ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

GROUND STORAGE TANK NO.1 REPLACEMENT
PARK GLEN WEST PUMPING STATION
10630 SOUTH KIRKWOOD
WBS NO. S-000600-0044-4
HOUSTON, TEXAS

Reported to

IDS Engineering Group
Houston, Texas

by

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

REPORT NO. G148-13

January 2014

AVILES ENGINEERING CORP.

January 28, 2014

Mr. Paul Wallick, P.E.
IDS Engineering Group
13333 Northwest Freeway, Suite 300
Houston, Texas 77040

Reference: **Geotechnical Investigation**
Ground Storage Tank No. 1 Replacement
Park Glen West Pumping Station
WBS No.: S-000600-0044-4
Houston, Texas
AEC Report No. G148-13

Dear Mr. Wallick,

Aviles Engineering Corporation (AEC) is pleased to present this report of our geotechnical investigation for the above referenced project. This investigation was authorized on September 9, 2013 by Mr. Paul Wallick, P.E., Senior Project Manager of IDS, based upon AEC Proposal No. G2013-08-01, dated August 2, 2013.

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

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

1-28-2014

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

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

Reports Submitted:	3	IDS Engineering Group
	1	File (electronic)

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TABLE OF CONTENTS

ATTACHMENTS

ENGINEERING CORP.

EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed 500,000 gallon ground water storage tank (GST) replacement at Park Glen West Pump Station, located at 10630 South Kirkwood in Houston, Texas (Houston/Harris County Key Map No.: 529W). Based on information provided by IDS Engineering Group (IDS), the existing GST at the site will be demolished and replaced with a new GST; the center of the new GST will be located approximately 5 feet to the south of the existing tank center. The proposed GST will be 24 foot tall steel tank with a diameter of 55 feet. A ring wall foundation will be used to support the new GST.

1. Subsurface Soil Conditions: Based on Borings B-1 and B-2, the subsurface conditions at the GST generally consist of approximately 2 feet of hard lean clay (CL) fill at the ground surface, underlain by approximately 25 to 28 feet of stiff to hard fat clay (CH), followed by approximately 5 feet of very stiff sandy lean clay (CL), then approximately 4 feet of clayey sand (SC), then approximately 12 feet of dense silty sand (SM), followed by approximately 7 feet of hard fat clay (CH) to the boring termination depths.

2. Subsurface Soil Properties: The cohesive soils encountered in our borings have Liquid Limits (LL) ranging from 44 to 67 and Plasticity Indices (PI) ranging from 31 to 46. This indicates that the cohesive soils have high to very high expansive potential. The cohesive soils encountered are classified as “CL” and “CH” type soils and the granular soils are classified as “SC” and “SM” type soils in accordance with ASTM D 2487.

3. Groundwater Conditions: Groundwater was encountered at a depth of 32 feet during drilling in Boring B-1, and subsequently rose to a depth of 22.3 feet approximately 15 minutes after the initial encounter. This indicates that the groundwater at the site could be pressurized. Groundwater was not encountered in Boring B-2.

4. Design parameters and recommendations for design and construction of the tank foundation and tank pad subgrade preparation are presented in Section 5.2 of this report.

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

AVILES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

GROUND STORAGE TANK NO.1 REPLACEMENT
PARK GLEN WEST PUMPING STATION
10630 SOUTH KIRKWOOD
WBS NO. S-000600-0044-3
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 Project Description

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed 500,000 gallon ground water storage tank (GST) replacement at Park Glen West Pump Station, located at 10630 South Kirkwood in Houston, Texas (Houston/Harris County Key Map No.: 529W). A vicinity map is presented on Plate 1 in the Attachments. Based on information provided by IDS Engineering Group (IDS), the existing GST at the site will be demolished and replaced with a new GST; the center of the new GST will be located approximately 5 feet to the south of the existing tank center. The proposed GST will be 24 foot tall steel tank with a diameter of 55 feet. A ring wall foundation will be used to support the new GST.

1.2 Purpose and Scope

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

1. Drilling and sampling two soil borings to depths ranging from 30 to 55 feet below existing grade;
2. Performing soil laboratory testing on selected soil samples;
3. Engineering analysis and recommendations for the GST foundation, allowable bearing capacity, tank settlement, and subgrade preparation; and
4. Construction recommendations for the GST foundation.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions were investigated by drilling two soil borings adjacent to the existing GST to depths ranging from 30 to 55 feet below existing grade. The boring locations are shown on the attached Boring Location Plan on Plate 2, in the Attachments. Boring survey data is included on the representative boring logs.

The borings were drilled using a truck-mounted drill rig. Borings were performed initially by dry auger method, then using wet rotary method once the borings caved in or saturated granular soils were encountered. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D 1587. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586. Standard Penetration Test resistance (N) values were recorded for the granular soils as "Blows per Foot" and are shown on the boring logs. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. The borings were backfilled with bentonite chips upon completion of drilling. Details of the soils encountered in the borings are presented on Plates 3 and 4, in the Attachments.

3.0 LABORATORY TESTING

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, and dry unit weight tests were performed on representative samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were estimated by means of Unconsolidated-Undrained (UU) triaxial tests performed on undisturbed samples. The test results are presented on their representative boring logs. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates 5 through 8, in the Attachments.

Two one-dimensional consolidation tests were performed on selected soil samples in order to evaluate the general compressibility characteristics of the clay soils at the proposed GST. The results of the consolidation tests are presented on Plates 9 and 10, in the Attachments. The initial void ratio, compression index, recompression index, preconsolidation pressure, and estimated overconsolidation ratio (OCR) for the consolidation tests are summarized in Table 1.

Table 1. Summary of Consolidation Test Results

Sample ID and Description	e_0	C_c	C_r	p_c (tsf)	OCR
B-1, 10'-12', Fat Clay (CH)	0.6988	0.2549	0.0400	5.0	7.6
B-1, 33'-35', Clayey Sand (SC)	0.5179	0.1212	0.0072	2.4	1.4

Note: (1) e_0 = initial void ratio;
(2) C_c = compression ratio;
(3) C_r = recompression ratio, which is derived from the recompression curve within the stress range from 1 to 4 tsf;
(4) p_c = preconsolidation pressure; and
(5) OCR = overconsolidation ratio.

