ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

GILLETTE TRUNKLINE (GENESEE SEGMENT)
DRAINAGE AND PAVING IMPROVEMENTS
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

Reported to:
HR Green, Inc.
Houston, Texas

by

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

REPORT NO. G166-12B - R1

September 2014

5790 Windfern Road
Houston, Texas 77041
Tel: (713)-895-7645
Fax: (713)-895-7943

September 18, 2014

Ms. Celeste Jain, P.E.
Project Engineer
HR Green, Inc.
11011 Richmond Avenue, Suite 375
Houston, Texas 77042

Reference: **Revised Geotechnical Investigation**
City of Houston Gillette Trunkline (Genesee Segment)
Drainage and Paving Improvements
Houston, Texas
AEC Report No. G166-12B-R1

Dear Ms. Jain,

Aviles Engineering Corporation (AEC) is pleased to present this report of the results of our revised geotechnical investigation for the above referenced project. Notice to proceed for the geotechnical investigation was provided via email on December 4, 2012 by Mr. Stephen Sparks, P.E., of HR Green, based on AECs proposal G2012-06-08R3, dated October 26, 2012. The contents of this report supersede AEC's previous report for this project, AEC Report G166-12B, dated July 9, 2014.

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

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

9/18/2014

STATE OF TEXAS
WILBER L. WANG
99226
LICENSED
PROFESSIONAL ENGINEER

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

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

Reports Submitted: 3 HR Green, Inc.
1 File (electronic)

Z:\ENGINEERING\REPORTS\2012\166-12 COH MONTROSE AREA & MIDTOWN STORM SEWER IMPROVEMENTS - HR GREEN, INC. (WILBER)\G166-12 SEGMENT B FINAL.DOCX

AVILES ENGINEERING CORP.

TABLE OF CONTENTS

AVILES ENGINEERING CORP.

APPENDICES

APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 thru A-10	Boring Logs
Plate A-11	Key to Symbols
Plate A-12	Classification of Soils for Engineering Purposes
Plate A-13	Terms Used on Boring Logs
Plate A-14	ASTM & TXDOT Designation for Soil Laboratory Tests
Plates A-15 thru A-18	Summary of Lab Data

APPENDIX B

Plate B-1	Generalized Soil Profile
Plates B-2 and B-3	Piezometer Installation Details

APPENDIX C

Plates C-1a thru C-1c	Recommended Geotechnical Design Parameters
Plate C-2	Load Coefficients for Pipe Loading
Plate C-3	Live Loads on Pipe Crossing Under Roadway

APPENDIX D

Plate D-1	Critical Heights of Cuts in Nonfissured Clays
Plate D-2	Maximum Allowable Slopes
Plate D-3	A Combination of Bracing and Open Cuts
Plate D-4	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Long Term Conditions
Plate D-5	Lateral Pressure Diagrams for Open Cuts in Cohesive Soil-Short Term Conditions
Plate D-6	Lateral Pressure Diagrams for Open Cuts in Sand
Plate D-7	Bottom Stability for Braced Excavation in Clay
Plate D-8	Buoyant Uplift Resistance for Buried Structures

APPENDIX E

Plates E-1 and E-2	Well Plugging Reports

APPENDIX F

Plates F-1 thru F-3	DARWin v3.0 Analysis Results

EXECUTIVE SUMMARY

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the City of Houston's (COH) proposed Gilette Trunkline (Genesee Segment) Drainage and Paving Improvements - Design Package B, in Houston, Texas (Houston Key Map 493P). A vicinity map is presented on Plate A-1, in Appendix A. According to HR Green, the project alignment starts at the intersection of West Dallas Street and Genesee Street, proceeds south along Genesse Street to the intersection with Tuam Street, then proceeds southeast along Tuam and terminates at the intersection of Tuam Street with Helena Street. The proposed improvements include: (i) installation of 8 foot by 8 foot and 10 foot by 10 foot concrete box storm sewers by open cut method; (ii) installation of storm sewer manholes and junction boxes; and (iii) reconstruction of existing roadway pavement with new concrete pavement. Based on drawings provided by HR Green, the invert depth of the storm sewers along the alignment varies from 20.5 to 27.5 feet. The contents of this report supersede AEC's previous report for this project, AEC Report G166-12B, dated July 9, 2014.

