fat Clay (CH)
	6 - 25	Firm to very stiff, tan and gray Fat Clay (CH)
B-13	0 - 5	Fill: stiff to very stiff, gray and tan Lean Clay (CL)
B-14	0 - 6	Fill: stiff to very stiff, gray and tan Lean Clay (CL)
	6 - 25	Stiff to very stiff, gray and brown Fat Clay (CH), with slickensides
B-15	0 - 5	Fill: stiff to very stiff, gray Fat Clay (CH)

Details of the soils encountered are presented on the boring logs, Plates A-3 through A-17.

4.2 Ground Water Levels

Ground water was encountered in some borings during drilling and approximately 15 minutes after initial encounter, details are summarized in the following table.

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TABLE 1. SUMMARY OF GROUND WATER LEVELS DURING AND AFTER DRILLING

Boring No.	Boring Depth (ft)	Ground Water Level Encountered during Drilling below Existing Grade (ft)	Ground Water Level Encountered ¼ Hour after Initial Encounter below Existing Grade (ft)	Ground Water Level Measured in Piezometer below Existing Grade (ft)
B-1	25	N/A	N/A	
B-2	40	26	15.8	
B-3	40	N/A	36	
B-4	65	30	15.3	
B-5	40	24	15.5	
B-6	40	26	N/A	
B-7	50	26	21	dry on 7/03/07 19.7 on 8/8/07
B-8	50	26	21.5	
B-9	15	N/A	N/A	
B-10	15	N/A	N/A	
B-11	15	N/A	N/A	
B-12	25	N/A	N/A	
B-13	5	N/A	N/A	
B-14	25	N/A	N/A	
B-15	5	N/A	N/A	

5.0 SUBSURFACE VARIATIONS

The information contained in this report summarizes the conditions encountered on the date the borings were drilled; due to past constructions, variation of the subsurface soil (especially fill thickness and characteristics) should be expected. It should be noted that the 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.

Cohesive soils in the Houston area typically have secondary features such as fissures and weak failure planes denoted as "slickensides', and contain sand/silt seams/layers/lenses/pockets. Information on the boring logs is based on 3-inch diameter soil samples which were obtained at two-foot intervals in the top 10 feet and at five-foot intervals thereafter. A detailed description of the soil secondary features may not have been obtained due to the sampling interval and small sample size. Therefore, while some AEC's logs show the soil secondary features, it should not be assumed that the secondary features are absent where not shown on the logs.

6.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

The foundation design must satisfy two independent foundation stability requirements simultaneously: 1) there should be adequate safety against a shear failure within the soil mass, i.e., the foundation bearing pressure should not exceed the allowable bearing capacity of the soil being built upon, and 2) the probable maximum and differential settlements of the soil, or various parts of the foundations must be limited to a safe and tolerable magnitude. In addition, when a structure is subjected to lateral loads and/or uplift loads, the foundations should have adequate sliding and overturning resistance, as well as adequate uplift resistance.

The allowable bearing pressure of the foundation soil stratum depends on the size, shape, depth of the foundations, shear strength parameters of bearing soils, as well as the loading condition. Sustained loads include structural dead load and sustained live load. Total loads include structural dead load, sustained live load, and transient (or temporary) live load. The allowable bearing pressures are obtained by dividing the ultimate bearing pressures with factors of safety of 3 and 2 for sustained loads and total loads, respectively.

6.1 Foundation of Proposed Sludge Blower Building

We understand that a 52 feet by 50 feet sludge blower building will be constructed. Soil boring B-1 was drilled at the proposed building area. The structural loads of the proposed building were not available during preparation of this report; we assume it is lightly loaded, and the finished floor elevation will be close to or slightly higher than the existing ground surface.

6.1.1 Drilled-and-Underreamed Footing

Based on soil subsurface data revealed by soil boring B-1, the top subsurface consist of fill fat clay to about 4 feet below existing grade, underlain by fat clay to the boring termination depth of 25 feet. The top 10 feet of subsurface soils are highly plastic with a liquid limit (LL) between 74 to 76, and PI between 51 and 54. The high plasticity fat clay soils can shrink or swell significantly with changes in their moisture contents, which can result in cracking of structures supported on them. To provide a more uniform foundation support and place the proposed foundation below the zone of seasonal moisture variation (typically 10 feet in Houston area), we recommend the proposed building be supported on drilled-and-underreamed footings, extending to a depth of at least 10 feet below final grade into the stiff fat clay.

Allowable Bearing Capacity Using a minimum safety factor of 3 for sustained dead loads and 2 for total loads, an allowable net bearing capacity of 3,000 pounds per square foot (psf) for dead loads and 4,500 psf for total loads, whichever is critical should be used.

The existing calcareous nodules and ferrous stains within the clay 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 2:1. If significant sloughing or caving occurs for drilled-and-underreamed shafts, further footing excavation should be stopped and straight-sided drilled shafts can be used.

To withstand uplift forces resulting from the expansive soils, each footing should contain reinforcing steel throughout its full length to sustain an uplift load of at least 25d kips, where "d" is the diameter of the shaft in feet. The uplift load with replacement of various thickness of select fill is presented in following table.

TABLE 2. ESTIMATED UPLIFT LOAD vs. THICKNESS OF SELECT FILL REPLACEMENT (B-1)

THICKNESS OF SELECT FILL REPLACEMENT BELOW EXISTING GRADE (FEET)	ESTIMATED UPLIFT LOAD (KIPS)
0	25d
1	23d
2	21d
3	18d
4	16d

Uplift Resistance The proposed structure may be subject to uplift loads due to wind. Using O'Neill and Reese method (Reference 1), the uplift resistance of a drilled-and-underreamed footing in cohesive soils can be calculated as:

$$Q\text{ (uplift)} = (S_u\, N_u\, A_u)/FS_1 + W_c/FS_2 + W_s/FS_3 \qquad (1)$$

$$N_u = 1.5\, D_b / B_b \;\leq 9 \qquad (2)$$

Where: Q = Allowable uplift resistance of a drilled-and-underreamed footing, lbs

S_u = Average undrained shear strength of the cohesive soil between the base of the bell and 2 B_b above the base; use S_u = 700 psf based on Boring B-1

N_u = Bearing capacity factor for base uplift resistance, $N_u \leq 9$

D_b = Depth of the base below the seasonal moisture change zone, ft; the seasonal moisture change zone in Houston can be considered as 10 feet below the existing ground surface

B_b = Diameter of the bell, ft

B_s = Diameter of the footing, ft

W_c = Effective weight of concrete footing, lbs

W_s = Effective weight of projected soil above the bell, lbs (use unit weight, γ = 120 pcf for soils above water table, and buoyant unit weight γ' = 60 pcf for soils below water table)

A_u = $\frac{\pi}{4}(B_b{}^2 - B_s{}^2)$, projected (annular) area of bell over footing, ft^2

FS_1, FS_2, FS_3 = Factors of Safety for uplift resistance

We recommend that the following safety factors be used for uplift design: safety factor of 2 for soil cohesion (FS_1 = 2), safety factor of 1.1 for dead weight of footing (FS_2 = 1.1), and safety factor of 1.5 for weight of soil above the bell (FS_3= 1.5). Based on above equations, the uplift resistance of a drilled-and-underreamed footing can be increased by extending the founding depth of the footing or increasing the diameter of the bell.

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; this minimum spacing should also apply to the spacing between new footings and existing footings.

Footing Settlements A consolidation test and detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, and the anticipated structural loads, we estimate that drilled-and-underreamed footings designed and constructed as recommended will experience total settlements on the order of 1 inch.

Drilled Footing Construction Drilled footing foundations should be constructed in accordance with Section 02465 of the 2002 City of Houston Standard Construction Specifications (COHSCS). Qualified geotechnical personnel should check each footing excavation prior to placing concrete to see 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 columns are concentric with the drilled footings; and
3) Excessive cuttings, any soft or compressible materials, and ponded water are removed from the bottom of the excavation for dry construction.

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. According to short-term ground water observation in soil borings during drilling, the drilled footing excavation may not encounter ground water at the proposed depth of 10 feet. However, ground water level will fluctuate with seasonal rainfall and other climatic events, and may be higher during construction. If groundwater is encountered within the cohesive soils during construction, sump pumps may be used to pump water out from the excavations. 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 pumping of fresh concrete from the recently filled footing to an adjacent unfilled footing.

Since the subsurface soils could change from one location to another, we recommend that subsurface soils at each drilled footing location be inspected by qualified geotechnical personnel to verify that soil type and strength is representative of those presented in this report. Hand penetrometers along with sound engineering judgment should be used during drilled-and-underreamed footing construction to obtain relative indication of soil shear strength from intact soil cuttings in the underream zone. Based on our soil boring B-1, we recommend a minimum penetrometer reading of 2.0 tons per square foot (tsf) for intact cohesive soil cuttings at the foundation depth. Penetrometers readings in granular soils and clayey soils that are easily disturbed (very sandy/silty clays, lean clay soils with significant amount of sand/silt/calcareous and ferrous nodules, lenses, seams) generally do not provide meaningful data.

6.1.2 Floor Slab Support

In the greater Houston area, 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 rainfall and evaporation rates. The moisture active zone extends to about 10 feet below ground in the greater Houston area.

<u>Estimated Soil Movements</u> The Potential Vertical Rise (PVR) is an estimate of the potential of an expansive soil to swell from its initial physical state, i.e., its moisture content, density and confining pressure. It is an estimate of the vertical movement that could occur if the high plasticity clay is allowed to absorb water. For the top 10 feet of the existing soils encountered, the PVR is estimated to be about 4.9 inches at Boring B-1, using the

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TxDOT method Tex-124-E and without considering structural surcharge. The relationship between estimated PVR and thickness of select fill replacement below existing grade is presented in the following table.

TABLE 3. ESTIMATED PVR vs. THICKNESS OF SELECT FILL REPLACEMENT (B-1)

THICKNESS OF SELECT FILL REPLACEMENT BELOW EXISTING GRADE (FEET)	ESTIMATED PVR (INCH)
0	4.9
1	4.4
2	3.9
3	3.2
4	2.5
5	2.0
6	1.5
7	1.1
8	0.7

Additional movements can occur in areas if water ponding is allowed to occur during or after construction and where soils with a higher PI are encountered, or if the high plastic soils are allowed to dry, prior to fill or concrete placement. On the other hand, the high plasticity clay may also experience shrinkage during periods of dry weather as moisture evaporation occurs at the ground surface and the ground water table drops. The actual PVR of the site will be highly dependent upon the moisture regime of the 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 of the floor slab.

<u>Floor Slab</u> In general, the tolerable PVR for a common building slab is 1 inch. To limit PVR within 1 inch, the following approaches can be used: (1) a structural floor slab with a minimum 10 inches of gap between the bottom of the slab and the top of subgrade soils would be effective in mitigating problems associated with movements of high plasticity clays below a slab-on-grade; or (2) replacing 8 feet of existing soils with compacted select fill, as based on Table 2 above. Structural floor slabs are the most effective in mitigating the potential shrink-swell movements of high plasticity clays.

Because structural floor slabs or deep soil replacement are expensive, reinforced slab-on-grade floors are commonly used in conjunction with subgrade preparation and limited thickness of select fill replacement to mitigate some of the risk. If this risk is acceptable, slab-on-grade floors may be used. Recommendations for subgrade preparation and measures to reduce moisture fluctuation of the subgrade soils are presented below.

Subgrade Preparation Based upon Table 3, the excavation depth required to achieve a PVR of 1 inch range is up to 8 feet. If desired, limited excavation can be performed in conjunction with subgrade preparation and measures to reduce moisture fluctuations of the subgrade soils, to reduce construction costs.

Subgrade preparation should extend a minimum of 5 feet beyond the floor slab perimeter. Existing building vegetation, roots, and other deleterious materials should be removed in advance with applicable portions of COHSCS Section 02233. The existing soils should be excavated to a depth of at least 4 feet below existing grade. The exposed soils should be proof-rolled in accordance with the TxDOT Standard Specifications for Construction of Highways, Streets, and Bridges, Item 216 (2004 Edition) to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with select fill. The clayey soils exposed after proof-rolling should be scarified to a minimum depth of 6 inches and stabilized with at least 7 percent hydrated lime (measured relative to dry soil weight). The actual percent lime required should be determined by laboratory testing prior to construction. The stabilized soils should be compacted to at least 95 percent of the ASTM D-698 (Standard Proctor) maximum dry density at a moisture content between optimum and 3 percent wet of optimum. Compacted select fill should then be used to achieve bottom elevation of the floor slab. We also recommend that a minimum 10 mil polyethylene (horizontal moisture barrier) be placed below the concrete slab to move edge effects away from the slab and reduce seasonal fluctuations of water content directly below the structure.

It should be noted that the PVR after 4 feet select fill replacement below the existing grade and 6 inches of lime-stabilized subgrade is higher than 1 inch (see Table 3).

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 upon 8-inch carton forms. When carton forms are being placed, 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 the carton form 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 and design for swelling pressure of 2,500 psf. The drilled shafts and beams should be tied together.

6.2 Foundations of Proposed Sludge Holding Tanks and the Sludge Thickener

We understand that three sludge holding tanks (each sized 185 feet × 30 feet in plan and about 20-foot high above existing grade, about 1 to 10.5 feet below existing grade) will be installed; two tanks will be built in the Phase I, and the third tank will be built in the Phase II. Borings B-2 through B-6 was drilled at the proposed tank site.

We also understand that the new 100-foot diameter Sludge Thickener will be located about 11 feet above grade, and the foundation will be located about 6 to 18 feet below the grade. Borings B-7 and B-8 were drilled at the proposed Sludge Thickener site, and Boring B-7 was converted into a 20-foot deep piezometer after completion of drilling. According to the drawing provided to AEC, the project site was occupied by two 36-foot diameter fuel oil storage tanks, which were removed around 1999. Currently, an existing sludge thickener is located to the west of the proposed sludge thickener.

6.2.1 Foundation Type and Depth

6.2.1.1 Foundation for Proposed Sludge Holding Tanks

We understand that the mat foundations will be used for proposed three Sludge Holding Tanks, the mat foundations will be located about 1 to 10.5 feet below the existing grade. Based on the subsurface soils encountered at Borings B-2 through B-6, and using a safety factor of 3 for sustained loads and 2 for total loads, a net allowable bearing capacity of 2,000 pounds per square foot (psf) for sustained loads and 3,000 psf for total loads, whichever is critical should be used for the proposed foundation design. We recommend that the underground tank be designed for waterproofing. The existing fill material should be replaced with compacted select fill or lime-stabilized on site clean clay. We recommend that a minimum 3 feet of subsurface soils below the mat foundations be stabilized with at least 7% hydrated lime (by dry weight of soil) in accordance with 2002 COHSCS, Section 02336. This is a preliminary estimate, the actual percentage of lime should be confirmed by laboratory test prior to construction. We also recommend that a minimum 10 mil polyethylene (horizontal moisture barrier) be placed below the concrete slab to move edge effects away from the slab and reduce seasonal fluctuations of water content directly below the structure.

6.2.1.2 Foundation for 100-foot diameter Sludge Thickener

We understand that the mat foundation will be used for the proposed 100-foot Sludge Thickener, the mat foundation will be located about 6 to 18 feet below the existing grade. Based on the subsurface soils encountered at Borings B-7 and B-8, and using a safety factor of 3 for sustained loads and 2 for total loads, a net allowable bearing capacity of 2,000 pounds per square foot (psf) for sustained loads and 3,000 psf for total loads, whichever is critical should be used for the proposed foundations. The existing fill material should be replaced with compacted select fill or lime-stabilized on site clean clay. We recommend that the underground thickener be designed for waterproofing. We recommend that a minimum 3 feet of subsurface soils below the mat foundations be stabilized with at least 7% hydrated lime (by dry weight of soil) in accordance with 2002 COHSCS, Section 02336. This is a preliminary estimate; the actual percentage of lime should be confirmed by laboratory test prior to construction. We also recommend that a minimum 10 mil polyethylene (horizontal moisture barrier) be placed below the concrete slab to move edge effects away from the slab and reduce seasonal fluctuations of water content directly below the structure.

6.2.2 Foundation Settlement Analysis

6.2.2.1 Settlement Analysis for Proposed Sludge Holding Tanks

A limited settlement analysis was performed using the results of the consolidation test. Using a footing pressure of 1,600 psf according to the drawing provided to AEC, the consolidation settlements at the center of the 185-foot long, 60-foot wide mat, the center at short edge, and the corner are estimated to be approximately 3.2, 2.1 and 2.3 inches respectively. We recommend that settlements be monitored during whole construction period to compare with the estimated values and evaluate further action if the measured settlements are within tolerance.

6.2.2.2 Settlement Analysis for Proposed 100-foot diameter Sludge Thickener

A limited settlement analysis was performed using the results of the consolidation test. We estimated a total consolidation settlement on the order of 2.0 inches for virgin soils under the center of the proposed tank foundation, based on the 50-foot deep Soil Borings B-7 and B-8.

Considering the potential of differential settlements, we recommend the use of flexible connections between pipelines and the treatment modules.

6.2.3 Modulus of Subgrade Soil Reaction

Based on soils encountered in Borings B-2 through B-8, and the proposed foundation depths of 1 to 18 feet below the existing grade, a value of 60 pci (pounds per cubic inch) can be used for the modulus of natural subgrade soil reaction for the mat footing design.

6.2.4 Lateral Earth Pressures

According to drawings provided to AEC, we understand that the walls of the proposed sludge holding tanks will extend from about 20 feet above the finished grade to about 1 to 10.5 feet below the finished grade. We also understand that the wall of the proposed sludge thickener will extend from 11 feet above the finished grade and to about 18 feet below the finished grade.

Lateral Earth Pressures The magnitudes of the lateral earth pressures on the proposed walls, will depend on the type and density of the backfill, surcharge on the finished grade 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. Typical backfill materials placed behind walls in the Houston area include select fill and cement-stabilized sand. Lateral pressure resulting from construction equipment, pavement and traffic, or other surcharge on the top of the walls should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure (we recommend that at least 240 psf surcharge be used for design of the walls). Hydrostatic pressure should also be included, unless adequate drainage is to be provided behind the walls.

If no movements are allowed for the proposed walls, the walls should be designed based on at-rest earth pressure. The at-rest earth pressure at depth z can be determined by Equation (3) as follows:

$$p_0 = (q_s+\gamma h_1+\gamma' h_2)K_0 + \gamma_w h_2 \qquad (3)$$

where,
- p_0 = at-rest earth pressure, psf.
- q_s = uniform surcharge pressure, psf.
- γ, γ' = wet and buoyant unit weights of soil, pcf, see Table 3 and Table 4 below.
- h_1 = depth from ground surface to ground water table, feet.
- h_2 = z-h_1, depth from ground water table to point under consideration, feet.
- z = depth below ground surface, feet.