4.0 SITE CONDITIONS

The site consists of an existing GST, pumps, and chlorinator control building.

4.1 Subsurface Conditions

Soil strata encountered in our borings are summarized below:

Boring	Depth	Description of Stratum
B-1	0' - 2'	Fill: hard, Lean Clay w/Sand (CL), with siltstone fragments, roots, and sand seams
	2' - 27'	Stiff to very stiff, Fat Clay (CH), with slickensides
	27' - 32'	Very stiff, Sandy Lean Clay (CL), with siltstone fragments
	32' - 36'	Clayey Sand (SC), wet
	36' - 48'	Dense, Silty Sand (SM), wet
	48' - 55'	Hard, Fat Clay (CH)
B-2	0' - 2'	Fill: hard, Lean Clay w/Sand (CL), with roots and sand seams
	2' - 30'	Stiff to hard, Fat Clay (CH), with slickensides

Details of the soils encountered during drilling are presented on the boring logs. The cohesive soils encountered in our borings have Liquid Limits (LL) ranging from 44 to 67 and Plasticity Indices (PI) ranging from 31 to 46. This indicates that the cohesive soils have high to very high expansive potential. The cohesive soils encountered are classified as "CL" and "CH" type soils and the granular soils are classified as "SC" and "SM" type soils in accordance with ASTM D 2487. "CH" soils can undergo significant volume changes due to seasonal changes in moisture contents. "CL" soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes. Slickensides were encountered in fat clay soil.

Groundwater was encountered at a depth of 32 feet during drilling in Boring B-1, and subsequently rose to a depth of 22.3 feet approximately 15 minutes after the initial encounter. This indicates that the groundwater at the site could be pressurized. Groundwater was not encountered in Boring B-2. The information in this report summarizes conditions found on the date the borings were drilled. However, it should be noted that our ground water observations are short term; ground water depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Subsurface Variations

It should be emphasized that: (i) at any given time, ground water depths can vary from location to location, and (ii) at any given location, ground water depths can change with time. Ground water depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary between and away from borings.

Clay soils in the Houston area typically have secondary features such as slickensides and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples which were generally obtained at intervals of 2 feet in the top 20 feet of the borings and at intervals of 5 feet thereafter to the boring termination depths. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

Based on information provided by IDS, the existing GST at the site will be demolished and replaced with a new GST; the center of the new GST will be located approximately 5 feet to the south of the existing tank center. The proposed GST will be 24 foot tall steel tank with a diameter of 55 feet. A ring wall foundation will be used to support the new GST.

Based on our borings, AEC recommends that the GST be supported on a ring wall foundation, founded at a minimum depth of 3 feet below existing grade.

5.1 Demolition of Existing Tank Foundation

AEC recommends that foundation of the existing GST be removed and backfilled with compacted select fill or lime-stabilized clay in accordance with Section 5.3 of this report. Loose soil or concrete still present within the foundation excavations shall be removed prior to backfilling. AEC recommends that an Owner's Representative be on site during demolition to ensure that all existing foundation is properly removed and backfilled.

5.2 500,000 Gallon Ground Storage Tank

5.2.1 Tank Ring Wall Foundation

A ring wall foundation at a depth of 3 feet below existing grade should be designed for an allowable net bearing capacity of 2,000 pounds per square foot (psf) for dead loads and 3,000 psf for total loads. A minimum factor of safety (FS) of 3 and 2 was applied for sustained loads and total loads, respectively; whichever bearing capacity is critical should be used for design.

Since the foundation will be subjected to hoop stresses, adequate reinforcement will be required to resist these forces. For the calculation of the lateral pressure on the ring wall foundation, we recommend that at-rest earth pressure be considered. The coefficient of earth pressure at-rest, $K_0 = 0.95$, can be used in the design. At-rest pressure, p_h (psf), at a depth of z ft below finished grade inside the ring wall can be calculated as:

$$P_h = (p_0 + \gamma z)^*K_0 \qquad \text{............Equation (1)}$$

where, p_0 = tank pressure at the finished grade elevation, psf;
γ = wet unit weight of soil, 125 pcf;
z = depth below finished grade, ft; and
K_0 = coefficient of earth pressure at-rest, 0.95

Foundation Settlements: AEC assumes that the foundation soils under the existing tank are fully consolidated; that is, foundation soil settlement due to the existing tank load has completed. We also assume that the new tank will provide the same load as the existing tank since the height and diameter of both tanks are the same. Any new settlements will be a result of additional loads on the foundation. However, since the new tank will be relocated 5 feet from the center of the existing tank, a portion of the new tank foundation will be supported on uncompressed/less compressed soils by the existing tank. For our analysis, we have calculated settlements based on our boring logs, soil laboratory testing results, and anticipated tank load. Considering a 24 foot high water head over a 55 foot diameter tank base, AEC estimated a tank pressure of approximately 1,500 psf. Based on the estimated tank pressure, AEC estimated total settlement (which includes both immediate and long-term settlement, respectively) at the center and edge of the tank. A summary of the tank settlements is presented on Table 2.

Table 2. 500,000 gallon GST Settlements (Based on Borings B-1 and B-2)

Tank	**Tank Height (ft)**	**δ_v (in)**	**S_{c1} (in)**	**S_{c2} (in)**	**Total S (in)**
Center	24	1.5	0.1	1.7	3.3
Edge	24	1.2	0.1	1.0	2.3

Note: (1) δ_v = immediate settlement, S_{c1} = Estimated settlement resulting from granular soils; S_{c2} = Estimated consolidation settlement resulting from clayey soils; Total settlement, $S = \delta_v + S_{c1} + S_{c2}$.

AEC notes that since the borings were performed outside the perimeter of the existing tank, and that a significant portion of the new tank footprint is located within the existing tank footprint, the actual settlement at the center of the new tank could be less than the amount presented in Table 2. However, there will still be settlement (i.e. settlement will *not* be negligible), because: (i) even though AEC's borings are located outside of the existing tank footprint, the borings are still close enough to the tank perimeter to have been partially influenced by existing tank load pressure below a certain depth below grade; and (ii) there will be rebound and recompression of the soils after the existing tank is demolished and the new tank is constructed and loaded. It is AEC's opinion that the amount of settlement presented in Table 2 is on the safe side but will not have a negative impact on the total construction cost of the tank. If the settlement of center of the tank is a concern, AEC recommends that an additional soil boring be performed at the center of the new tank after the existing tank is demolished to verify that the amount of estimated settlement is reasonable.