1. Subsurface Soil Conditions: A generalized subsurface profile along the storm sewer alignment is presented on Plate B-1, in Appendix B. Based on Borings B-5 through B-12, subsurface soil conditions along the project alignment generally consist of approximately 33 to 45 feet of firm to hard fat/lean clay (CH/CL), underlain by 8 to 12 feet of dense to very dense silty sand/silt (SM/SP-SM/ML) to the boring termination depths.

2. Subsurface Soil Properties: The subsurface clayey soils have low to very high plasticity, with liquid limits (LL) ranging from 24 to 82, and plasticity indices (PI) ranging from 6 to 52. The cohesive soils encountered are classified as "CL-ML", "CL", and "CH" type soils and granular soils were classified as "SP-SM", "SM", "SC", and "ML" in accordance with ASTM D 2487.

3. Groundwater Conditions: Groundwater was encountered at a depth of 20 to 43 feet below grade during drilling and was subsequently observed at a depth of 16.2 to 35.7 feet drilling was complete. Groundwater along the alignment may be pressurized. After completion of drilling, Borings B-6 and B-11 were converted to piezometers. A detailed description of ground water readings is presented on Table 3 in Section 4.1 of this report.

4. Hazardous Materials: No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

5. Design parameters and recommendations for installation of storm sewers by open cut method are presented in Sections 5.1 and 5.2 of this report.

6. Design parameters and recommendations for installation of manholes and junction boxes by open cut method are presented in Section 5.3 of this report.

7. Design parameters and recommendations for concrete pavement are presented in Section 5.4 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.

GEOTECHNICAL INVESTIGATION

GILLETTE TRUNKLINE (GENESEE SEGMENT)
DRAINAGE AND PAVING IMPROVEMENTS
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 General

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the City of Houston's (COH) proposed Gilette Trunkline (Genesee Segment) Drainage and Paving Improvements - Design Package B, in Houston, Texas (Houston Key Map 493P). A vicinity map is presented on Plate A-1, in Appendix A. According to HR Green, the project alignment starts at the intersection of West Dallas Street and Genesee Street, proceeds south along Genesse Street to the intersection with Tuam Street, then proceeds southeast along Tuam and terminates at the intersection of Tuam Street with Helena Street. The proposed improvements include: (i) installation of 8 foot by 8 foot and 10 foot by 10 foot concrete box storm sewers by open cut method; (ii) installation of storm sewer manholes and junction boxes; and (iii) reconstruction of existing roadway pavement with new concrete pavement. Based on drawings provided by HR Green, the invert depth of the storm sewers along the alignment varies from 20.5 to 27.5 feet. The contents of this report supersede AEC's previous report for this project, AEC Report G166-12B, dated July 9, 2014.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil conditions along the alignment and develop geotechnical engineering recommendations for design and construction of storm sewers by open cut method, as well as street reconstruction, including pavement thickness and subgrade preparation. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling eight geotechnical borings, ranging from 35 to 50 feet below existing grade;
2. Soil laboratory testing on selected soil samples;
3. Engineering analyses and recommendations for the installation of storm sewers, manholes, and junction boxes by open cut method, including loadings on pipes, bedding, lateral earth pressure parameters, trench stability, and backfill requirements;

4. Engineering analyses and recommendations for the design of rigid pavement, including pavement thickness and subgrade preparation;
5. Construction recommendations for installation of storm sewers, manholes, and junction boxes by open cut method, as well as rigid pavements.

2.0 SUBSURFACE EXPLORATION

2.1 Soil Borings

The boring layout and depths were selected by AEC in general accordance with Chapter 11 of the 2011 COH Infrastructure Design Manual (IDM), based on preliminary information provided by HR Green on August 27, 2012. The subsurface exploration consisted of drilling and sampling a total of eight soil borings (Borings B-5 through B-12) ranging from 35 to 50 feet below existing grade. Borings B-1 through B-4 were performed for the connecting Montrose Area and Midtown Drainage and Pavement Sub-project II, WBS No. M-000290-0002-3. The boring locations are shown on the Boring Location Plan on Plate A-2, in Appendix A. Total drilling footage is 345 feet. Boring survey data was provided to AEC and is included on the boring logs. The boring designations and depths and corresponding storm sewer invert depths are presented in Table 1 below.