K_0 = coefficient of at-rest earth pressure, see Table 4 and Table 5.
γ_w = unit weight of water, 62.4 pcf.

If the proposed walls are allowed to move away from the soil, the walls can be designed based on active earth pressure. The active earth pressure at depth z can be determined by Equation (4) as follows:

$$p_a = (q_s + rh_1 + r' h_2) K_a - 2c\sqrt{K_a} + r_w h_2 \qquad (4)$$

Where, p_a = active earth pressure, psf.
K_a = coefficient of active earth pressure, see Table 4 and Table 5.
c = cohesion of clayey soils, see Table 4 and Table 5.
q_s, γ_w, γ, γ', h_1, h_2, z are described in Equation (3).

TABLE 4. Design Soil Parameters for Walls of Sludge Holding Tanks (B-2 through B-6)

Depth (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
0 - 2	Fill: SM/SC	115	55	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
2 - 8	Fill: CH/CL	120	60	800	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
8 - 26	Stiff CH/CL	120	60	1000	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
26 - 40	SM/ML	115	55	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
NA	Select Fill	120	60	1600	0	1.00	1.00	1.00	200	22	0.45	0.63	2.20

TABLE 5. Design Soil Parameters for Wall of Sludge Thickener (B-7 and B-8)

Depth (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
0 - 8	Fill: CH/CL	120	60	800	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
8 - 28	Stiff CH/CL	120	65	1000	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
28 - 50	ML/SM	115	55	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
NA	Select Fill	120	60	1600	0	1.00	1.00	1.00	200	22	0.45	0.63	2.20

Notes: (1) The depth shown in above table is from existing ground surface; CH = Fat Clay, CL = Lean Clay, ML = Silt, SM= Silty Sand (SM).
(2) γ = unit weight for soil above water level, γ' = buoyant unit weight for soil below water level.
(3) C = ultimate cohesion, φ = ultimate angle of internal friction.
(4) 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.
(5) AEC recommends the use of FS = 2 for passive earth pressure if it is to be used in the design.

Backfill Material We recommend use of select fill or cement-stabilized sand as backfill behind the proposed walls. Select fill criteria are presented in Section 6.7 of this report; cement-stabilized sand should be in accordance with the 2002 City of Houston Standard Construction Specifications (COHSCS), Section 02321. The backfill should be placed in lifts not exceeding 8 inches in loose thickness (or 4 inches if hand-tamping or light compaction equipment is used). The select fill should be compacted to at least 95 percent of maximum dry density determined by ASTM D-698, at a moisture content between optimum and 3% wet of optimum; the cement-stabilized sand should be compacted to at least 95 percent of maximum dry density determined by ASTM D-558, at a moisture content between ± 3% of optimum.

6.2.5. Hydrostatic Uplift Resistance

The proposed sludge holding tanks and sludge thickener should be designed to resist hydrostatic uplift. For uplift design of the underground structures, we recommend that the water level be assumed to be at the ground surface or 100-year flood elevation, whichever is more critical. If the dead weights of the structures are inadequate to resist uplift forces, toe extensions of the base slabs may be constructed so that the effective weight of the soil above the extended slabs can be utilized to resist the uplift forces. The buoyant unit weight of concrete can be taken as 90 pcf. The minimum recommended factors of safety against uplift should be 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. Design soil parameters are included on Tables 3 and 4. Recommended design criteria for uplift resistance are shown on Plate B-9.

6.2.6 Foundation Excavation and Support

The Contractor should be responsible for designing, constructing and maintaining safe excavations and protecting existing structures in the vicinity, if any, from adverse effects resulting from construction. Excavation should be in accordance with Occupational Safety and Health Administration (OSHA), Safety and Health Regulations, 29 CFR, Part 1926, Subpart P (Excavation and Trenches) and applicable local regulations.

a) *Trenches Deeper than 20 Feet*: OSHA requires that shoring or bracing for trenches deeper than 20 feet be designed by a licensed professional engineer.
b) *Trenches 20 Feet Deep or Less*: Trench excavations that are 20 feet deep or shallower may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, public and adjacent structures in accordance with OSHA Regulations. For the subsurface soils encountered in Borings B-2 through B-8, the top fill soils, granular soils (SM/ML), and submerged clayey soils (CL/CH) should be classified as Type "C" soils, while other stiff to very stiff cohesive soils (CL/CH) can be classified as Type "B" soils.

Plate B-2 presents the maximum allowable slopes for Soil Type A, B, and C for excavations less than 20 feet deep. If limited space is available for the required side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated in Plate B-3. Other means of support for excavations include trench boxes, closely-spaced drilled shafts, inter-locking steel sheet piles and H-pile soldier beams and timber lagging. Pressure distribution for the design of struts in open cuts for clay and sand are illustrated on Plates B-4 through B-6. If there is water behind the bracing, hydrostatic pressure should be considered in the design. Lateral pressure resulting from construction equipment or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure.

Caution should be taken due to the close spacing between the proposed structure and existing structure. Excavation too close to the existing structure will reduce the overburden pressure, which may be an important component of the bearing capacity for the existing foundation. To reduce the potential of soil bulging from loss of overburden pressure and distressing the existing structure in the vicinity, the excavation face adjacent to the existing structure should be adequately shored, such as steel sheet piles or bracing. We also recommend the new foundations be installed as soon as the excavation is completed.

Foundation excavation should be inspected by a qualified Owner's Representative prior to placing concrete, to check that (a) the footing excavation has been constructed to the specified dimensions at the recommended depth and formation, and (b) excessive cuttings and any soft-compressible materials and ponded water have been removed from the bottom of the excavation. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the state of stress and possible sloughing in the foundation soils. No foundations should be left open overnight or poured without the prior approval of the Owner's Representative.

We recommend that the exposed walls of the foundation excavation 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 and floor slab excavation be covered by a minimum 2-inch thick lean concrete seal slab if the foundations and/or slab will not be poured within 24 hours. Central to this recommendation is the importance of preserving the moisture regime that exists in the fat clay. This moisture condition is essential in minimizing swelling of the very high plasticity fat clay at the project site.

Excavation Bottom Stability In open-cuts within clayey soils, 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 clay when the excavation depth is sufficient 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. Guidelines for evaluating bottom stability in clayey soils are presented on Plate B-7.

If the excavation extends below ground water, and the soils at or near the bottom of the excavation are mainly sand or silt, 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 2 feet below the bottom of the excavation.

6.3 Foundation for the proposed Sludge Screening Facility

We understand that a Sludge Screening Facility will be installed at about 13 feet above grade. Soil boring B-12 was drilled at the proposed facility area. The structural loads of the proposed facility were not available during preparation of this report, and we assume the finished grade elevation will be close to or slightly higher than the existing ground surface.

6.3.1 Drilled-and-Underreamed Footing

Based on soil subsurface data revealed by soil boring B-12, the top subsurface consist of fill fat clay to about 8 feet below existing grade. Below this layer is a stratum of fat clay to the boring termination depth of 25 feet. The top 10 feet of subsurface soils are highly plastic with a liquid limit (LL) between 65 to 82, and PI between 45 and 59. The high plasticity clayey soils can shrink or swell significantly with changes in their moisture contents, which can result in cracking of structures supported on them. To provide a more uniform foundation support and place the proposed foundation below the zone of seasonal moisture variation (typically 10 feet in Houston area), we recommend the proposed sludge screening facility be supported on drilled-and-underreamed footings, extending to a depth of at least 10 feet below final grade into the stiff fat clay.

<u>Allowable Bearing Capacity</u> Using a minimum safety factor of 3 for sustained dead loads and 2 for total loads, an allowable net bearing capacity of 2,500 pounds per square foot (psf) for dead loads and 3,750 psf for total loads, whichever is critical should be used.

The existing calcareous and ferrous nodules within the clay 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

2:1. If significant sloughing or caving occurs for drilled-and-underreamed shafts, further footing excavation should be stopped and straight-sided drilled shafts can be used.

To withstand uplift forces resulting from the expansive soils, each footing should contain reinforcing steel throughout its full length to sustain an uplift load of at least 25d kips, where "d" is the diameter of the shaft in feet.

Uplift Resistance The uplift resistance can refer section 6.1.1.

Lateral Load Analyses The foundation of the proposed pavilion will be subjected to lateral loads from wind. Design lateral loads were not available during preparation of this report. The minimum required footing embedment depth (for lateral load support), under free-head condition and with maximum lateral deflection of 0.5 inch at footing head, can be calculated based on the following equations (International Code Council, 2000, "International Building Code", Section 1805.7.2.1):

$$D_{min} = 0.5A\ [1 + (1+4.36h/A)^{1/2}] \qquad (5)$$

$$A = 2.34V/\ (S_1B) \qquad (6)$$

Where, D_{min} = minimum required depth of shaft embedment in feet but not over 12 feet for purpose of computing lateral pressure

V = applied lateral shear load (wind load), pounds

h = vertical distance from ground surface to point of application of V, feet

B = diameter of drilled footing, feet

z = $D_{min}/3$, depth below the ground surface, feet

S_1 = allowable lateral soil-bearing pressure at z, $S_1 = 100*z$ (per 2000 IBC Table 1804.2), lb/ft^2; an increase of one-third (i.e. $S_1 = 133*z$) is permitted when considering load combinations, including wind or earthquake loads, as permitted by 2000 IBC Section 1605.3.2.

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; this minimum spacing should also apply to the spacing between new footings and existing footings.

Footing Settlements A consolidation test and detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, and the anticipated structural loads, we estimate that drilled-and-underreamed footings designed and constructed as recommended will experience total settlements on the order of 1 inch.

Drilled Footing Construction The recommendations on drilled footing construction are included in section 6.1.1 of this report.

6.3.2 Floor Slab Support

Based on the drawings provided to AEC, we recommend that 8-inch reinforced concrete slab-on-grade floors be used in conjunction with 8-inch lime stabilized subgrade soils. The steel rebars should be grade 60, the concrete compressive strength at 28 days should be 3,000 psi.

Subgrade Preparation We recommend that subgrade preparation include removal of vegetation, roots, as well as debris to a minimum depth of 6 inches below existing ground. The exposed surface should then be proof-rolled in accordance with the 2004 TxDOT Standard Specifications Item 216. The surface should then be checked by qualified geotechnical professionals to identify any weak or dry soils, and deleterious materials. These materials should be removed and replaced with select fill.

To provide a stable subgrade for the slab-on-grade, we recommend that the top 8 inches of exposed soils after proof-rolling be scarified and then stabilized with at least 7 percent hydrated lime (by dry soil weight). The percent lime required for stabilization should be determined by laboratory testing prior to construction. Stabilized soils should be placed in no more than 8-inch loose lifts and compacted to at least 95 percent of the ASTM D-698 (Standard Proctor) maximum dry density at a moisture content between optimum and 3% wet of optimum. Lime stabilization should be done in accordance with Section 02336 of the 2002 COHSCS.

6.4 Installation of Wastewater Drainage Lines

We understand that the 14-inch to 18- inch diameter wastewater drainage lines and four manholes will be installed, and the maximum invert depth of storm sewer will be approximately less than 10 feet. The drainage lines will most likely be installed using open-cut methods. Soil borings B-9, B-10, and B-11 were drilled to 15 feet deep below existing grade along the proposed alignment of wastewater drainage lines. According to soil conditions revealed by these three soil borings, the subsurface soils along the alignment of proposed waste water drainage lines generally consist of 2 to 8 feet of fill soils (silty sand, fat clay and lean clay), underlain by firm to very stiff fat clay to boring termination depth of 15 feet. The clay soils generally consist of very high plasticity clay soils, with liquid limits (LL) ranging from 71 to 82, and plasticity indices (PI) ranging from about 49 to 60. Calcareous nodules, ferrous stains, and siltstone fragments were encountered in the clayey soils.

6.4.1 Geotechnical Parameters

A summary of the recommended geotechnical parameters is presented in the following table. These values are based on the results of field and laboratory test data as well as our experience. It should be noted that because of the nature of the soil stratigraphy, parameters at locations away from the borings may vary substantially from values reported in the following table.

TABLE 6. DESIGN SOIL PARAMETERS FOR TRENCH EXCAVATION (B-9, B-10, B-11)

Depth (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
0 - 2	Fill: soft CL	115	55	300	0	1.00	1.00	1.00	0	16	0.57	0.72	1.76
2 - 8	Fill: stiff CH	120	60	800	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
8- 15	Stiff CH	125	65	1000	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
N/A	Select Fill	120	60	1600	0	1.00	1.00	1.00	200	22	0.45	0.63	2.20

Notes: (1) The depth shown in above table is from existing ground surface; CH = Fat Clay, CL = Lean Clay
(2) γ = unit weight for soil above water level, γ' = buoyant unit weight for soil below water level.
(3) C = ultimate cohesion, φ = ultimate angle of internal friction.
(4) K_a = coefficient of active earth pressure, K_0 = coefficient of at-rest earth pressure, K_p = coefficient of passive earth pressure, for level backfill.

6.4.2 Trench Stability

The Contractor should be responsible for designing, constructing and maintaining safe excavations. Trench excavations may be shored (such as trench boxes or timber lagging), sheeted and braced, laid back to a stable slope, or other appropriate means used for the safety of workers, public and adjacent roadway and structures. The excavation and trenching should be in accordance with Occupational Safety and Health Administrations (OSHA), Safety and Health Regulations, 29 CFR 1926, Subpart P. OSHA soil types are presented on the following table.

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TABLE 7. OSHA SOIL CLASSIFICATION FOR TRENCH EXCAVATION SUPPORT (B-9, B-10, B-11)

BORING NO.	0'-2'	2'-4'	4'-6'	6'-8'	8'-10'	13'-15'
B-9	C	C	B	B	B	B
B-10	C	C	C	B	B	B
B-11	C	C	C	C	B	B

Notes: (1) Soil Types
A q_u = 1.5 tsf or greater,
B q_u = 0.5 tsf or greater,
C q_u = less than 0.5 tsf or granular soils, submerged soils or soils with significant weak secondary structure.
(2) q_u = Unconfined Compression Strength

In cohesive soils, Critical Height is defined as the height that a slope will stand unsupported for a short time; it is used to determine maximum depth of open cuts at given side slopes. Critical Height may be calculated based on the cohesion of the soil. Critical Heights for various slopes and cohesion values are illustrated on Plate B-1, Appendix B. Several cautions should be exercised in the 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 other unsuitable soils 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 cracks 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, for example, where granular soils will not stand on stable slopes and/or for deep open cuts.

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

Plate B-2 in Appendix B presents the maximum allowable slopes in Soil Types A, B, and C for excavations less than 20 feet. If limited space is available for the required open trench side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated in Plate B-3, Appendix B. A discussion of the calculation of bracing pressure is presented in subsequent paragraphs. Guidelines for bracing schemes and methods of calculating bracing stress are presented below.

Computation of Bracing Pressures The following method can be used for calculating earth pressure against bracing for open cuts. Lateral pressure resulting from construction equipment 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 the following relationship:

$$p_a = (q_s + rh_1 + rh_2)K_a - 2c\sqrt{k_a} + r_w h_2 \qquad (7)$$

Where, p_a = active earth pressure, psf.
q_s = uniform surcharge pressure, psf.
γ, γ' = wet and buoyant unit weights of soil, as in Table 6.
h_1 = depth from ground surface to ground water table.
h_2 = z-h_1, depth from ground water table to the point under consideration.
z = depth below ground surface.
γ_w = unit weight of water, 62.4 pcf.
K_a = coefficient of active earth pressure, see Table 6.

Pressure distribution for the design of struts in open cuts for clay and sand are illustrated in Plates B-4 through B-6, Appendix B. If there is water behind the bracing, hydrostatic pressure should be considered in the design.

Bottom Stability In open-cuts, the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil, must be considered. In clays and sandy lean 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. This can be mitigated if a well point system is used to dewater the area. Guidelines for evaluating bottom stability are presented on Plate B-7 in Appendix B.

If the excavation is carried out below the groundwater table and a significant amount of the soils at or near the bottom of the excavation are sands or silts or low plasticity clays, the bottom can fail by blow-out (boiling) at the bottom when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized.

To avoid bottom boiling of braced excavation terminating in granular soils below groundwater, the groundwater table should be lowered to at least 3 feet below the excavation. In extreme conditions, mechanical or chemical stabilization of the granular soils may be required.

If the braced excavation terminates in a cohesive soil, but is underlain by granular soils and is subject to hydraulic pressure, the factor of safety against bottom failure can be conservatively calculated (neglecting shear strength) from the equation presented below (Reference 6):

$$F = \gamma_S h_s / \gamma_w h_w \qquad (8)$$

Where F = safety factor against blow-out, minimum 1.25

γ_s = unit weight of cohesive soil above sand layer, pcf

h_s = height of cohesive soil above sand layer, feet

γ_w = unit weight of water, pcf

h_w = hydrostatic head, feet

The most effective means of improving the bottom stability is to lower the groundwater table to at least 3 feet (or more as determined by analysis) below the bottom of the braced excavation.

<u>Earth Pressure on Pipes</u> The buried pipes will be subjected to overburden pressure, and pressure due to traffic loads when they cross the roadway. The loads resulting from the overburden and traffic loads are presented on Plate B-8, Appendix B.

6.4.3 Bedding and Backfill

Excavation and backfill for utilities should be in accordance with Sections 02317 and 02320 of the 2002 COHSCS. In general, bedding and backfill for the storm sewers should be in accordance with the 2002 City of Houston Standard Construction Detail Drawings, Drawing No. 02317- 02 and 02317- 03, as applicable.

Granular soils other than silt may be used as backfill. Cohesive soils with a LL of less than 45 and a PI in the range of 8 to 20 can also be used as backfill material. Backfill should be placed in lifts not exceeding 8 inches in loose thickness and compacted to at least 95% of ASTM D 698 maximum dry density at a moisture content between optimum and 3 percent wet of optimum.

6.4.4 Manholes

This section addresses geotechnical recommendations relating to design of the manholes. In general, we anticipate the soils up to 15 feet along the proposed drainage line alignment will generally be firm to very stiff lean clay/fat clay.

Allowable Bearing Capacity The net allowable bearing capacities for manholes at depths from 6 to 10 feet below existing pavement are presented in following table. Safety factors of 3 for dead loads and 2 for total loads are incorporated in the allowable bearing capacity; whichever is critical should be used.