<u>Time Rate of Consolidation Settlement:</u> Time rates of foundation settlements are plotted as curves of percent total consolidation settlement versus time for the GST on Plate 11, in the Attachments. The curve is based on the assumption of a one-month linear construction period, i.e. the foundation soils will be loaded linearly during construction.

Frequently, the predicted settlement time is longer than that observed in the field for the following reasons: (1) theoretical conditions assumed for the consolidation analysis do not hold in-situ because of intermediate lateral drainage, anisotropy in permeability, time dependency of real loading, and the variation of soil properties with effective stress; and (2) the coefficient of consolidation, as determined in the laboratory, decreases with sample disturbance; therefore, predicted settlement time tends to be greater than actual settlement time.

5.2.2 Tank Pad Preparation

Subgrade Preparation: Demolition of the existing tank foundation should be in accordance with Section 5.1 of this report. Subgrade preparation should extend a minimum of 5 feet beyond the tank perimeter. A minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials shall be removed and wasted. The excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths.

Afterward surface stripping, an additional 1.5 feet (total depth of 2 feet, which includes the 6 inches of surface removal) of existing soils should be removed. The exposed subgrade should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill or clean stabilized soils.

After proof rolling, compacted select fill or clean, stabilized soils should then be used to achieve the finished floor elevation (FFE) of the tank. Select fill or stabilized soil should be in accordance with Section 5.3 of this report. We recommend that the final subgrade surface be crowned about 2 inches higher at the tank center than the edge, since the settlement at the tank center is typically higher than the tank edge.

5.3 Select Fill

Select fill should consist of uniform, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Excavated material delivered to the site for use as select fill shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Prior to construction, the Contractor should determine if he or she can obtain qualified select fill meeting the above select fill criteria.

As an alternative to imported fill, on-site soils excavated during construction can be stabilized with a minimum of 7 percent hydrated lime by dry soil weight. The percentage of lime stabilization is a preliminary estimate for planning purposes only; the amount of stabilization should be determined by lime-series curve or pH method in a laboratory prior to construction. Lime stabilization shall be performed in accordance with Section 02336 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS). AEC prefers using stabilized on-site clay as select fill since compacted lime-stabilized clay generally has high shear strength, low compressibility, and relatively low permeability. Blended or mixed soils (sand and clay) should not be used as select fill.

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 walls or columns should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors. The lime-stabilized onsite soils 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.

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

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation and Grading

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Construction Monitoring

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

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7.0 GENERAL

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

The information contained in this report summarizes conditions found on the date the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the date of drilling. Due to variations encountered in the subsurface conditions across the site, changes in soil conditions from those presented in this report should be anticipated. AEC should be notified immediately when conditions encountered during construction are significantly different from those presented in this report.

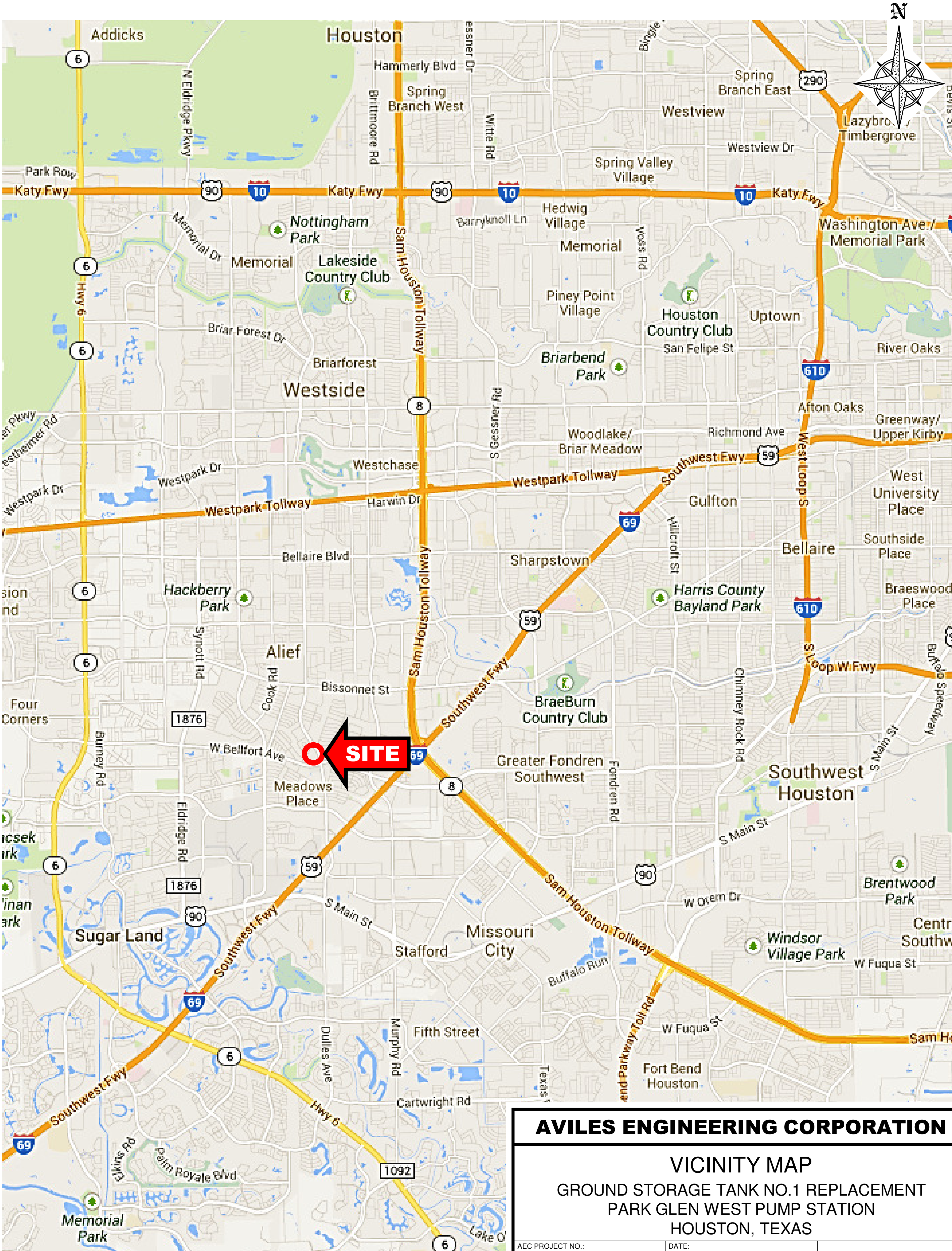
8.0 LIMITATIONS

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

ENGINEERING CORP.

ATTACHMENTS

Plate 1	Vicinity Map
Plate 2	Boring Location Plan
Plates 3 & 4	Boring Logs
Plate 5	Key to Symbols
Plate 6	Classification of Soils for Engineering Purposes
Plate 7	Terms Used on Boring Logs
Plate 8	ASTM & TXDOT Designation for Soil Laboratory Tests
Plates 9 & 10	Consolidation Test Results
Plate 11	Estimated Time Rate of Consolidation Settlement

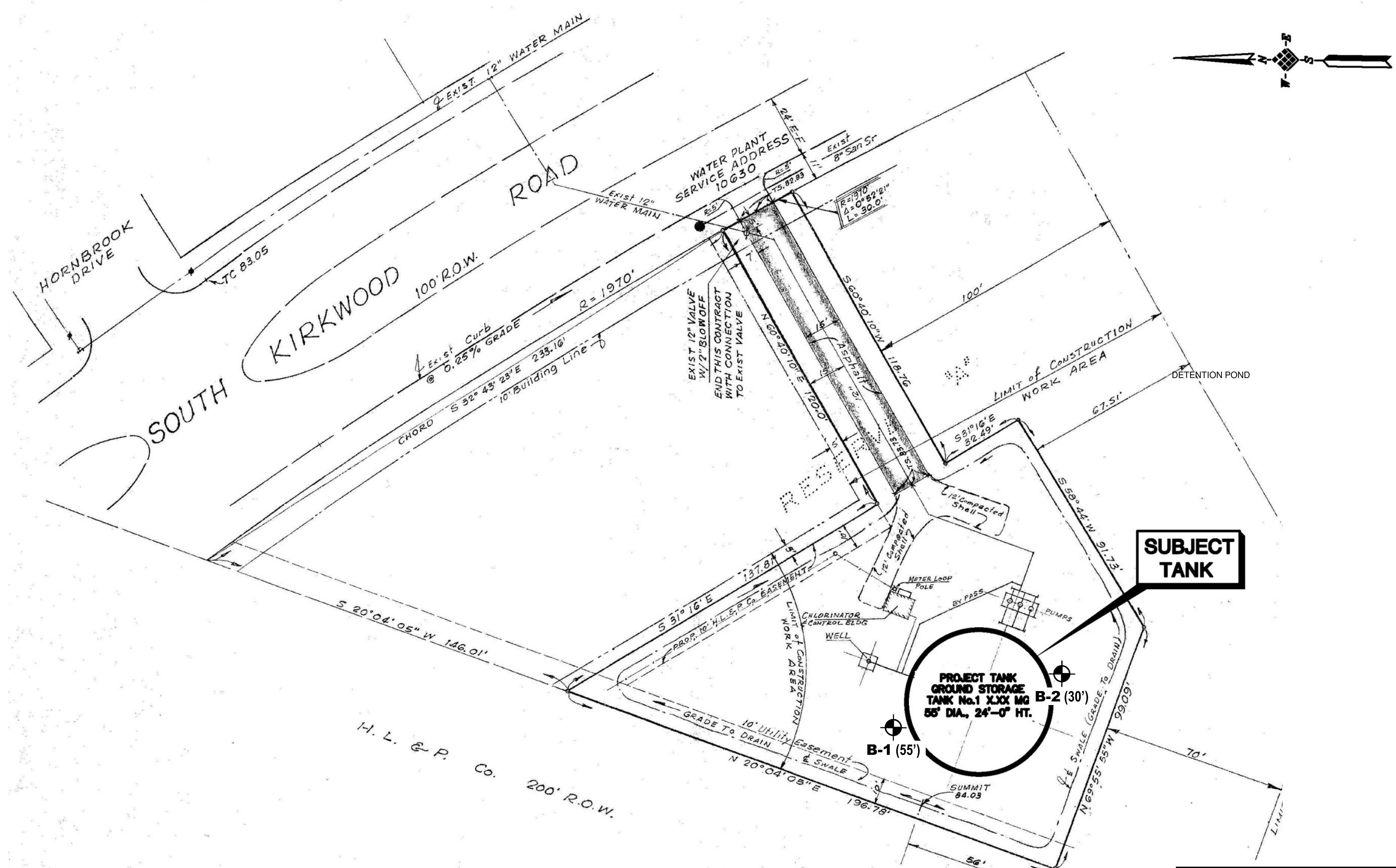


AVILES ENGINEERING CORPORATION

VICINITY MAP

GROUND STORAGE TANK NO.1 REPLACEMENT
PARK GLEN WEST PUMP STATION
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	
G148-13	09-26-13	
APPROX. SCALE: N.T.S.	DRAFTED BY: BpJ	PLATE NO.: PLATE 1



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

GROUND STORAGE TANK NO. 1 REPLACEMENT
PARK GLEN WEST PUMP STATION
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G148-13	09-26-13	IDS ENGINEERING
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
1" = 40'	BpJ	PLATE 2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