TABLE 8. NET ALLOWABLE BEARING CAPACITY FOR MANHOLES

LOCATION	DEPTH BELOW EXISTING GRADE (ft)	NET ALLOWABLE BEARING CAPACITIES (psf)	
		DEAD LOAD ONLY	TOTAL LOAD
General	6-10	1,650	2,500

Uplift The manholes should be designed to resist hydrostatic uplift. For uplift design of the underground structures, we recommend that the water level be assumed to be at the ground surface or 100-year flood elevation, whichever is more critical. If the dead weights of the structures are inadequate to resist uplift forces, toe extensions of the base slabs may be constructed so that the effective weight of the soil above the extended slabs 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 are included in Table 6. Recommended design criteria for uplift resistance are shown on Plate B-9, Appendix B.

Lateral Earth Pressures No movement will be allowed for the walls of the manholes. Therefore, the walls should be designed for at-rest earth pressure. The magnitudes of these pressures 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. Typical backfill materials placed behind walls in the Houston area include select fill and cement-stabilized sand.

Lateral pressure resulting from construction equipment or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure should also be included, unless adequate drainage is provided behind the walls. The at-rest earth pressure at depth z can be determined by the Equation (3) presented in Section 6.2.4.

<u>Backfill Material</u> We recommend use of select fill or cement-stabilized sand as backfill for the manholes. The select fill criteria are presented in Section 6.7 of this report. The select fill or cement-stabilized sand should be placed in lifts not exceeding 8 inches in loose thickness (or 4 inches if hand-tamping or light compaction equipment is used) and compacted to requirements specified in Section 3.08 of COHSCS Section 02316.

6.4.5 Thrust Restraint Design Recommendations

Thrust forces are generated in pressure pipes as a result of changes in pipe diameter, pipe direction or at the termination point of the pipes. The pipes could disengage at the joints if the forces are not balanced or if the pipe restraint is not adequate. Various methods of thrust restraint are used including thrust blocks, restrained joints, encasement and tie-rods.

Thrust restraint design can refer to Chapter 9 of the 1995 American Water Works Association Manual M9 "Concrete Pressure Pipes" (Reference 5). Plate B-10 shows the force diagram generated by flow in a bend in a pipe and also gives the equation for computing the thrust force. An example computation of a thrust force for a given surge pressure and a bend angle is presented on Plate B-11, Appendix B.

<u>Joint with Small Deflections</u> The thrust at bevel pipe or standard pipe installed with angular deflection is usually so small that supplement restraint is not required. The unbalanced force due to changes in grade or alignment can be resisted by frictional force between the pipe and the surrounding soil.

For small horizontal deflections with joints free to rotate, thrust T at deflected joints on long-radius horizontal curves is resisted by friction on the top and bottom of pipe as shown in Figure (a) on Plate B-12. The total friction developed is equal to the thrust and acts in the opposite direction. Additional restraint is not required when

$$T \leq f\,L_p(2W_e + W_w + W_p) \qquad (9)$$

Where, T = 2PA sin(θ/2) = resultant thrust force, lb, where θ is deflection angle created by the deflected joint, in degrees
f = coefficient of friction between pipe and soil
L_p = length of standard or beveled pipe (ft)
W_e = earth cover load over pipe (lb/ft)
W_w = weight of water inside the pipe (lb/ft)
W_p = weight of pipe (lb/ft)

The value of the frictional resistance depends on the material in contact with the backfill and the soil used in the backfill. For a concrete pipe with crushed stone, compacted cement-stabilized sand, or sand backfill, an allowable coefficient of friction of 0.3 can be used. To account for submerged conditions, a soil unit weight of 60 pcf should be used to compute the weight of compacted backfill on the pipe.

For small vertical deflections with joints free to rotate, uplift thrust at deflected joints on long-radius vertical curves is resisted by the combined dead weight W_t as shown in Figure (b) on Plate B-12. Additional restraint is not required when

$$T \leq L_p(W_e + W_w + W_p)\cos(\varphi - \theta/2) \qquad (10)$$

Where, T = 2PA sin(θ/2) = resultant thrust force, lb, where θ is deflection angle created by the angular deflection of joint, in degrees
φ = slope angle, in degrees
L_p = length of standard or beveled pipe (ft)
W_e = earth cover load over pipe (lb/ft)
W_w = weight of water inside the pipe (lb/ft)
W_p = weight of pipe (lb/ft)

6.5 Proposed Accesses Road Improvements

The pavement design recommendations developed herein are in accordance with the "AASHTO Guide for Design of Pavement Structures," 1993 edition (Reference 3). The detail traffic load for the driveway is not available at this time, but is expected to be light. We assumed 50,000 and 300,000 of 18-kip equivalent single axle load (ESAL) repetitions over the 20-year design life for the automobile traffic and truck traffic respectively. These assumptions should be verified during design and after traffic data is available. AEC also recommends that areas subjected to frequent truck traffic be provided with Portland Cement Concrete pavement.

6.5.1 Pavement Thickness

AEC recommend that areas subject to frequent truck traffic be provided with rigid pavement as they require less maintenance and rehabilitation. The parameters used to compute the rigid pavement sections are as follows:

Overall Standard Deviation (S_0)	0.34
Initial Serviceability (P_0)	4.5
Terminal Serviceability (P_t)	2.5
Reliability Level (R)	90%
Overall Drainage Coefficient (C_d)	1
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	30,000 psi
Effective Modulus of Subgrade Reaction (k)	74 pci
Mean Concrete Modulus of Rupture (S'_c)	600 psi (28 days)
Concrete Elastic Modulus (E_c)	3.4×10^6 psi
Compressive Strength of Concrete (f'_c)	3,500 psi (28 days)

We recommend the following rigid pavement sections based on the above assumed traffic load:

TABLE 9. RECOMMENDED RIGID PAVEMENT SECTION THICKNESS

SECTION	THICKNESS	
	Truck Traffic	Automobile Traffic
Surface Course	8-inch Concrete	6-inch Concrete
Subgrade	8-inch Stabilized Subgrade (at least 7% Lime)	

Note: The actual percentage of lime should be confirmed by laboratory testing prior to construction.

Rigid Pavement Steel Reinforcing 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:

$$A_s = FLW/(2f_s) \tag{11}$$

Where:
- A_s = Required cross-sectional area of reinforcing steel per foot of width
- 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
- f_s = Allowable working stress in steel, psi; typically, a value equivalent to 75 percent of steel yield strength is used for working stress, for Grade 60 steel, use 45,000 psi

The above equation is for both longitudinal and transverse steel. The steel rebars should be grade 60, the concrete compressive strength at 28 days should be 3,500 psi.

6.5.2 Pavement Subgrade Preparation

We recommend that subgrade preparation include removal of vegetation, roots, as well as debris to a minimum depth of 6 inches below existing ground. The exposed surface should then be proof-rolled in accordance with the 2004 TxDOT Standard Specifications Item 216. The surface should then be checked by qualified geotechnical professionals to identify any weak or dry soils, and deleterious materials. These materials should be removed and replaced with select fill.

To provide a stable subgrade for the pavement, we recommend that the top 8 inches of exposed soils after proof-rolling be scarified and then stabilized with at least 7 percent hydrated lime (by dry soil weight). The percent lime required for stabilization should be determined by laboratory testing prior to construction. Stabilized soils should be placed in no more than 8-inch loose lifts and compacted to at least 95 percent of the ASTM D-698 (Standard Proctor) maximum dry density at a moisture content between optimum and 3% wet of optimum. Lime stabilization should be done in accordance with Section 02336 of the 2002 COHSCS.

6.6 Proposed Site Drainage Improvements

We understand that a storm sewer outfall will be installed in open-cut method at the area near soil boring B-14; the approximate invert depth will be less than 15 feet. According to the soil information revealed by the 25-foot deep soil boring B-4, the top surface soil consists of 6 feet of fill fat clay (CH), which can be classified as OSHA type "C" soil, underlain by firm to very stiff fat clay (CH), which can be classified as OSHA type "B" soil, to the boring termination depth of 25 feet. Design soil parameters for trench excavation are presented in the following table. Recommendations for open-cut excavation can refer Section 6.4 of this report.

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TABLE 10. DESIGN SOIL PARAMETERS FOR TRENCH EXCAVATION (B-14)

Depth (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
0 - 6	Fill: stiff CH	120	60	800	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
6- 15	Firm to Very Stiff CH	125	65	1000	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
N/A	Select Fill	120	60	1600	0	1.00	1.00	1.00	200	22	0.45	0.63	2.20

Notes: (1) The depth shown in above table is from existing ground surface; CH = Fat Clay, CL = Lean Clay
(2) γ = unit weight for soil above water level, γ' = buoyant unit weight for soil below water level.
(3) C = ultimate cohesion, φ = ultimate angle of internal friction.
(4) K_a = coefficient of active earth pressure, K_0 = coefficient of at-rest earth pressure, K_p = coefficient of passive earth pressure, for level backfill.

6.7 Select Fill

Select fill should consist of uniform, non-active inorganic lean clays with a PI between 10 and 20 percent, and not more than 50 percent retained on No. 200 sieve. If clean, on-site fat clays are to be used, they should be stabilized with a percentage of lime determined by lime-series curve or pH method in a laboratory prior to construction. For estimating purposes, AEC recommends a minimum of 7 percent hydrated lime by dry weight prior to use. Lime stabilization should be done in accordance with Section 02336 of the 2002 COHSCS.

All material intended for use as select fill should be tested prior to use to confirm that it meets select fill criteria. The fill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of columns or walls should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors. The lime-stabilized onsite clays or select fill should be compacted to a minimum of 95 percent of the ASTM D 698 (Standard Proctor) maximum dry unit weight at a moisture content ranging between optimum and 3 percent above optimum.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 General

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.

7.2 Ground Water Control

Based on the ground water readings in the piezometer (Table 1), ground water might not be encountered during footing excavation for the proposed facilities. However, the ground water level will fluctuate with seasonal rainfall and other climatic events, and may be higher or lower during construction; therefore, AEC recommends that the Contractor verify the ground water conditions before starting excavation to determine if dewatering is required. The Contractor should be responsible for designing, installing and maintaining a dewatering system for ground water control and taking precautions to avoid distress to existing structures, as a result of dewatering. The following discussion is intended to guide the Contractor during design and construction of the dewatering system.

Seepage in the sandy lean clays/fat clay will probably be low. Seepage influx will be primarily from sand seams and pockets. Gravity drainage with sumps can be effective in removing seepage water in these clayey soil zones. If excavations extend into water-bearing silt or sand, well points may be used to lower the ground water level. Well points are not generally effective below a depth of about 15 feet below the top of the well point. Two-staged well points or deep wells with submersible pumps will be required when the dewatering depth is greater than 15 feet. Generally, the ground water depth should be lowered at least 2 feet below the excavation bottom to be able to work on a firm surface. Extended and/or extensive dewatering can result in settlement of existing structures in the vicinity; the Contractor is to take necessary precautions to minimize the effects on these structures.

7.3 Construction Monitoring

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

8.0 DESIGN REVIEW

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

9.0 GENERAL

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the 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.

10.0 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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11.0 CLOSING REMARKS

AEC appreciates the opportunity to be of service on this project and looks forward to our continuing association during the construction phase of this project and on future projects.

AVILES ENGINEERING CORPORATION

STATE OF TEXAS
DAJUN WANG
95106
LICENSED
PROFESSIONAL ENGINEER
10-23-07

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Chief Engineer

October 23, 2007

Reports Submitted:	4	Binkley & Barfield, Inc.
	1	File

Z:\ENGINEERING\REPORTS\2007\150-07 ALMEDA SIMS SLUDGE PROCESSING FACILITY IMPROVEMENT - BINKLEY & BARFIELD (DENNIS)\G150-07 FINAL.DOC

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2. NAVFAC DM-7.2, 1982, AFoundations and Earth Structures Design Manual 7.2", Alexandria, VA.

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4. AASHTO, 1997, AAASHTOWARE™ DARWin™ 3.0, Pavement Design and Analysis System@, Washington, D.C.

5. American Water Works Association, Manual M9, 1995, "Concrete Pressure Pipes", Denver, Colorado.

6. Joseph E. Bowles, Foundation Analysis and Design, published by McGraw Hill Book Co.

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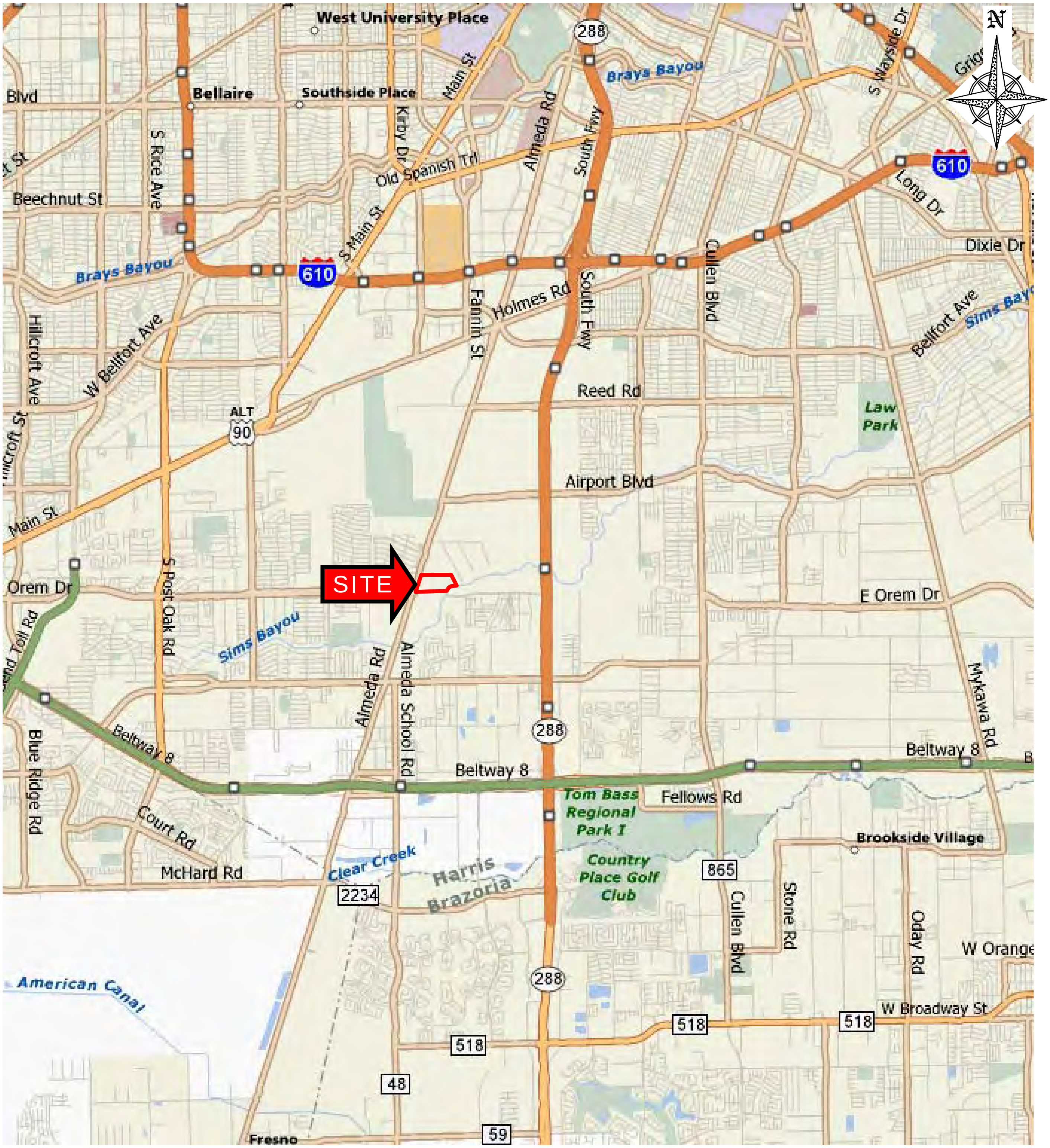
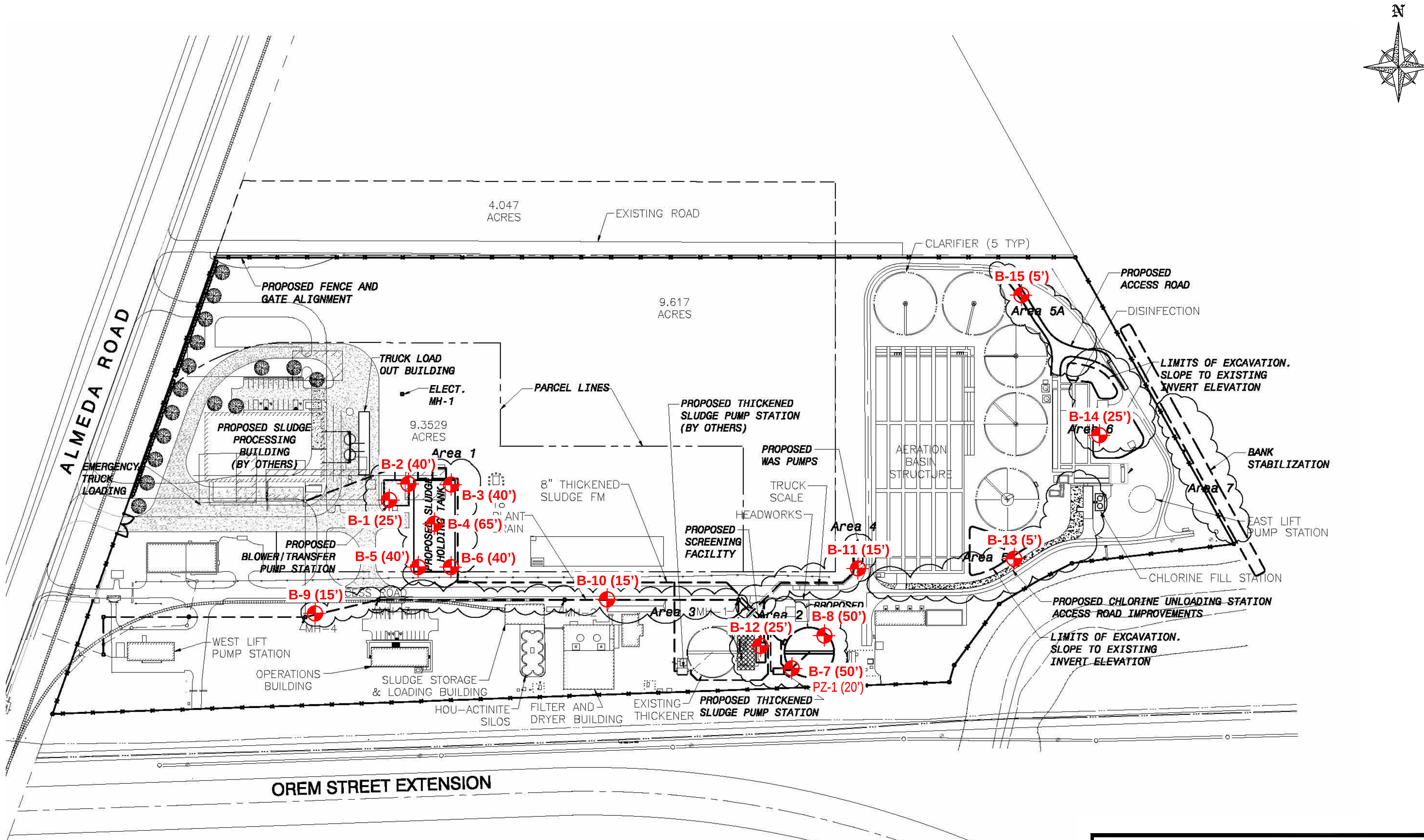
APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 thru A-17	Boring Logs
Plate A-18	Key to Symbols
Plate A-19	Classification of Soils for Engineering Purposes
Plate A-20	Terms Used on Boring Logs
Plate A-21	ASTM & TXDOT Designation for Soil Laboratory Tests
Plate A-22	Piezometer Installation Details (B-7)
Plate A-23	Consolidation Test Results (B-4, 13' - 15')
Plate A-24	Consolidation Test Results (B-7, 18' - 20')



AVILES ENGINEERING CORPORATION		
VICINITY MAP PROPOSED ALMEDA SIMS S.P.F. IMPROVEMENTS WBS R-000298-004-3, FILE NO. WW4903 HOUSTON, TEXAS		
AEC PROJECT NO. : G150-07	DATE: 07-19-07	
SCALE: N.T.S.	DRAFTED BY: BpJ	PLATE NO. : PLATE A-1



NOTE:

The following boring locations are proposed for Package I: B-1, B-2, B-4, B-5 and B-7 thru B-12.