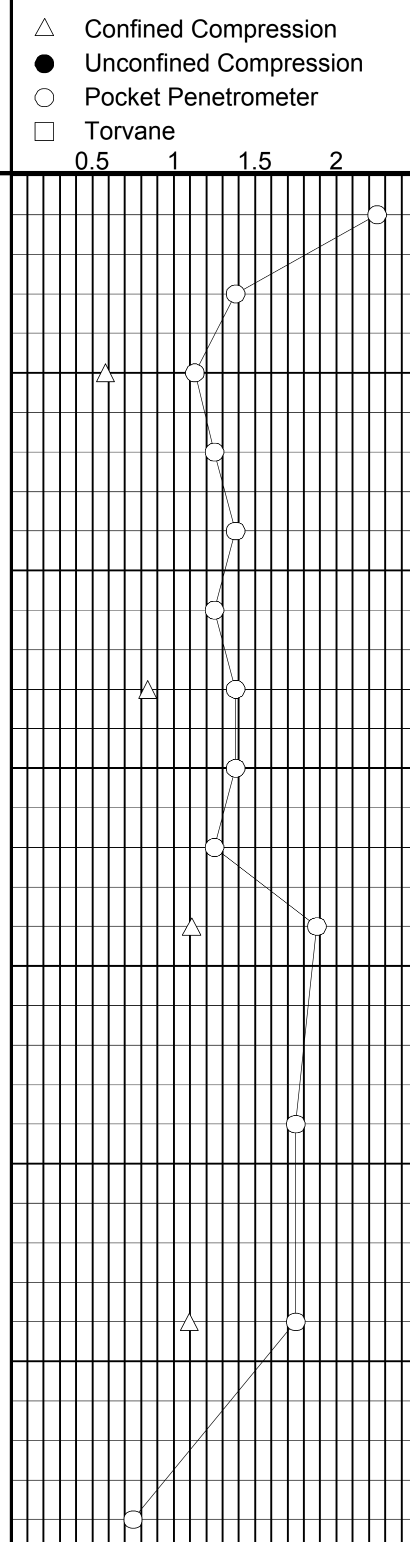
PROJECT: **GST No.1 Replacement at Park Glen West** BORING **B-1**

DATE **8/26/13** TYPE **4" Dry Auger/Wet Rotary** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: *3053229.3197*
Northing: *13805283.6014*
Elevation: *80.52*

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: hard, dark gray Lean Clay w/Sand (CL), with siltstone fragments, calcareous nodules, roots, and sand seams		10		80	44	13	31
	Stiff to very stiff, dark gray Fat Clay (CH), with slickensides		22					
5	-with ferrous nodules 2'-6' -gray and tan 4'-8'		25	101				
			24					
	-brown, light gray, and tan		26					
10	-red, brown, and light gray 10'-20'		25	101	95	58	20	38
			29	95				
15			26					
			26		100	64	18	46
	-with silt pockets 18'-20'		21	103				
20								
25	-brown and tan 23'-25'		23					
	Very stiff, light gray and tan Sandy Lean Clay (CL), with abundant calcareous and ferrous nodules, and siltstone fragments		17	111				
30								
	Tan and red Clayey Sand (SC), with calcareous nodules, wet		16	113	25	28	13	15

SHEAR STRENGTH, TSF



BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **32** FEET WHILE DRILLING
WATER LEVEL AT **22.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **AEC**

ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **GST No.1 Replacement at Park Glen West** BORING **B-1**

DATE **8/26/13** 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	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
35–48	Dense, tan and light gray Silty Sand (SM), wet							
39		40	21					
44		42	18		14			
48–55	Hard, tan and light gray Fat Clay (CH)							
49			29					
54			31					
55	Termination depth = 55 feet.							

BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **32** FEET WHILE DRILLING
WATER LEVEL AT **22.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **AEC**

PROJECT: GST No.1 Replacement at Park Glen West

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

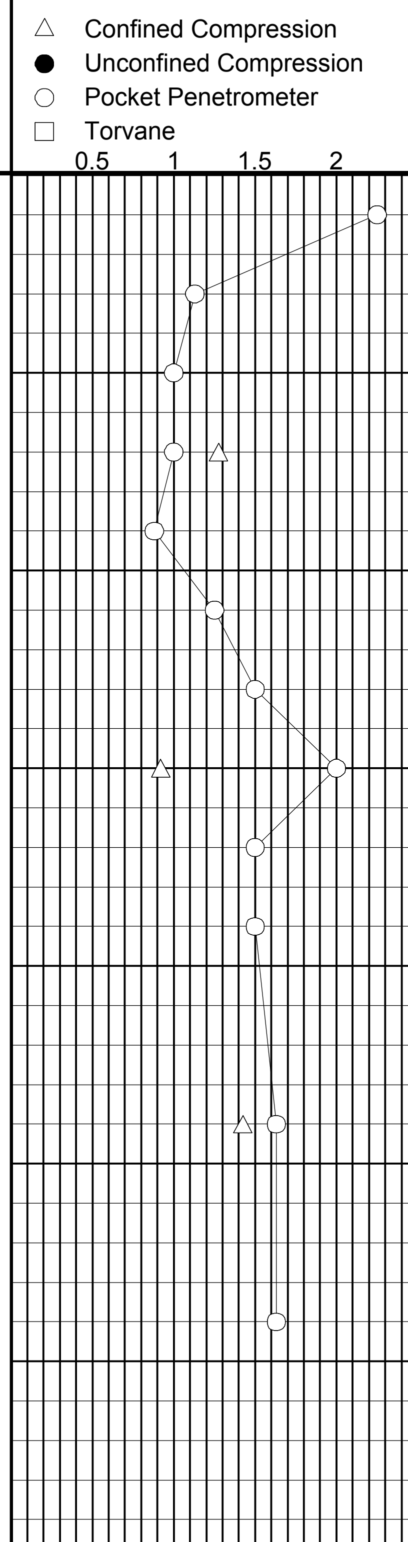
BORING B-2

DATE 8/26/13 **TYPE** 4" Dry Auger **LOCATION** See Boring Location Plan

Survey Coordinates (ft):
Easting: 3053245.8948
Northing: 13805194.3227
Elevation: 80.26

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0–2	Fill: hard, dark gray Lean Clay w/Sand (CL), with calcareous and ferrous nodules, roots, and sand seams		10		74	49	13	36
2–30	Stiff to hard, dark gray and tan Fat Clay (CH), with slickensides -with ferrous nodules 2'-4', and calcareous nodules 2'-12' -tan, with siltstone fragments 4'-6' -red, brown, and light gray 6'-25', with silty clay pockets 6'-8'		20					
5			18					
			20	113				
					98	53	16	37
			23					
			26					
			27					
15			23	102				
					100	67	21	46
			27					
			26					
	-with silt pockets 23'-25'		24	103				
	-light gray and red, with sand seams, siltstone fragments, and calcareous and ferrous nodules		17					
30	Termination depth = 30 feet.							

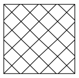
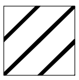
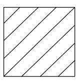
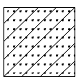
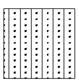
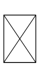
SHEAR STRENGTH, TSF



BORING DRILLED TO 30 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT N/A FEET WHILE DRILLING
WATER LEVEL AT N/A FEET AFTER COMPLETE
DRILLED BY V&S CHECKED BY CHL LOGGED BY AEC

PROJECT NO. G148-13 **PLATE 4**

KEY TO SYMBOLS

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

AYLES ENGINEERING CORP.

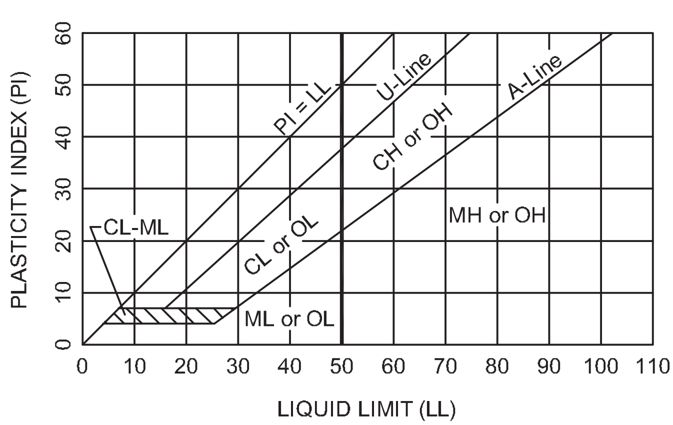
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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

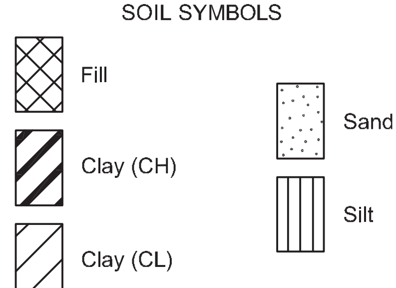


PLATE 6

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

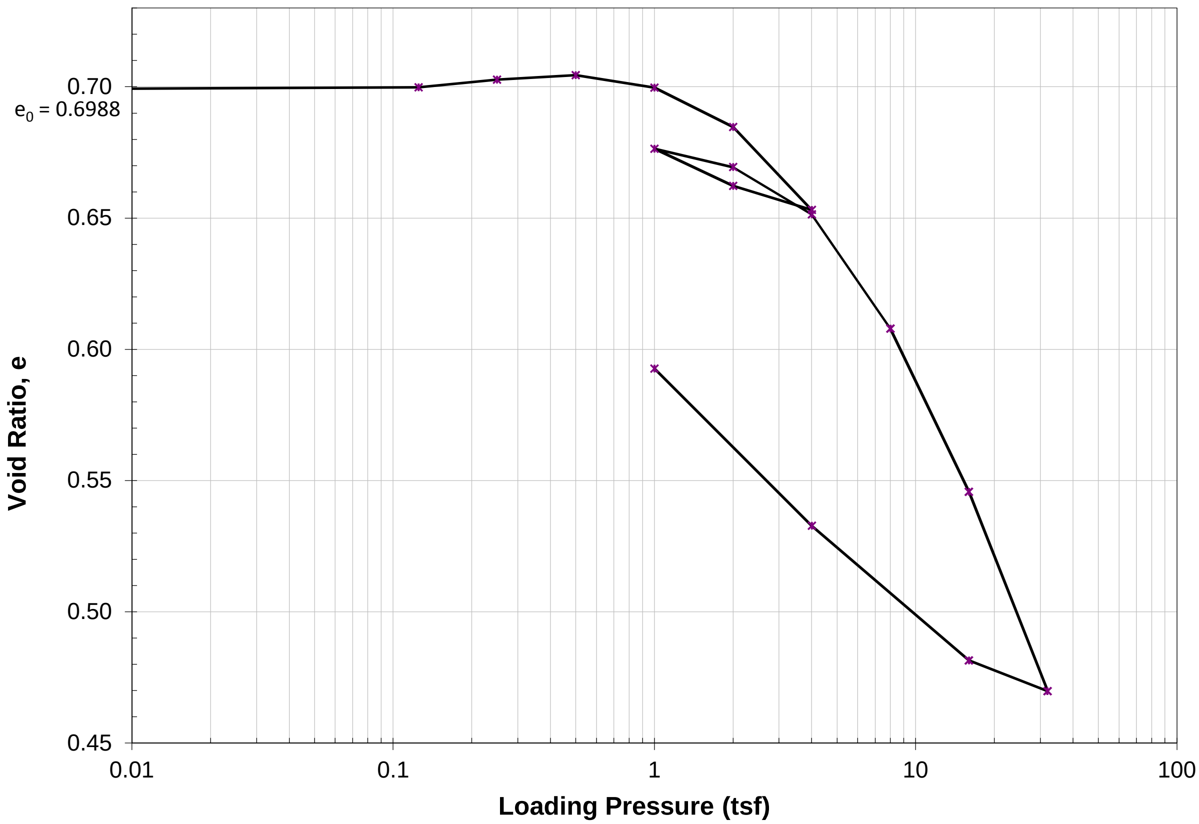
Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
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.