The following boring locations are proposed for Package II: B-3, B-6 and B-13 thru B-15.

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BORING LOCATION PLAN

PROPOSED ALMEDA SIMS S.P.F. IMPROVEMENTS
WBS R-000298-004-3, FILE NO. WW4903
HOUSTON, TEXAS

AEC PROPOSAL NO.: G150-07	DATE: 07-19-07	SOURCE DRAWING PROVIDED BY: BINKLEY & BARFIELD
APPROXIMATE SCALE: 1" ~ 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2

AYLES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

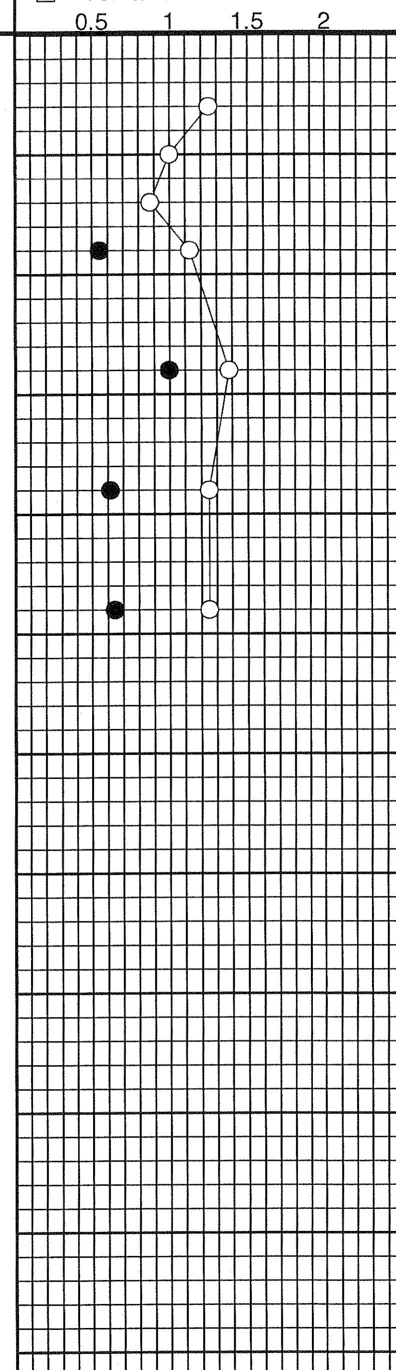
PROJECT: **Almeda Sims SPF Improvements** BORING **B-1**

DATE **07-10-07** 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
0			Fill: dark gray Fat Clay (CH), with wood pieces, gravel, sand seams, and shell		24					
					29			74	23	51
			Stiff to very stiff, gray Fat Clay (CH), with ferrous stains		35					
			-tan and gray 6'-8'		32		87	76	22	54
8			-tan, brown and light gray, with calcareous nodules 8'-10'		30	92				
			-reddish brown and light gray 13'-25'		28	93		53	19	34
16					32	91				
24					32	90				
			Termination depth = 25 feet.							
32										
40										
48										
56										

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 **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT NO. G150-07 **PLATE A-3**

PROJECT: Almeda Sims SPF Improvements

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

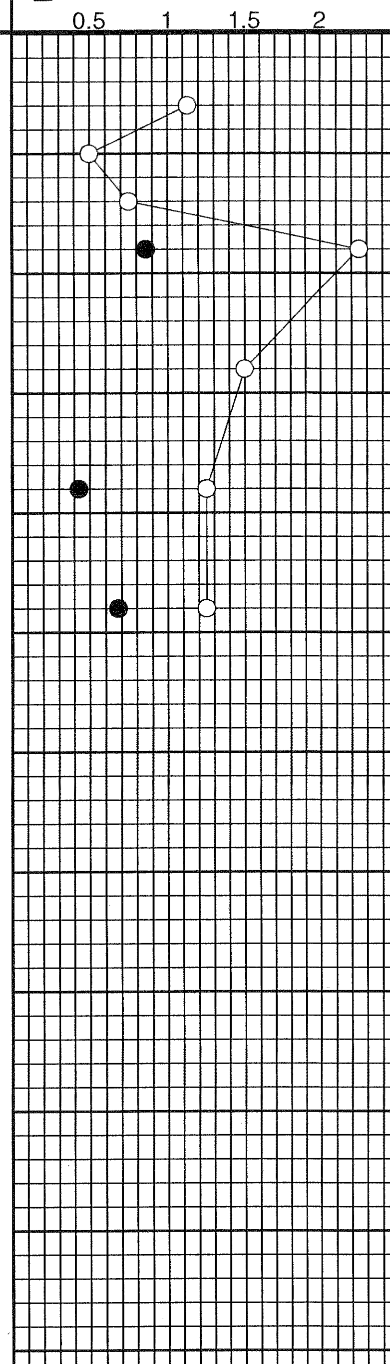
BORING B-2

DATE 07-11-07 **TYPE** 4" Dry Auger / Wet Rotary **LOCATION** See Boring Location Plan

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: dark gray Clayey Sand (SC), with gravel, wood and glass pieces and nails		20			46	20	26
	Fill: very stiff, dark gray Sandy Lean Clay (CL), with sand seams, gravel, shell and glass pieces		25					
			36					
	Firm to hard, light gray and tan Fat Clay (CH), with ferrous stains -reddish brown and light gray 8'-25', with calcareous nodules 8'-10'		34			77	21	56
8			21	107				
			27		99	68	23	45
16			33	89				
24			31	92				
	Medium dense, tan Silty Sand (SM), wet	24	26					
32	Brown and light gray Lean Clay w/Sand (CL), with silt seams		22		71	28	12	16
	Medium dense, brown and light gray Silt (ML)	27	28					
40	Termination depth = 40 feet.							
48								
56								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 26 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 26 FEET WHILE DRILLING

WATER LEVEL AT 15.8 FEET AFTER 1/4

DRILLED BY V&S CHECKED BY DDW LOGGED BY V&S

PROJECT NO. G150-07 **PLATE** A-4

PROJECT: **Almeda Sims SPF Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-3**

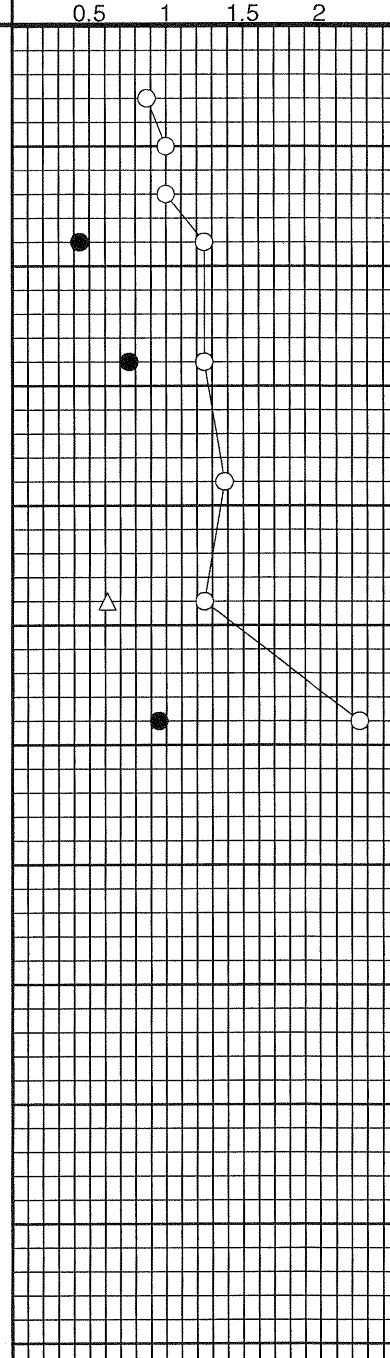
DATE **07-11-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET: 0, 8, 16, 24, 32, 40, 48, 56

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Fill: stabilized gray Sand (SM), with gravel		13					
Fill: stiff, dark gray Fat Clay (CH), with gravel and sandy clay seams 2'-6', sand seams, shell and ferrous stains		25			68	21	47
		31					
		34		94	80	24	56
Firm to hard, gray and tan Fat Clay (CH), with ferrous stains		32	91				
-reddish brown and light gray 13'-30'		27	94				
		31			82	25	57
		31	89				
-with silt pockets and calcareous nodules 28'-30'		28	92				
Medium dense, brown Silt (ML)	11	29		99			
- caved in at 36'							
Brown Lean CLay (CL), with silt seams	18	30			27	18	9
Termination depth = 40 feet.							

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **33** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING

WATER LEVEL AT **36** FEET AFTER **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT NO. G150-07 **PLATE A-5**

PROJECT: **Almeda Sims SPF Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

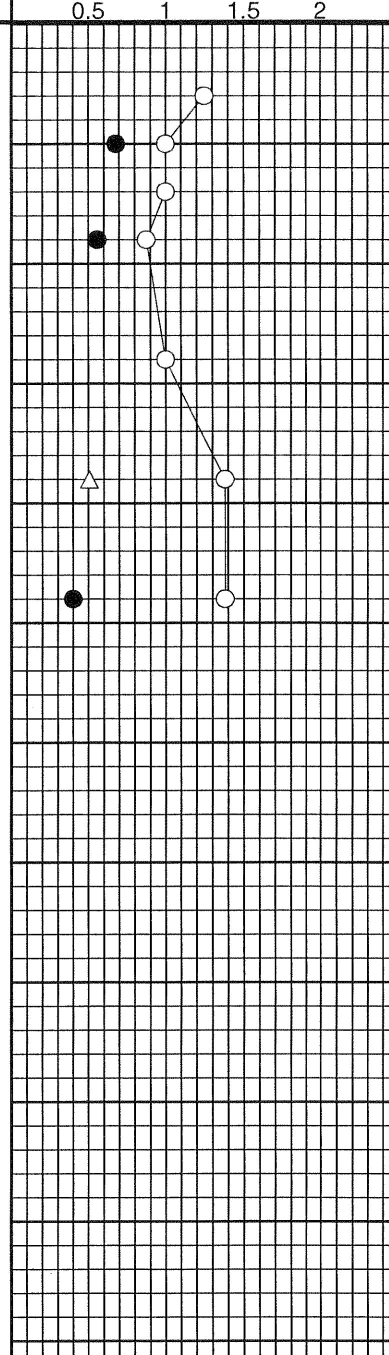
BORING **B-4**

DATE **07-10-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: stabilized dark gray Sand (SM), with gravel		9					
	Fill: very stiff, dark gray Fat Clay (CH), with shell, sand seams, gravel and ferrous stains		23			67	22	45
			31	89				
	Firm to very stiff, dark gray Fat Clay (CH), with ferrous stains		32			74	22	52
8	-reddish brown and light gray 8'-25', with calcareous nodules 8'-10'		30	94				
			27			66	23	43
16			31	90				
24			29	95				
	Medium dense, brown Sandy Silt (ML)							
	-wet at 30'	19	30		70			
32	Medium dense to dense, brown Silt (ML), wet	25	27					
40		29	26		98			
		37	27					
48	Dense, brown and light gray Silty Sand (SM), wet	32	28					
56		41	25		23			

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 **15.3** FEET AFTER **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-4**

DATE **07-10-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

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

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
				40	25						
64			Stiff, dark gray Fat Clay (CH)		26	96	● ○		57	19	38
			Termination depth = 65 feet.								
72											
80											
88											
96											
104											
112											

BORING DRILLED TO **30** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **30** FEET WHILE DRILLING
WATER LEVEL AT **15.3** FEET AFTER **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT NO. G150-07 **PLATE A-6**

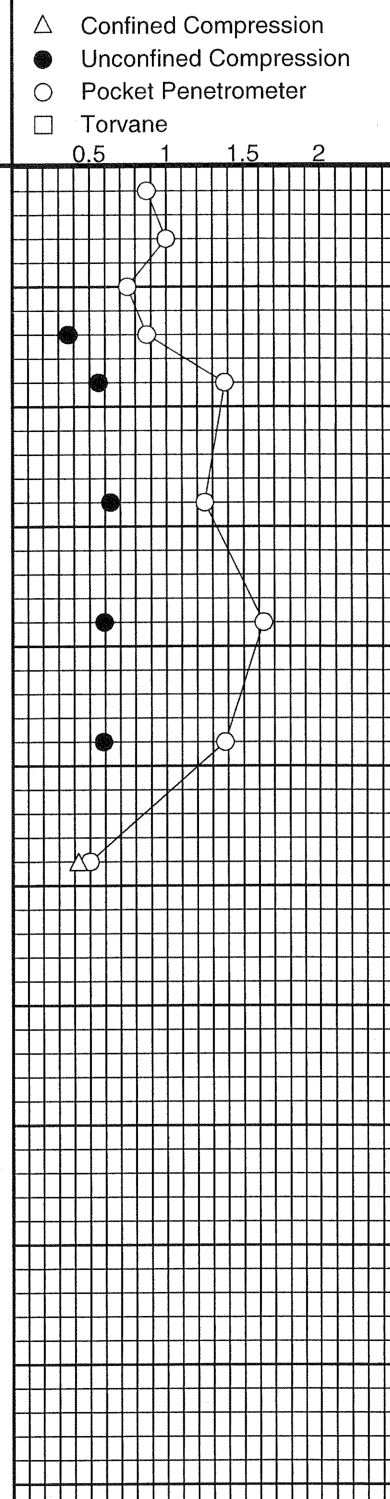
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-5**

DATE **07-10-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: stiff, dark gray Fat Clay (CH), with gravel, calcareous nodules and ferrous stains		28			79	22	57
			30					
	Firm to very stiff, tan and light gray Fat Clay (CH), with ferrous stains		33		93	76	22	54
	-with calcareous nodules 6'-8'		33	89				
8	-reddish brown and light gray 8'-20'		31	93				
			26	96		56	22	34
16								
			29	93				
24	Firm to very stiff, reddish brown and light gray Lean Clay w/Sand (CL)		29	97	84	30	16	14
	-tan 28'-30'		26	98				
32	Medium dense, brown Sandy Silt (ML), wet	30	27		53			
40	Hard, brown Lean Clay (CL), with silt seams	40	29					
	Termination depth = 40 feet.							
48								
56								

SHEAR STRENGTH, TSF



BORING DRILLED TO 24 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 24 FEET WHILE DRILLING

WATER LEVEL AT 15.5 FEET AFTER 1/4

DRILLED BY V&S CHECKED BY DDW LOGGED BY V&S

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

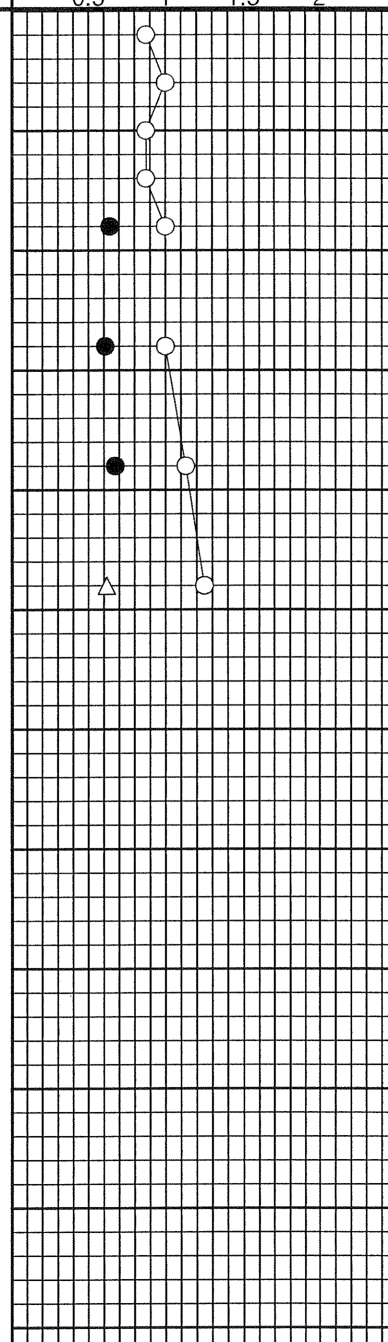
PROJECT: **Almeda Sims SPF Improvements** BORING **B-6**

DATE **07-10-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: stiff, dark gray Fat Clay (CH), with limestone fragments and calcareous nodules		33			70	24	46
			32					
	Stiff to very stiff, dark gray Fat Clay (CH), with ferrous stains		33			75	22	53
	-tan and light gray 4'-8'		34					
8	-with calcareous nodules 6'-8' -reddish brown 8'-10'		37	86				
	-reddish brown and light gray 13'-25'		31	88		67	21	46
16								
			31	91				
24			31	90				
	Medium dense to dense, tan Silty Sand (SM)							
		30	28					
32								
		25	28					
		40	31					
40	Termination depth = 40 feet.							
48								
56								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **26** FEET WHILE DRILLING