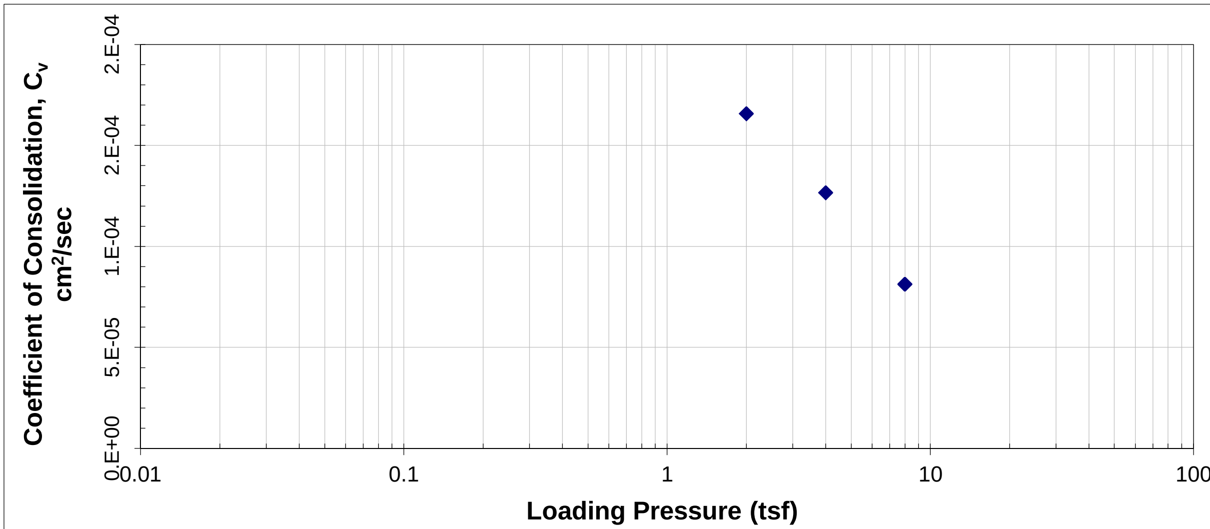
ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

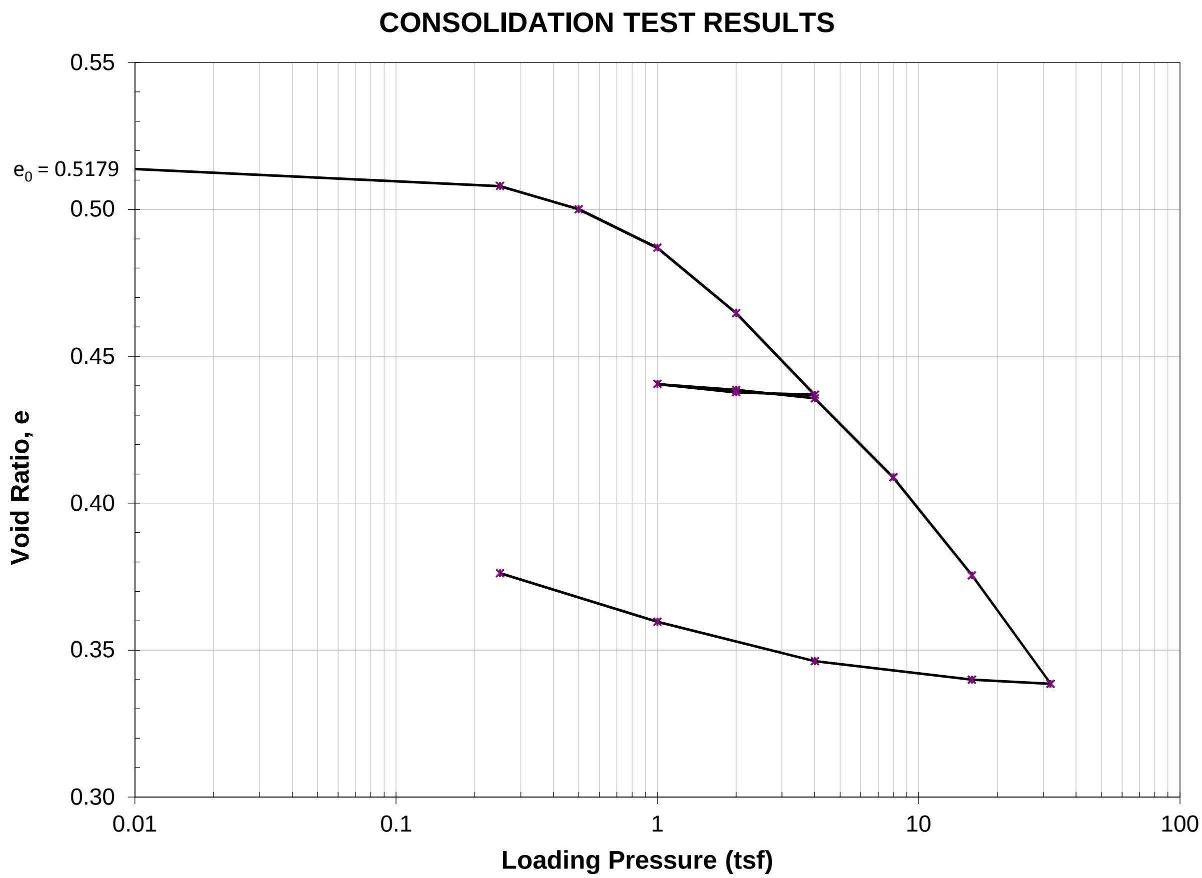
NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	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

CONSOLIDATION TEST RESULTS



Project No.: G148-13
Project: GST No.1 Replacement at Park Glen West
Sample ID: B-1, 10 to 12 feet
Dry Unit Weight (γ_d): 101 pcf
Sample Description: Tan and red Fat Clay (CH)
Estimated Consolidation Index (Cc): 0.2549
Estimated Recompression Index (Cr): 0.0400
Estimated OCR: 7.6
Estimated Preconsolidation Pressure (Pc): 5.0 tsf





Project No.: G148-13

Project: GST No.1 Replacement at Park Glen West

Sample ID: B-1, 33 to 35 feet

Dry Unit Weight (γ_d): 113 pcf

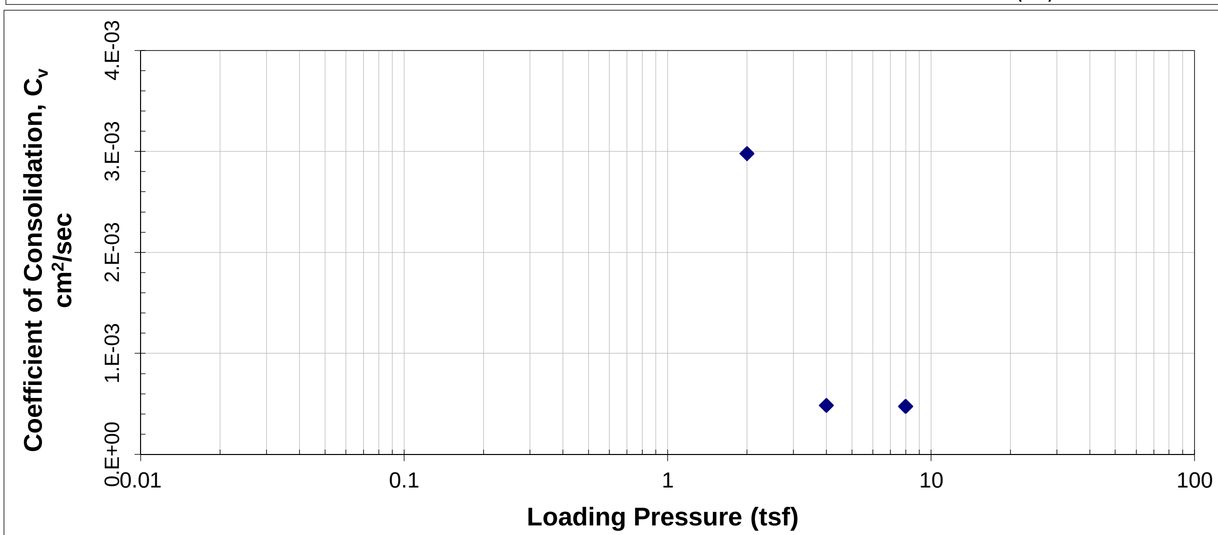
Sample Description: Tan and red Clayey Sand (SC)

Estimated Consolidation Index (Cc): 0.1212

Estimated Recompression Index (Cr): 0.0072

Estimated OCR: 1.4

Estimated Preconsolidation Pressure (Pc): 2.4 tsf



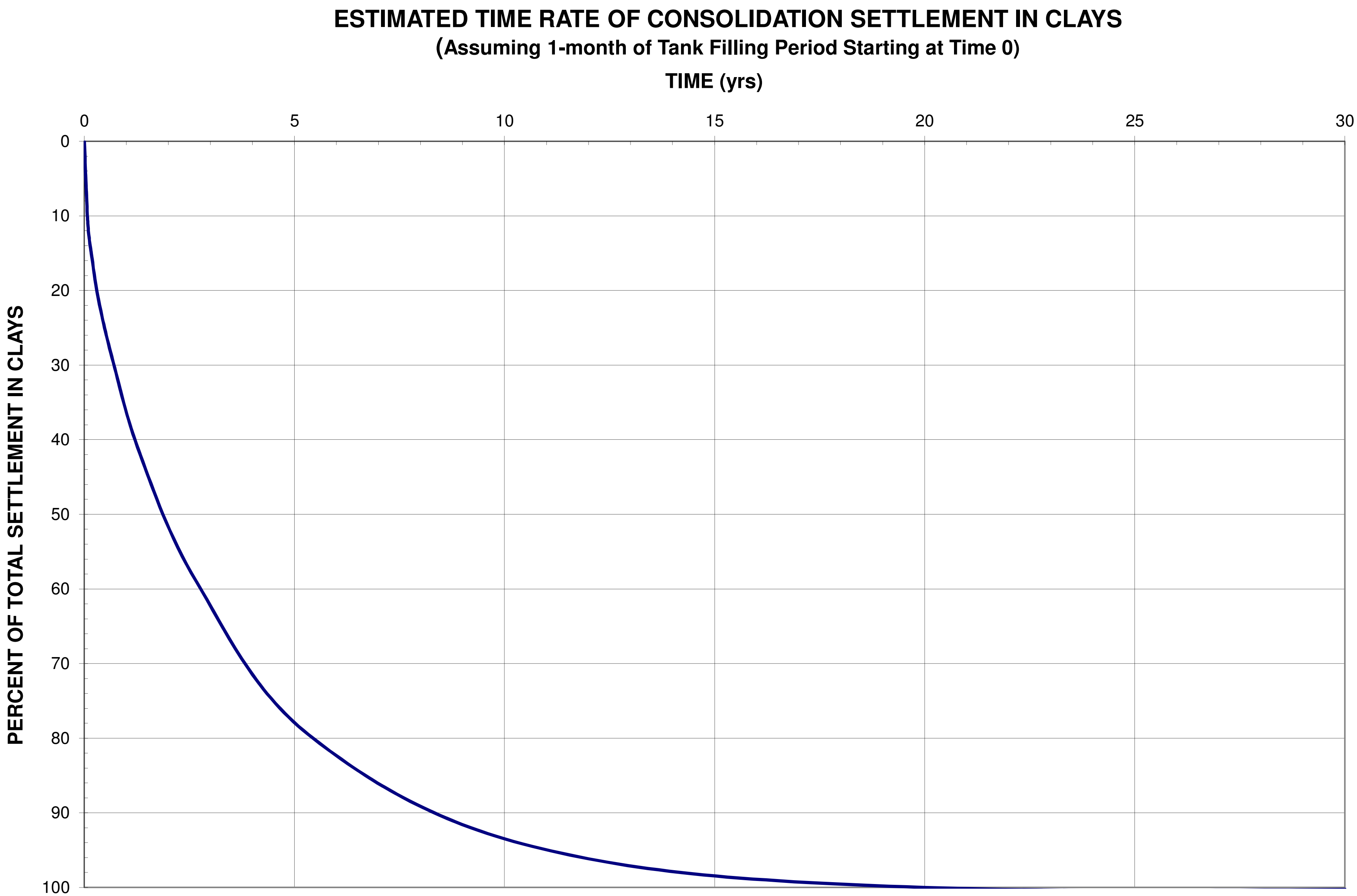

ESTIMATED TIME RATE OF CONSOLIDATION SETTLEMENT IN CLAYS
(Assuming 1-month of Tank Filling Period Starting at Time 0)
TIME (yrs)
0
5
10
15
20
25
30
PERCENT OF TOTAL SETTLEMENT IN CLAYS
0
10
20
30
40
50
60
70
80
90
100

PLATE 11