WATER LEVEL AT **N/A** FEET AFTER **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT: **Almeda Sims SPF Improvements**

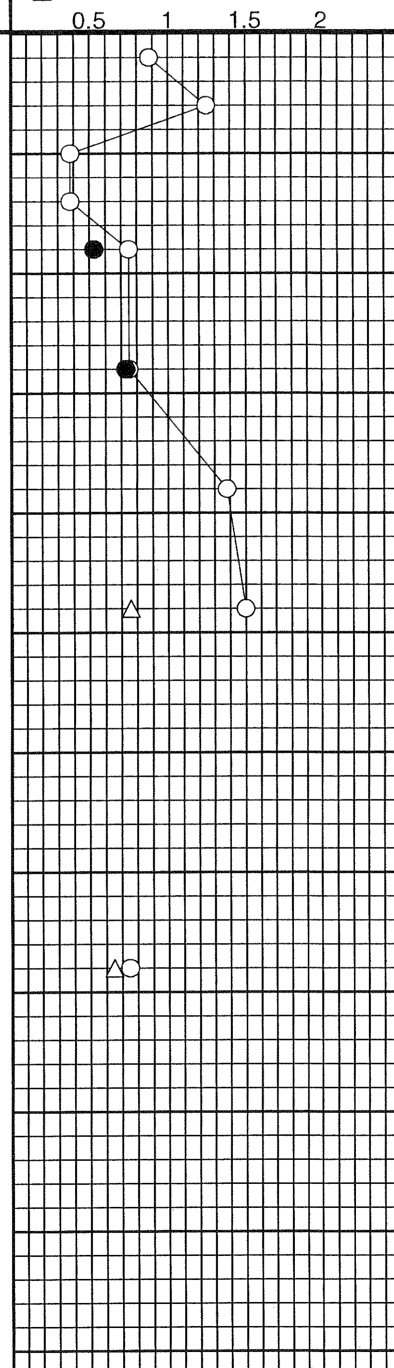
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-7**

DATE **06-25-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Fill: stiff to very stiff, brown and light gray Lean Clay (CL), with shell, calcareous nodules and roots		27					
		26			39	14	25
Fill: Firm to stiff, dark gray and tan Fat Clay (CH), with shell and gravel		28					
		28			60	19	41
Stiff to very stiff, dark gray Fat Clay (CH), with calcareous nodules and ferrous stains		30	91				
		30	92				
-reddish brown and light gray 18'-25'		34			93	27	66
		31	92				
Brown Silt w/Sand (ML), wet		25		84			
Medium dense, brown Silty Sand (SM)	26	24					
Stiff, brown Fat Clay (CH), with silt pockets		31	92	99	63	22	41
Medium dense, brown Silty Sand (SM), wet	19	25					
	15	26					
Termination depth = 50 feet.							

DEPTH IN FEET: 0, 8, 16, 24, 32, 40, 48, 56

BORING DRILLED TO **26** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **26** FEET WHILE DRILLING
WATER LEVEL AT **21** FEET AFTER **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

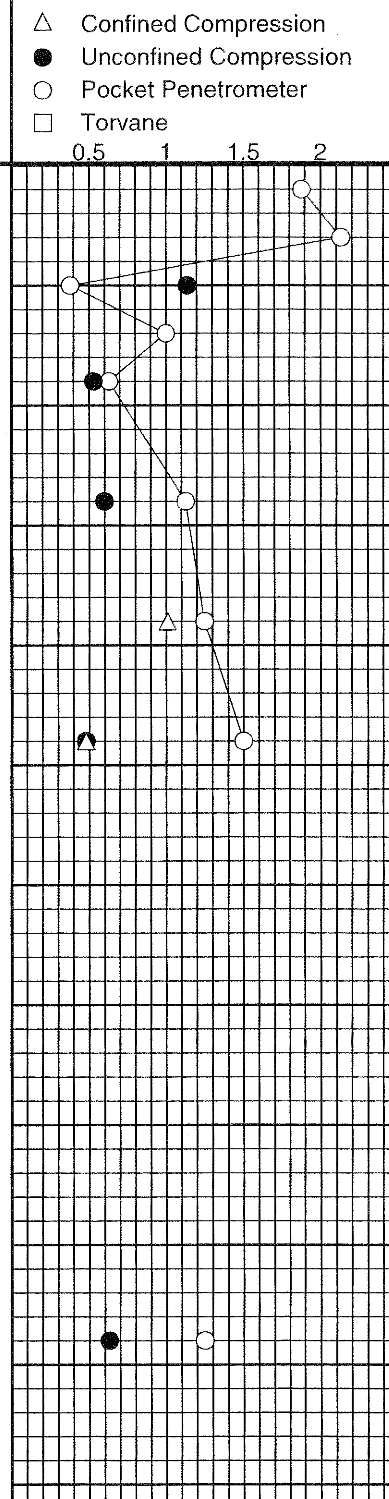
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-8**

DATE **06-25-07** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: firm to hard, brown and light gray Fat Clay (CH), with shell, calcareous nodules and roots		17					
			21			83	20	63
			24	100				
			32			61	19	42
8	Stiff to very stiff, gray Fat Clay (CH), with ferrous stains and calcareous nodules		39	91				
			31	90		78	22	56
16	-reddish brown and light gray 18'-25'		25	100				
24	Firm to very stiff, reddish brown Lean Clay (CL), with silt seams -caved in at 25.3'		26	98		40	16	24
	Brown Sandy Silt (ML), wet		25		70			
32	Loose to medium dense, brown Silty Sand (SM), wet -loose 33' - 35'	10	27					
40			30		45			
	-medium dense 43'-45'	19	23					
48	Stiff to very stiff, gray and tan Lean Clay (CL), with silt seams, calcareous nodules and ferrous stains		28	97				
56	Termination depth = 50 feet.							

SHEAR STRENGTH, TSF



BORING DRILLED TO **26** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **26** FEET WHILE DRILLING
WATER LEVEL AT **21.5** FEET AFTER **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT: **Almeda Sims SPF Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

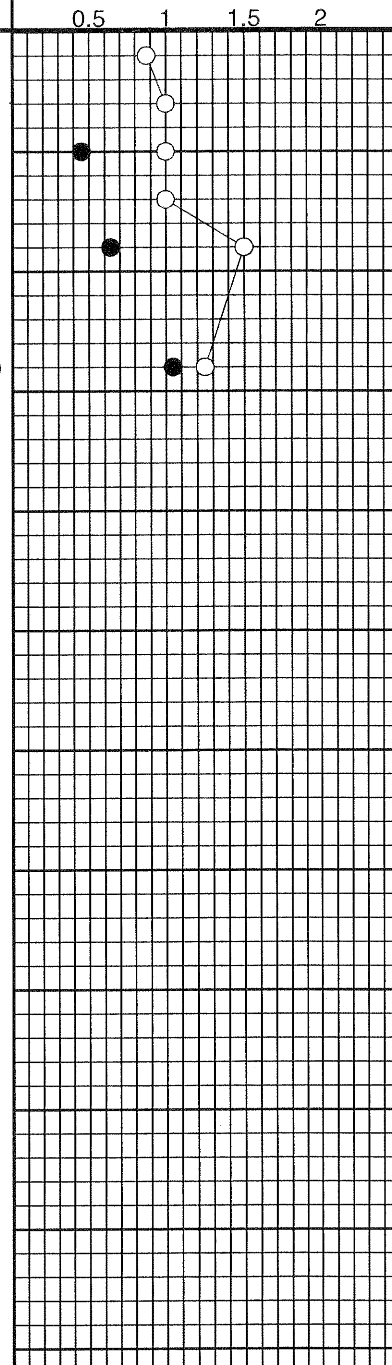
BORING **B-9**

DATE **07-11-07** 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	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Fill: stiff, dark gray Sand (SM), with gravel, clay seams, shell and roots		25						
			Firm to very stiff, dark gray Fat Clay (CH), with ferrous stains		30				71	22	49
			-gray and tan 4'-8'		30	89					
					31			91	76	21	55
8			-reddish brown and light gray, with siltstone fragments 8'-10' and calcareous nodules 8'-15'		30	95					
					25	100					
16			Termination depth = 15 feet.								
24											
32											
40											
48											
56											

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 1/4

DRILLED BY V&S CHECKED BY DDW LOGGED BY V&S

PROJECT NO. G150-07 **PLATE A-11**

PROJECT: **Almeda Sims SPF Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

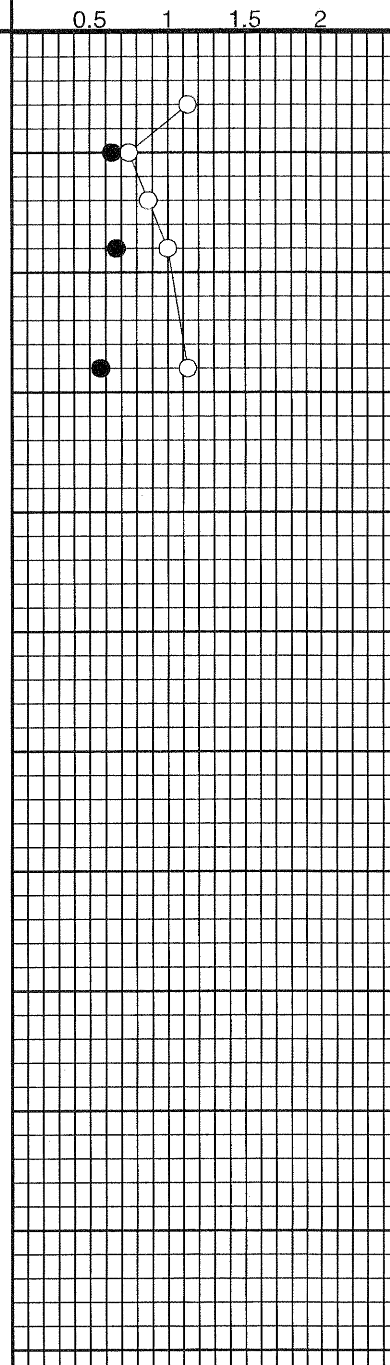
BORING **B-10**

DATE **07-11-07** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Concrete pavement: 8-inch							
	Crushed limestone base: 8-in		18					
	Fill: brown and light gray Sandy lean Clay (CL), with silt seams, gravel, shell, calcareous nodules and ferrous stains		29			82	22	60
			36	75				
8	Fill: Stiff to very stiff, gray and tan Fat Clay (CH), with ferrous stains		33			77	28	49
	-brown and gray, with shell 4'-6'		38	80				
	Stiff to very stiff, dark gray Fat Clay (CH), with ferrous stains							
	-gray and tan, with calcareous nodules 13'-15'		35	86				
16	Termination depth = 15 feet.							
24								
32								
40								
48								
56								

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 **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-11**

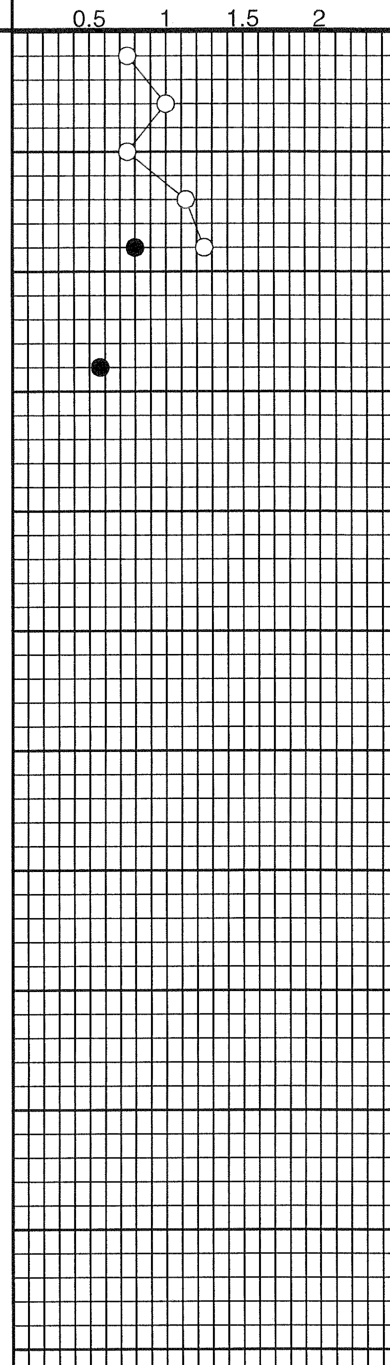
DATE **06-25-07** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: stiff to very stiff, gray and tan Fat Clay (CH)		27			76	22	54
	-brown and gray 2'-4'		28					
			31			76	22	54
			29					
8	Stiff to very stiff, dark gray Fat Clay (CH), with calcareous and ferrous nodules		31	89				
			32	90				
16	Termination depth = 15 feet.							

Depth scale: 0, 8, 16, 24, 32, 40, 48, 56

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 **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

PROJECT NO. G150-07 **PLATE A-13**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-12**

DATE **06-25-07** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: very stiff, gray and tan Fat Clay (CH), with shell, gravel, calcareous nodules and ferrous stains		19			65	20	45
			29					
			32		92	82	23	59
			34					
8	Firm to very stiff, tan and gray Fat Clay (CH), with calcareous and ferrous nodules		32	88				
			31	91				
16	-reddish brown and light gray 18'-25'		34	91		83	24	59
24			31	92				
	Termination depth = 25 feet.							
32								
40								
48								
56								

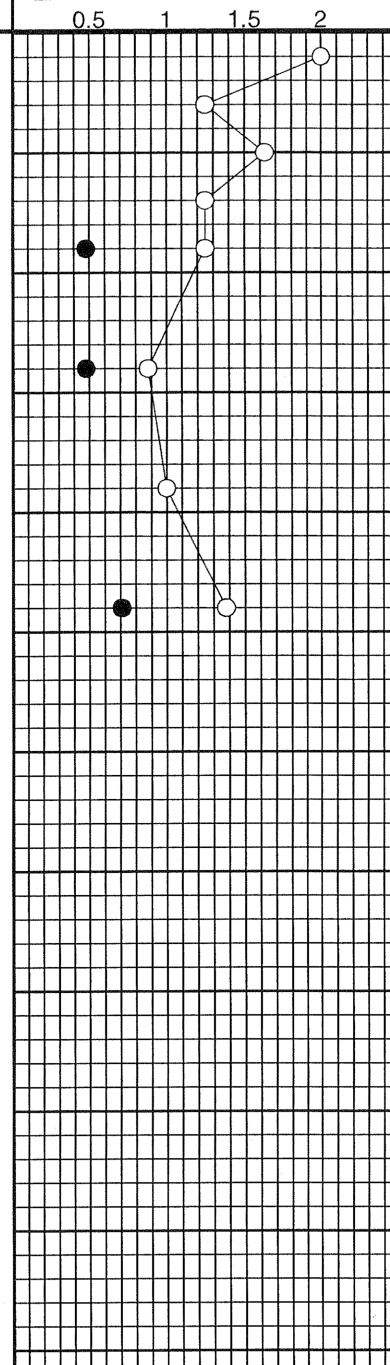
SHEAR STRENGTH, TSF

△ 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 **1/4**

DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

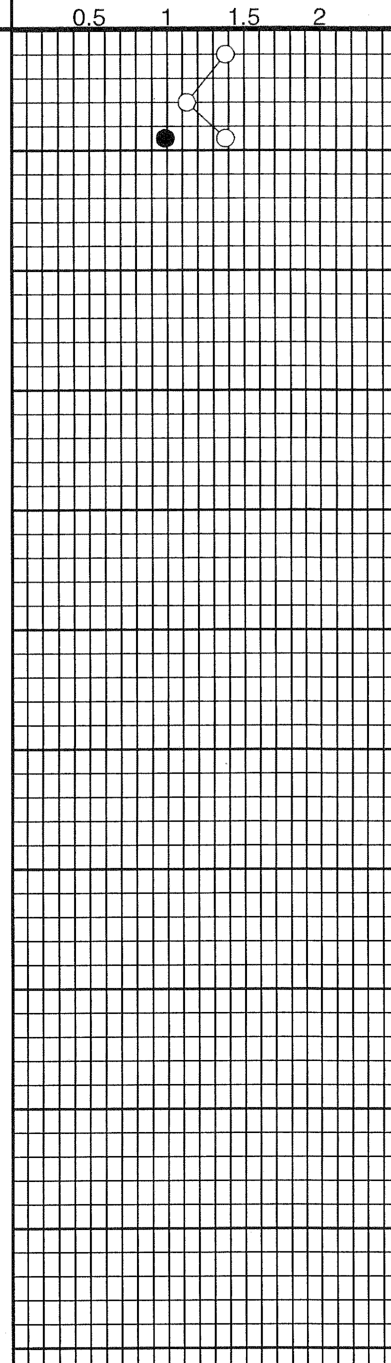
AYLES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements** BORING **B-13**

DATE **06-25-07** 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	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Fill: stiff to very stiff, gray and tan Lean Clay (CL), with gravel and calcareous nodules		15				40	14	26
					30						
			Fill: stiff to very stiff, brown and gray Fat Clay (CH), with calcareous and ferrous nodules, wood pieces		28	95					
8			Termination depth = 5 feet.								
16											
24											
32											
40											
48											
56											

SHEAR STRENGTH, TSF legend:
△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **5** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING
WATER LEVEL AT **N/A** FEET AFTER **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

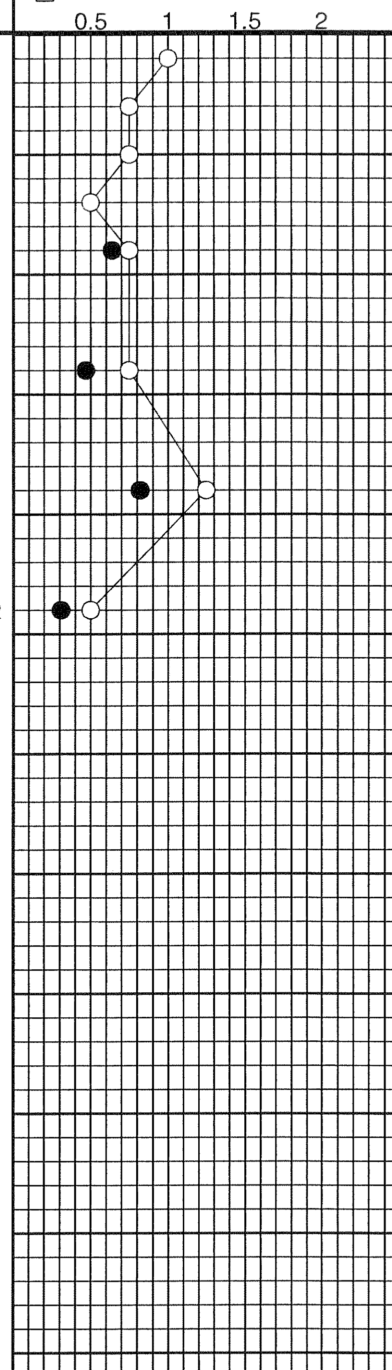
PROJECT: **Almeda Sims SPF Improvements** BORING **B-14**

DATE **06-25-07** 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
0			Fill: stiff, gray and tan Fat Clay (CH), with calcareous nodules and ferrous stains		32					
					33			75	23	52
					36					
			Firm to very stiff, gray Fat Clay (CH), with calcareous nodules and ferrous stains		33		93	79	22	57
8					33	92				
			-brown and light gray 13'-25'		37	85		81	24	57
16			-with silt seams 18'-25'		26	101				
24					25	102				
			Termination depth = 25 feet.							
32										
40										
48										
56										

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 **1/4**
DRILLED BY **V&S** CHECKED BY **DDW** LOGGED BY **V&S**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Almeda Sims SPF Improvements**

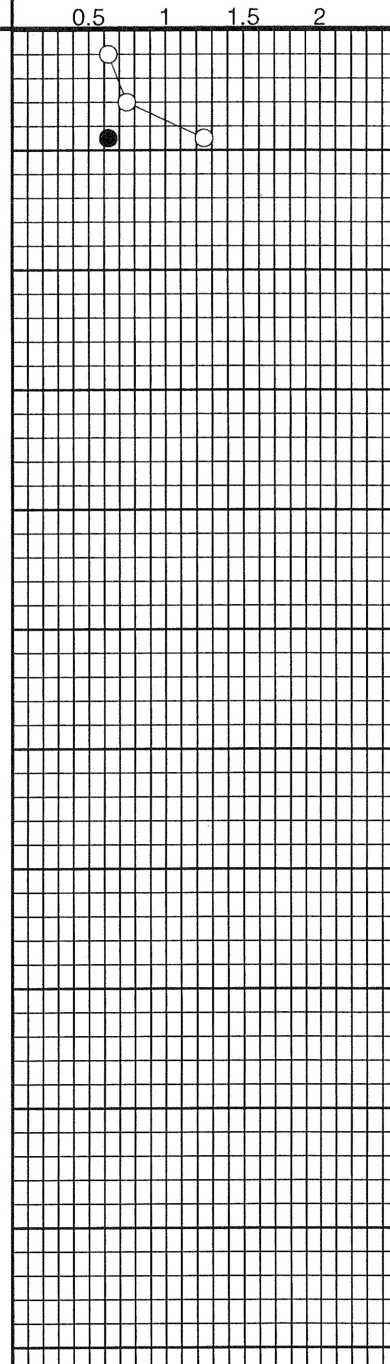
BORING **B-15**

DATE **06-25-07** 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	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Fill: stiff to very stiff, gray Fat Clay (CH), with gravel, shell, calcareous nodules, ferrous stains and roots		27						
					26				61	19	42
			-gray and brown 4'-5'		21	102					
			Termination depth = 5 feet.								
8											
16											
24											
32											
40											
48											
56											

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 5 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT N/A FEET WHILE DRILLING

WATER LEVEL AT N/A FEET AFTER 1/4

DRILLED BY V&S CHECKED BY DDW LOGGED BY V&S

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Fill
	High plasticity clay
	Silty sand
	Low plasticity clay
	Silt
	Paving
Soil Samplers	
	Auger
	Shelby Tube sampler
	Standard penetration test
	Rock core
Misc Symbols	
●	Shear strength; unconfined compression
○	Shear strength; pocket penetrometer
	Groundwater encountered during drilling
	Groundwater measured after drilling
△	Shear strength; confined compression

AYLES ENGINEERING CORP.

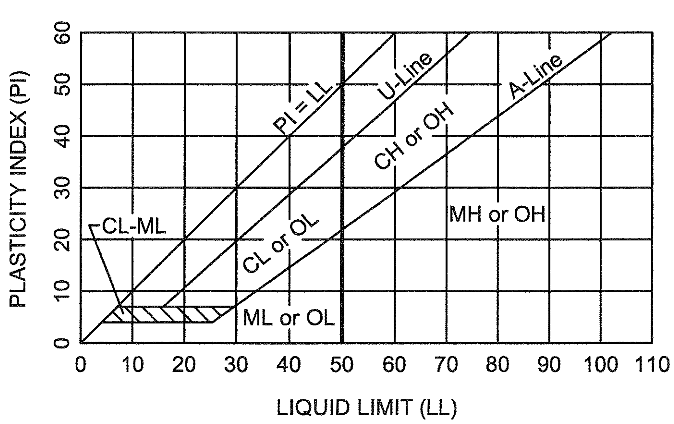
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

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

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

PLASTICITY CHART



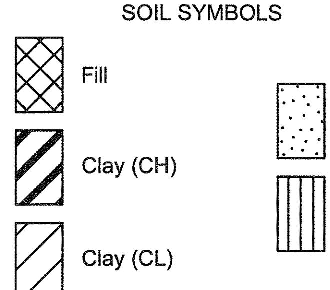
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

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

DEGREE OF PLASTICITY OF COHESIVE SOILS

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

SOIL SYMBOLS



AVILES ENGINEERING CORP.

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

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

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
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

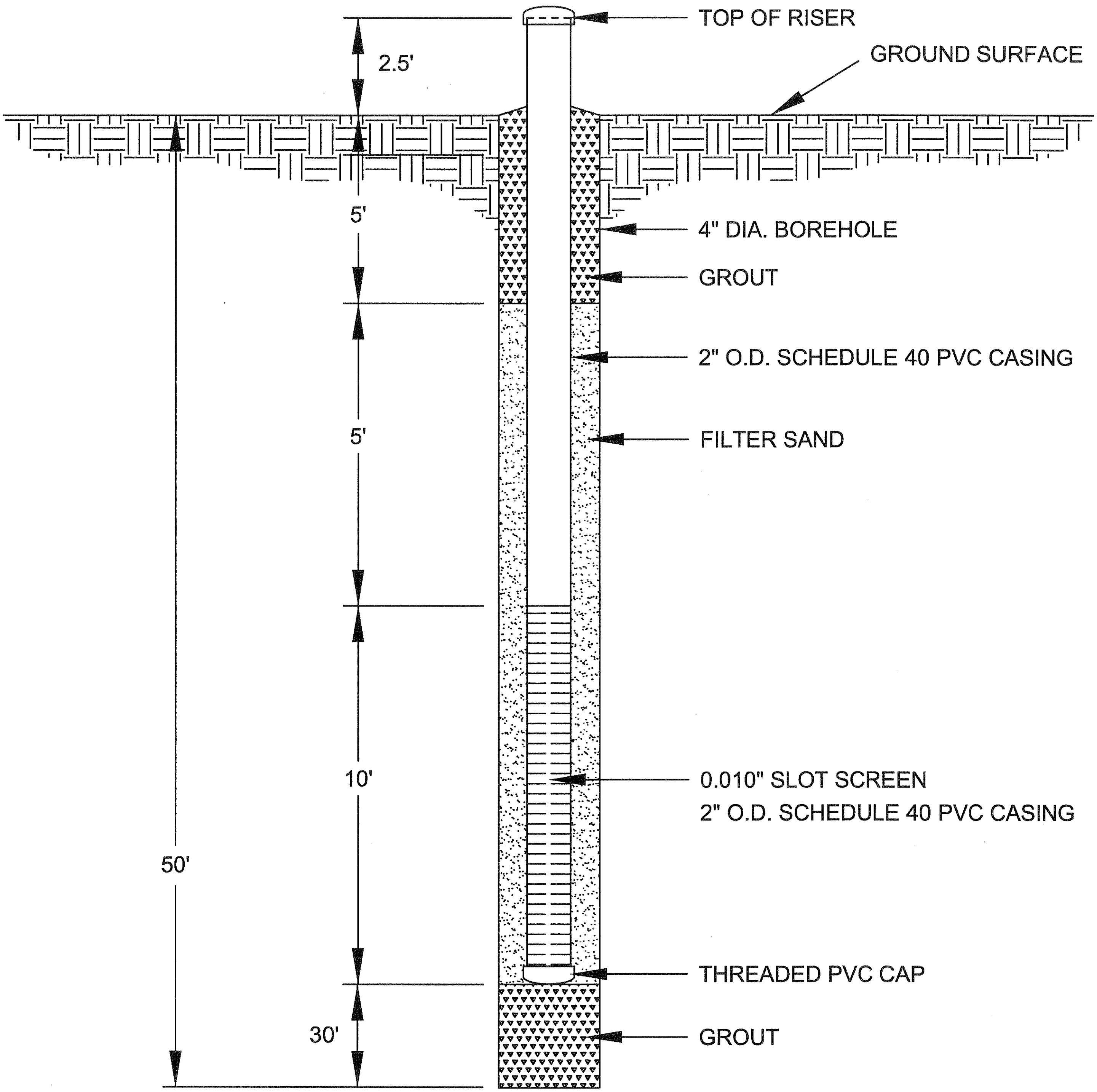
SOIL STRUCTURE

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



GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
N/A	07-03-07
19.7 FT	08-08-07

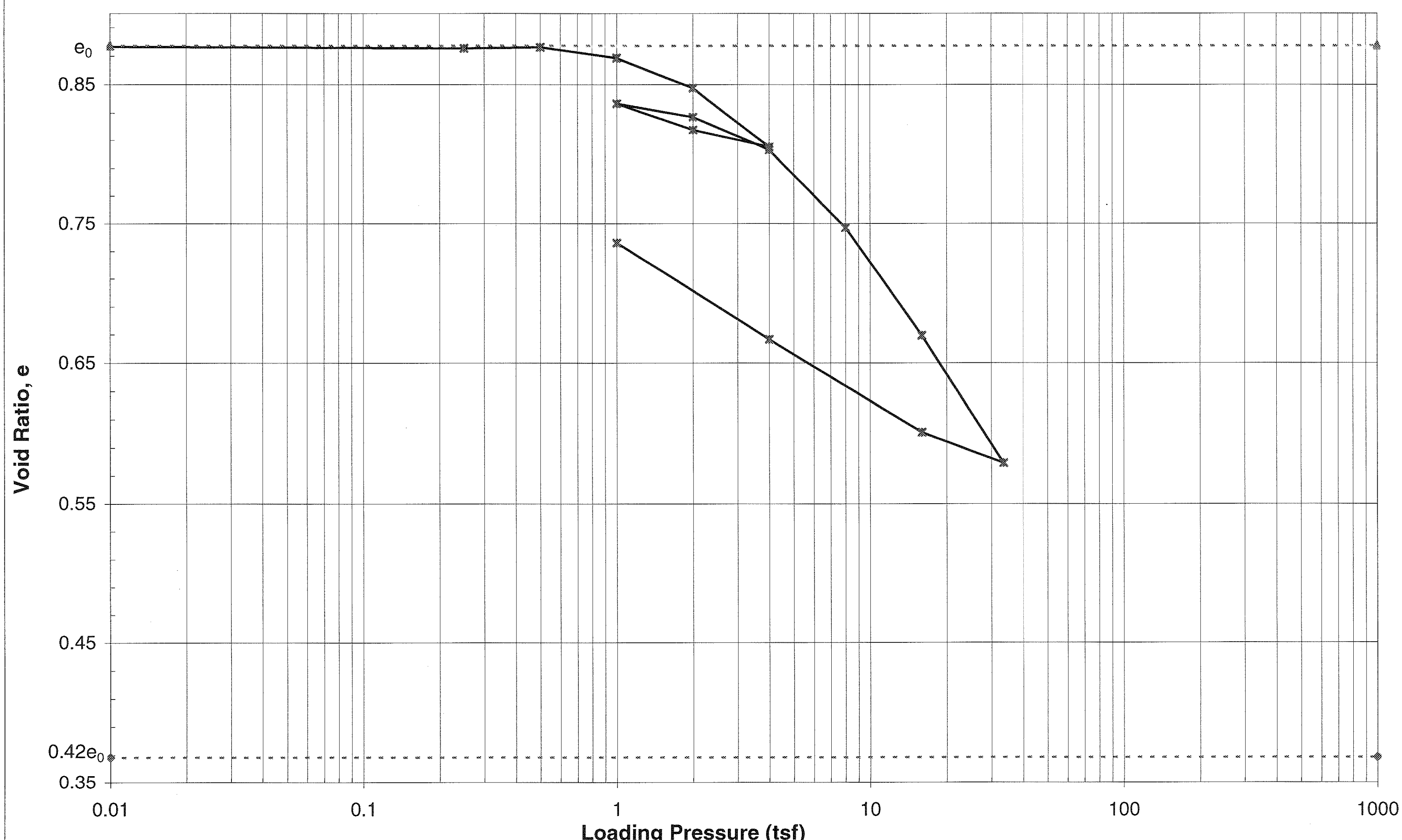
AVILES ENGINEERING CORPORATION

PIEZOMETER INSTALLATION DETAIL
BORING B-7 (PZ-1)

PROPOSED ALMEDA SIMS S.P.F. IMPROVEMENTS
WBS R-000298-004-3, FILE NO. WW4903
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DWG. BY:
G150-07	10-2-07	AVILES ENGINEERING CORP.
SCALE: N.T.S.	DRAWN BY: BpJ	PLATE NO.: PLATE A-22

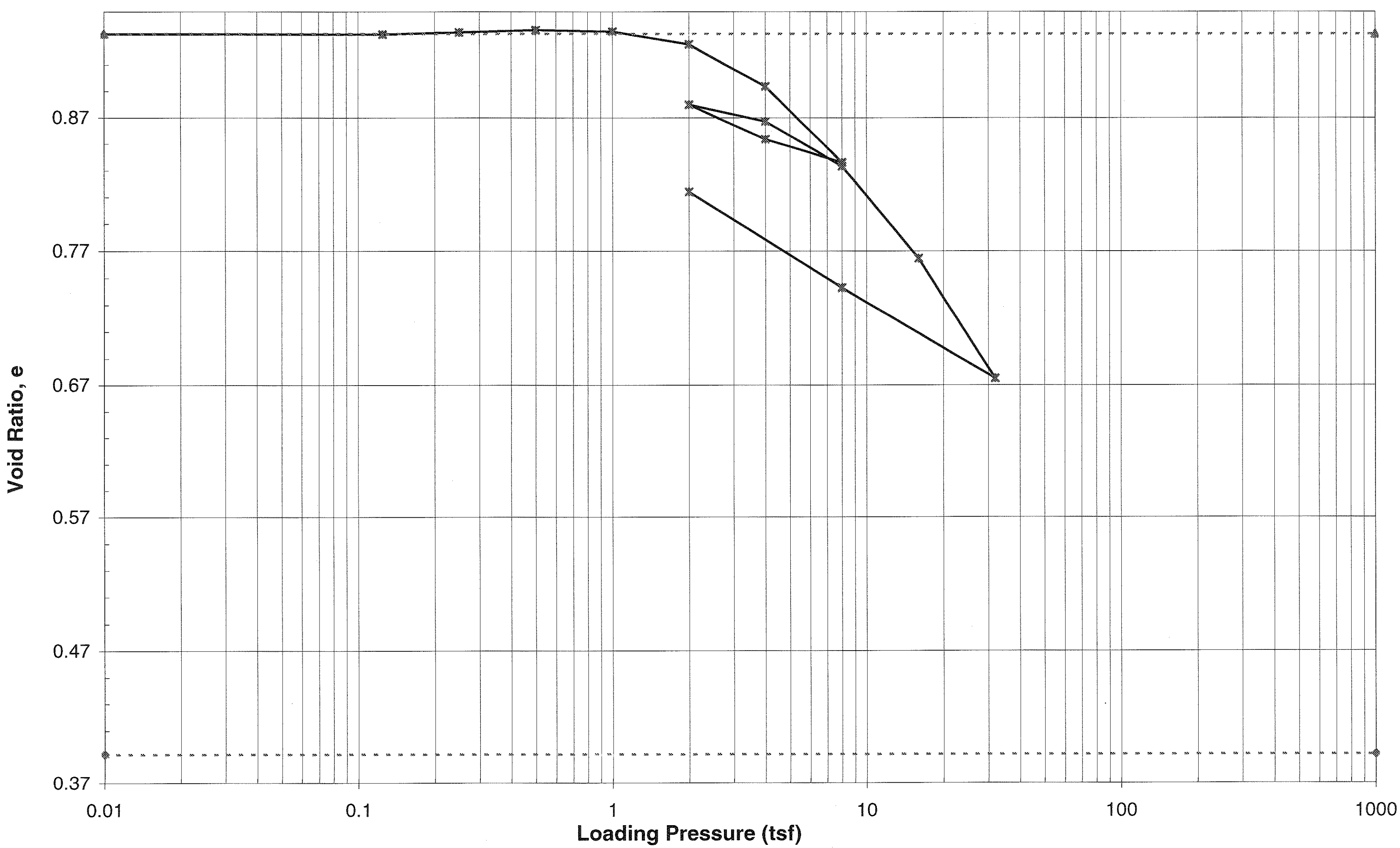
CONSOLIDATION TEST RESULTS



Project No.: G150-07 Project: Almedia Sims SPF Improvements Sample ID: B-4, 13 to 15 ft
Sample Description: Stiff, reddish brown and light gray Fat Clay (CH) e_0 = 0.877
Estimated consolidation Index (C_c): 0.284 Estimated recompression index (Cr): 0.050
Estimated OCR: 4.36 Estimated Preconsolidation Pressure (Pc): 4.1 tsf

PLATE A-23

CONSOLIDATION TEST RESULTS



Project No.: G150-07 Project: Almedia Sims SPF Improvements Sample ID: B-7, 18 to 20 ft

Sample Description: Stiff, reddish brown and light gray Fat Clay (CH) $e_0 = 0.933$

Estimated consolidation Index (C_c): 0.275 Estimated recompression index (Cr): 0.079

Estimated OCR: 3.52 Estimated Preconsolidation Pressure (Pc): 4.4 tsf

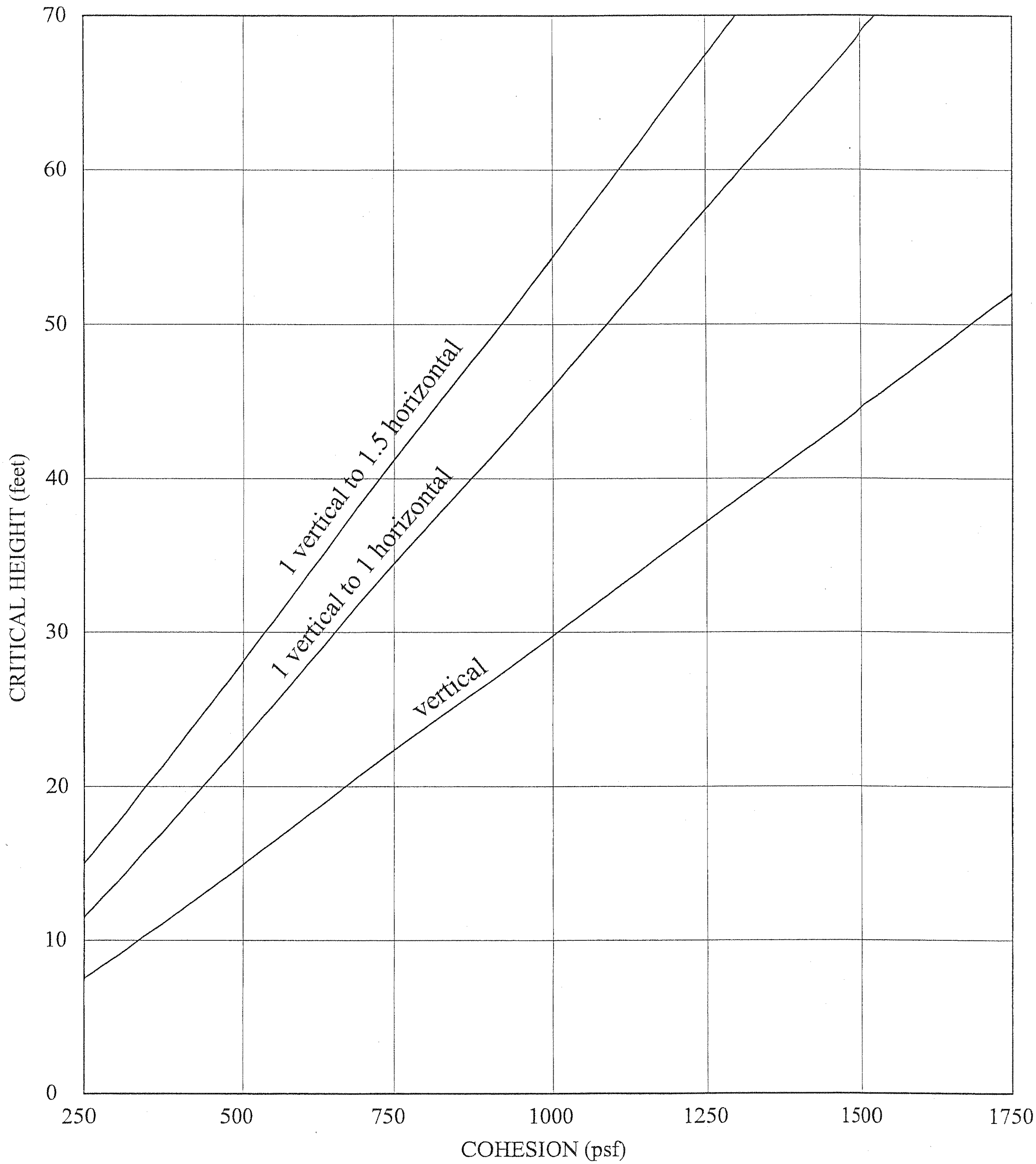
PLATE A-24

AVILES ENGINEERING CORP.

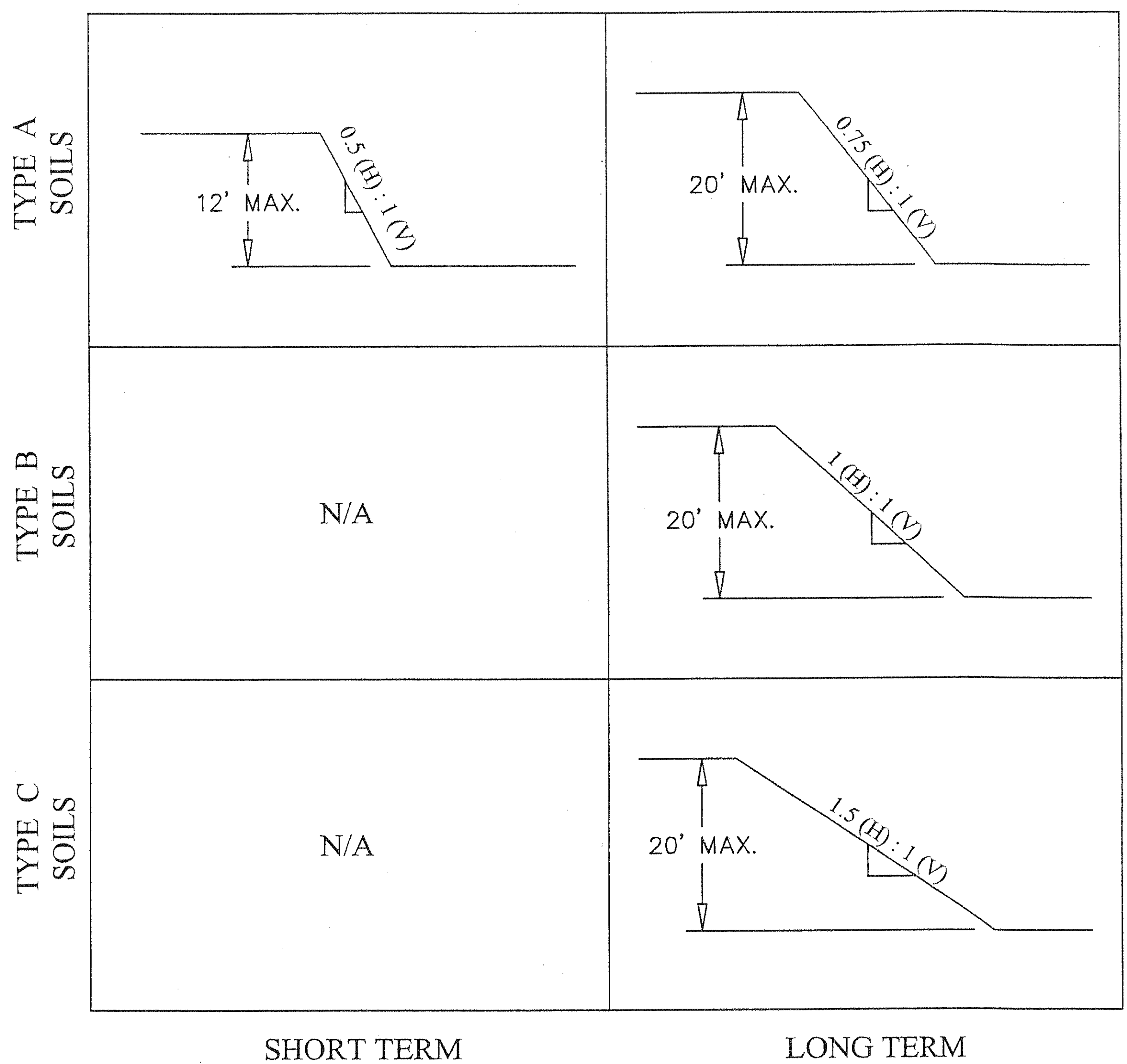
APENDIX B

AYLES ENGINEERING CORP.

Critical Heights of Cut Slopes in Nonfissured Clays



MAXIMUM ALLOWABLE SLOPES



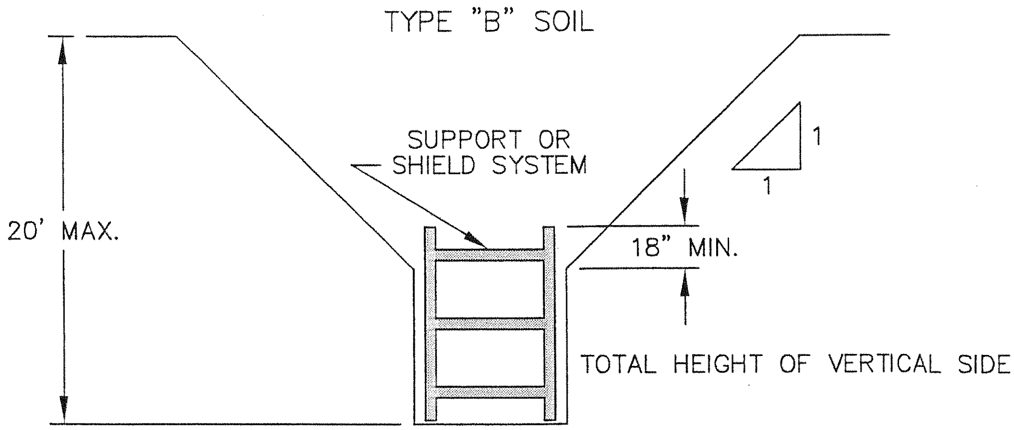
NOTES:

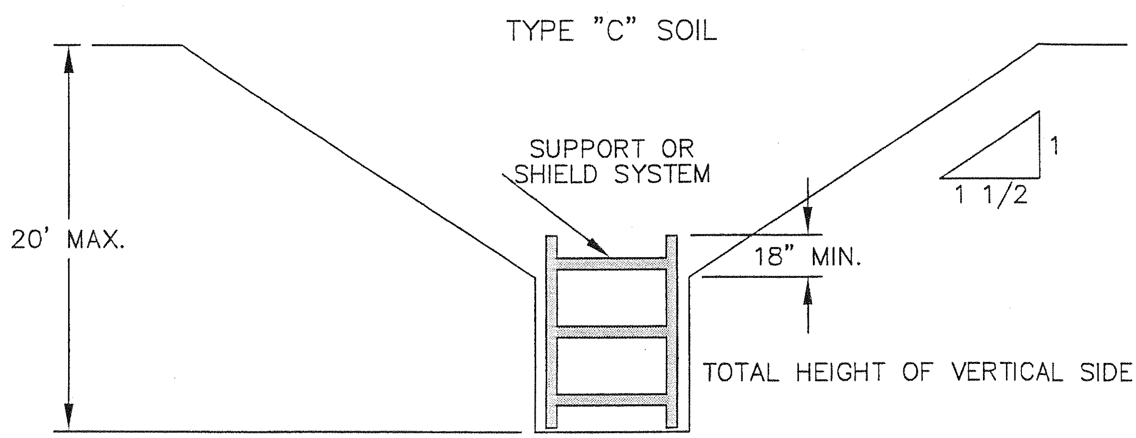
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(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





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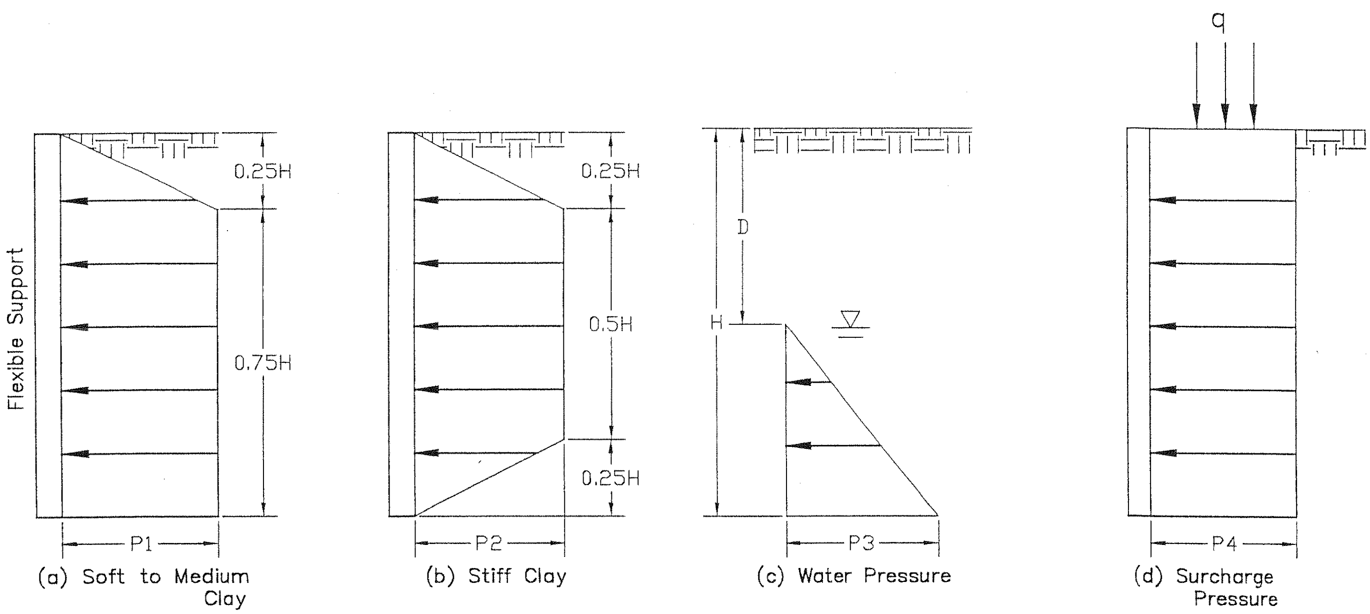
LATERAL PRESSURE DIAGRAMS

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 = 0.4 γH, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

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

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

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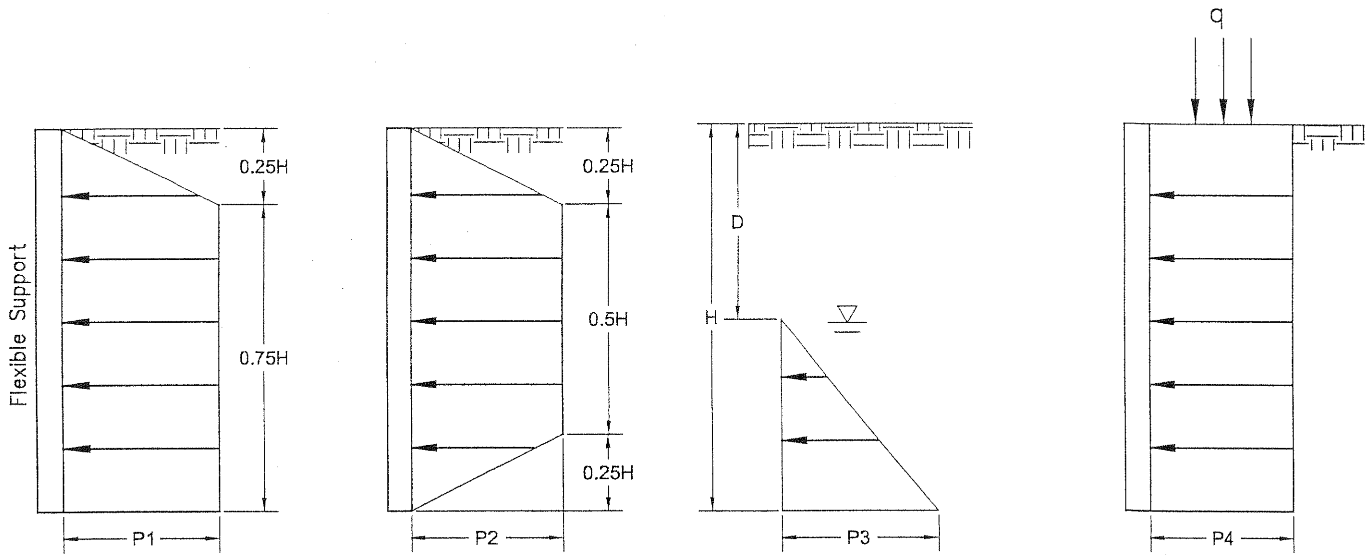
LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS



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

Empirical Pressure Distributions

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

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

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

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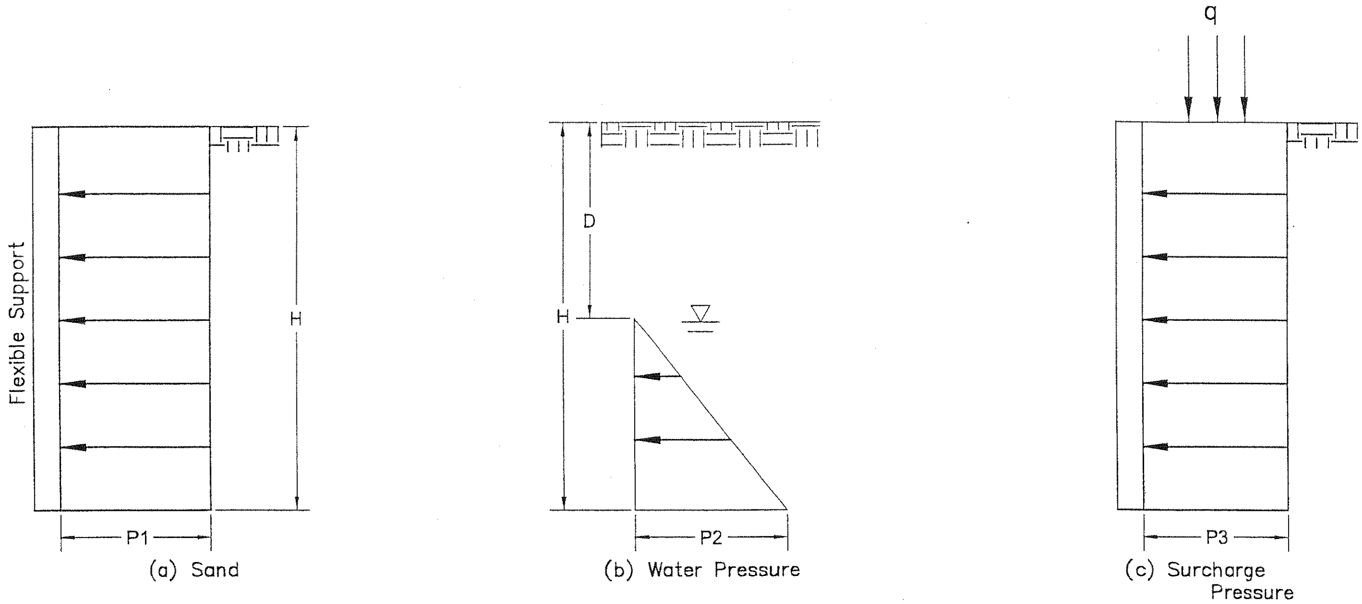
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



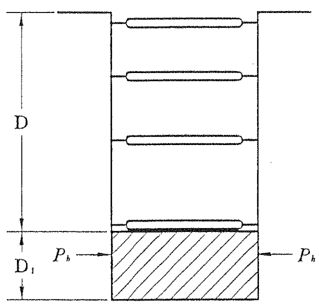
Empirical Pressure Distributions

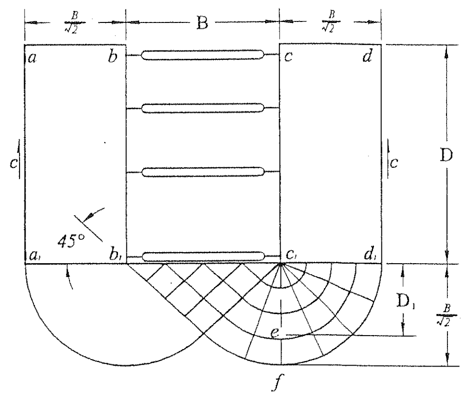
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma HK_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient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

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

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

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

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

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

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

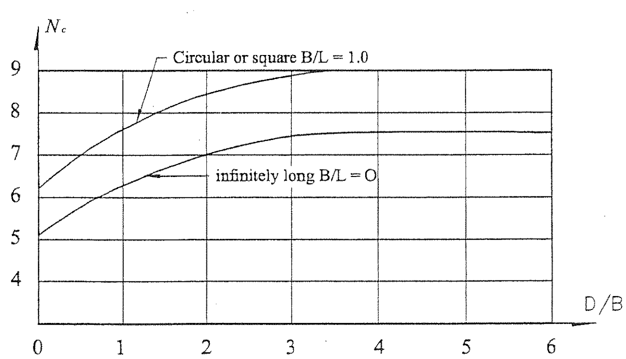
$$\textit{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \; ; D_1 \geq 5 \textit{ ft.}$$

Pressure on buried length, P_h:

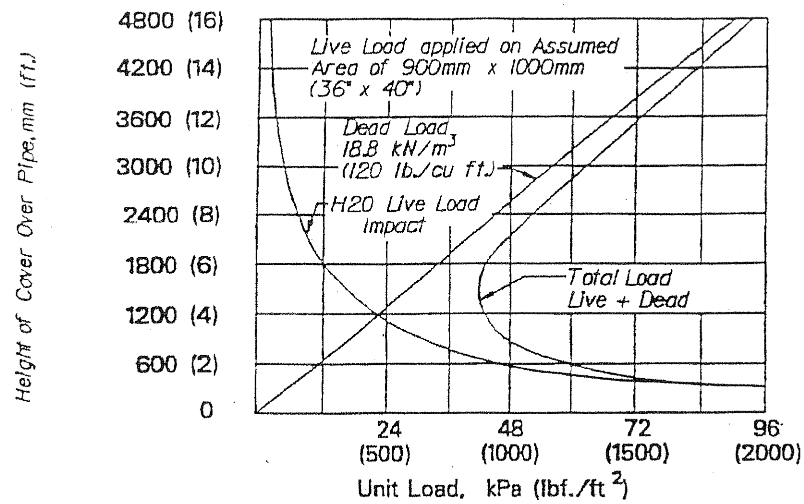
For $D_1 < 0.47B$; $P_h = 1.5\, D_1(\gamma D - 1.4\ CD/B - 3.14C)$

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

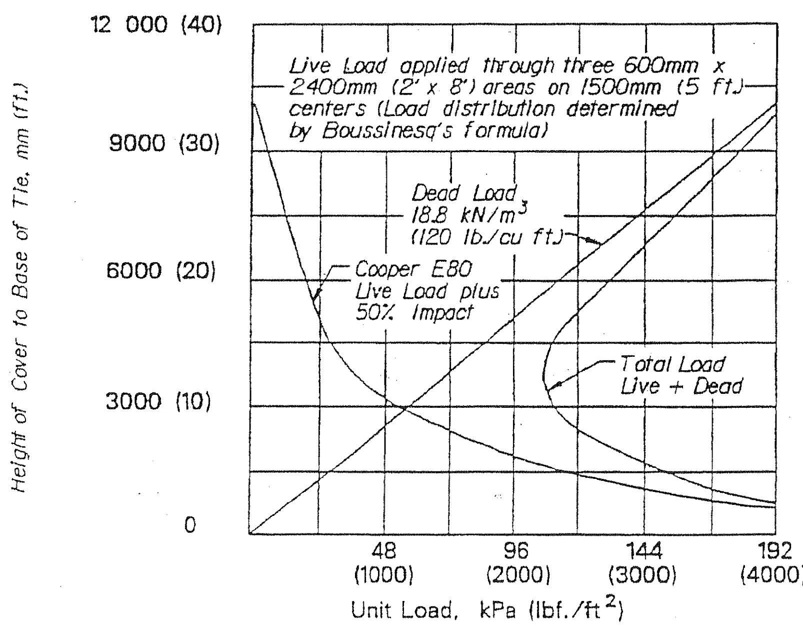
where; B = width of excavation



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



Combined H20 highway live load and dead load is a minimum at about 1500mm (5 ft.) of cover, applied through a pavement 300mm (1 ft.) thick.



Railroad live load, Cooper E80, combined with dead load is a minimum at about 3600mm (12 ft.) Load is applied through three 600mm x 2400mm (2x8 ft.) areas on 1500mm (5 ft.) centers.

Reference: U.S. Army Corps of Engineers, (1997), "Conduits, Culverts, and Pipes", Engineer Manual 1110-2-2902, Page 5-3

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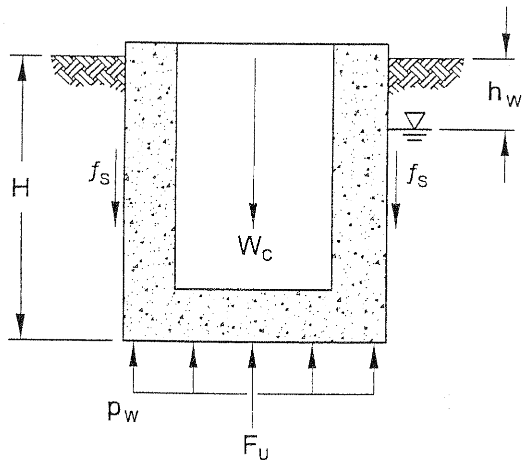
BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

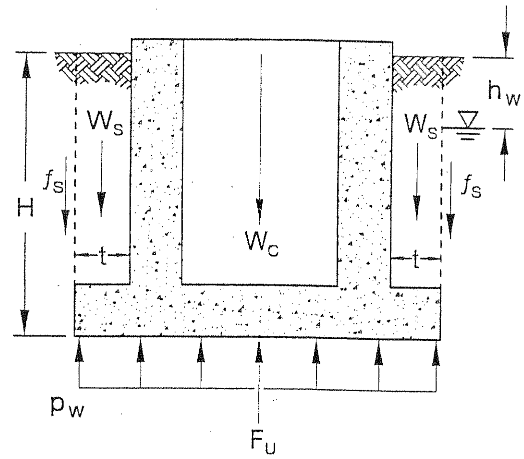


SOIL LAYER 1

SOIL LAYER 2

SOIL LAYER " j "

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)

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

cohesionless soils: $f_{S_j} = 0.75 \; K_S \; \acute{o}_{V_j} \; \tan\delta_j$

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

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

(b)

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

cohesionless soils: $f_{S_j} = 0.75 \; K_S \; \acute{o}_{V_j} \; \tan\Phi_j$

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

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

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
- $\acute{o}_{V_j}$ = effective overburden pressure at midpoint of soil layer " j ", psf.
- δ_j = 0.75 Φ_j, friction angle between soil layer " j " and concrete wall, degrees
- Φ_j = internal angle of friction of soil layer " j ", degrees
- K_S = 0.4, coefficient of lateral pressure
- h_j = thickness of soil layer " j ", ft.
- j = 1, 2, ...
- P_S = perimeter of structure base, ft.
- Q_S = ultimate skin friction, lbs.
- W_C = weight of structure, lbs.
- W_S = weight of backfill above base extension, lbs.
- S_{f_a} = 1.1, factor of safety for dead weight of structure
- S_{f_b} = 3.0, factor of safety for soil / structure friction
- S_{f_c} = 1.5, factor of safety for soil weight above base extension
- t = width of base extension, ft.

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

Reference:

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

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

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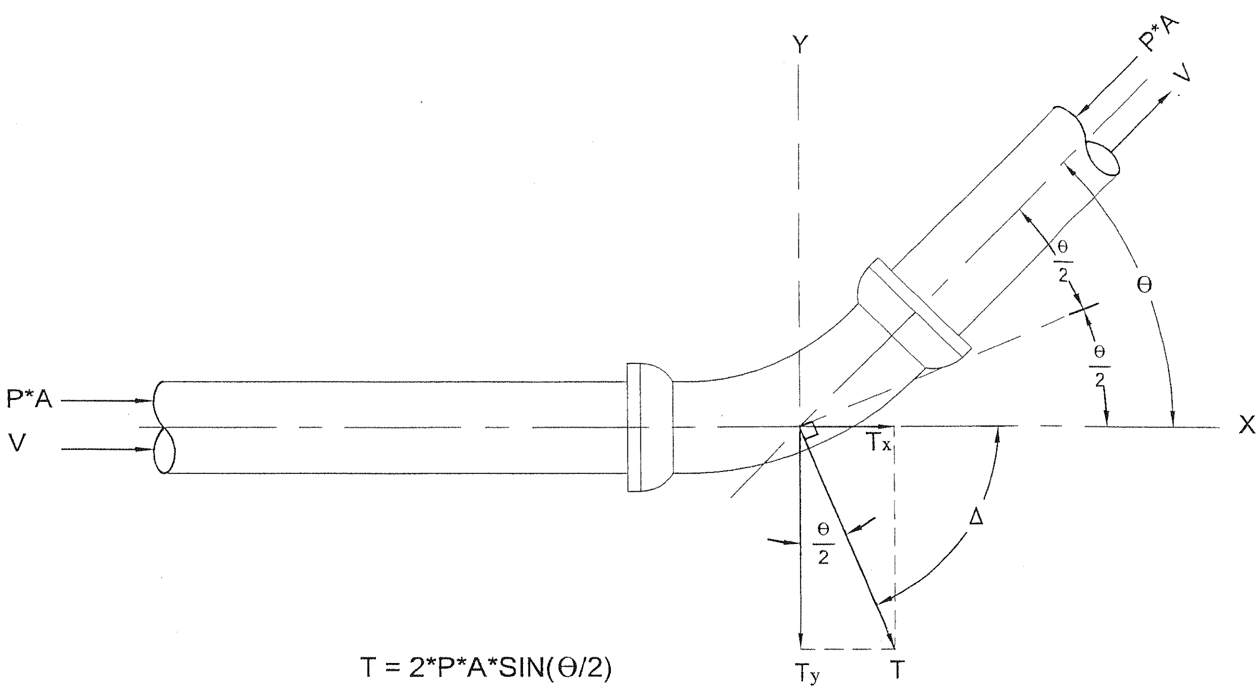
THRUST FORCE CALCULATION



$T = 2*P*A*SIN(\Theta/2)$

$T_x = P*A*(1-COS\Theta)$

$T_y = P*A*SIN\Theta$

$\Delta = (90-\Theta/2)$

Where:

T = resultant thrust force

T_x= thrust force component along the X axis

T_y= thrust force component along the Y axis

P = maximum sustained pressure

A = cross-sectional area of pipe = $(\pi/4)*(D)^2$

D = inside diameter conduit

Θ = angle of bend

Δ = angle between X axis and T

V = fluid velocity

THRUST FORCE EXAMPLE CALCULATION

Trust Force Example Calculation

$T = 2*P*A*SIN(\Theta/2)$

$T_x = P*A*SIN(1-COS\Theta)$

$T_y = P*A*SIN\ \Theta$

Where:

T = resultant thrust force

T_x = thrust force component along the X axis

T_y = thrust force component along the Y axis

P = maximum sustained pressure

A = cross-section area of pipe = $(\pi/4)*(D)^2$

D = inside diameter of conduit

U = angle of bend

Given: D = 24", P = 200 psi, $\Theta = 60°$

Find: T, T_x and T_y

$A = (\pi/4)*(24)^2 = 452.39\ in^2$

$T = 2*200*452.39*SIN(60/2) = 90,478$ lb

$T_x = 200*452.39*(1-COS60) = 45,239$ lb

$T_y = 200*452.39*SIN60 = 78,356$ lb

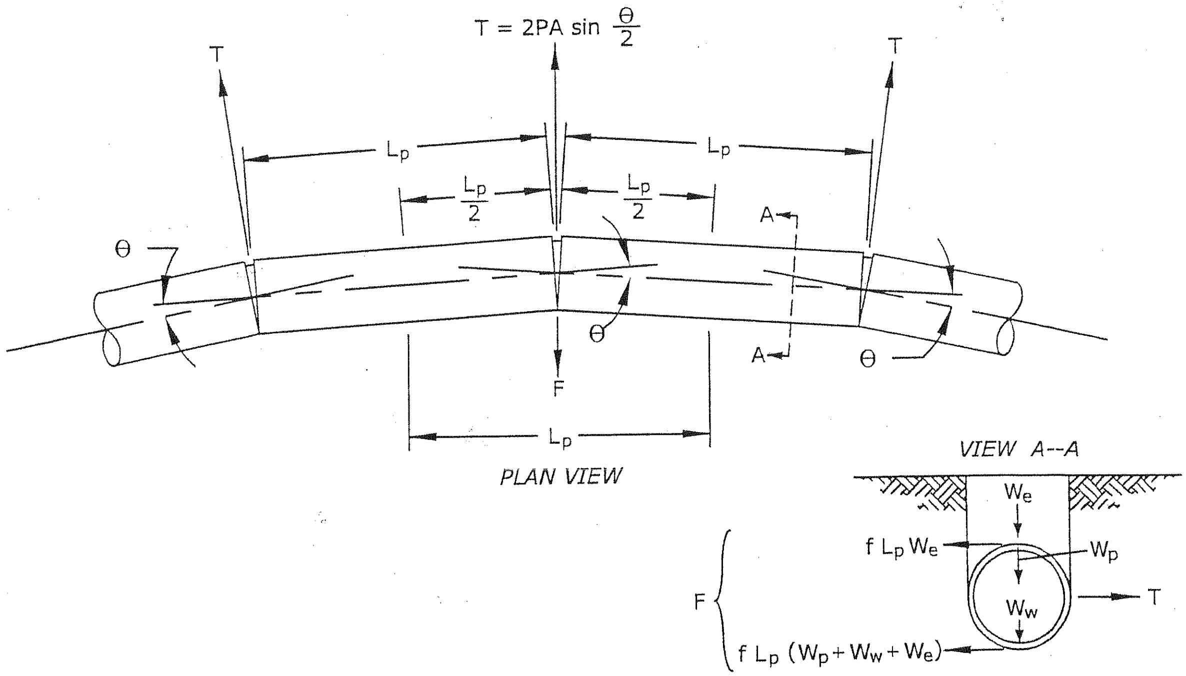


Figure a: RESTRAINT OF THRUST AT DEFLECTED JOINTS ON LONG-RADIUS HORIZONTAL CURVES

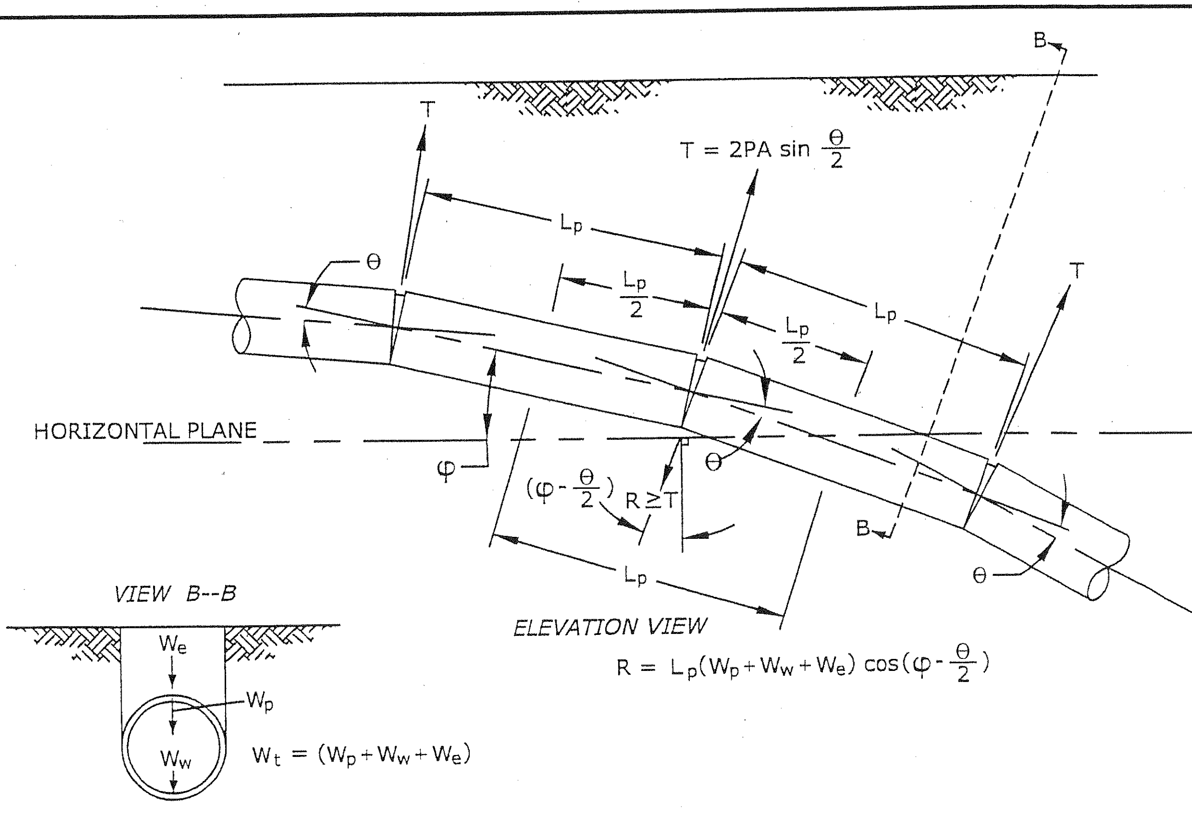


Figure b: RESTRAINT OF UPLIFT THRUST AT DEFLECTED JOINTS ON LONG-RADIUS VERTICAL CURVES

Reference: Manual Of Water Supply Practices, "Concrete Pressure Pipe", AWWA Manual M9