Table 1. Boring Number, Station, and Depth

Boring No.	Boring Depth (ft)	Station No./Alignment	Invert Depth near Boring (ft)	Piezometer Depth (ft)
B-5	40	32+93.08/Genesee	27.5	-
B-6 (PZ-2)	50	27+58.21/Genesee	27.5	30
B-7	45	22+67.60/Genesee	26.5	-
B-8	40	18+19.31/Genesee	25	-
B-9	45	12+45.46/Genesee	24	-
B-10	45	7+43.12/Genesee	23.5	-
B-11 (PZ-3)	45	2+78.77/Genesee	23	30
B-12	35	6+42.83/Tuam	20.5	-

Existing pavement at the borings was first cut with a core barrel prior to field drilling. The field drilling was performed with a truck-mounted drilling rig primarily using dry auger method, and then using wet rotary method once water-bearing granular soils were encountered or the borings began to cave in.

Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in general 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. Borings B-6 and B-11 were converted to piezometers upon completion of drilling. Borings B-5, B-7 through B-10, and B-12 were grouted with cement-bentonite upon completion of drilling and the existing pavement was patched with asphalt.

3.0 LABORATORY TESTING PROGRAM

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under the 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 typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of undrained-unconsolidated (UU) triaxial tests performed on undisturbed samples. The test results are presented on the boring logs. Details of the soils encountered in the borings are presented on Plates A-3 through A-10, in Appendix A. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-11 through A-14, in Appendix A. A summary of the lab data is presented on Plates A-15 through A-18, in Appendix A.

4.0 SITE CONDITIONS

Based on our site visit, Genesee Street between West Dallas Street and West Gray Street is a one-way roadway and between West Gray Street and Tuam Street is a narrow two lane (one lane in each direction) roadway; based on our borings, the roadway is a combination of asphalt pavement and concrete pavement.

Tuam Street between Genesee Street and Helena Street is a four lane (two lanes in each direction) roadway, and is asphalt pavment. In general, the pavement surface along both Genesee Street and Tuam Street is in poor condition, with numerous cracks and surface depressions. A summary of pavement types encountered in our borings is presented on Table 2.

Table 2. Existing Pavement Encountered at Pavement Borings

Boring No.	Street	Pavement Section
B-5	Genesee	4” asphalt, 10” sand and shell
B-6	Genesee	8” concrete, 8” stabilized clay and shell
B-7	Genesee	8.5” concrete, 7.5” stabilized sand and shell
B-8	Genesee	1” asphalt, 10” stabilized sand and shell
B-9	Genesee	2” asphalt, 9” stabilized sand and shell
B-10	Genesee	5” asphalt, 7” sand and gravel
B-11	Genesee/Tuam	2” asphalt, 9” stabilized sand and shell
B-12	Tuam	5.5” asphalt, 12.5” sand and gravel

4.1 Subsurface Conditions

A generalized subsurface profile along the storm sewer alignment is presented on Plate B-1, in Appendix B. Soil strata encountered in our borings are summarized below:

Boring	Depth (ft)	Description of Stratum
B-5	0 - 0.3	Pavement: 4” asphalt
	0.3 - 1.2	Base: 10” sand and shell
	1.2 - 6	Stiff to very stiff, Fat Clay w/Sand (CH)
	6 - 14	Very stiff to hard, Lean Clay (CL)
	14 - 16	Clayey Sand (SC)
	16 - 40	Stiff to hard, Fat Clay (CH), with slickensides
B-6	0 - 0.7	Pavement: 8” concrete
	0.7 - 1.3	Base: 8” stabilized clay and shell
	1.3 - 2	Fill: Fat Clay (CH), with roots
	2 - 6	Very stiff, Fat Clay (CH), with slickensides
	6 - 8	Very stiff, Lean Clay w/Sand (CL), with silt seams and siltstone fragments
	8 - 10	Very stiff, Fat Clay (CH), with siltstone fragments
	10 - 18	Very stiff to hard, Lean Clay (CL)
	18 - 22	Very stiff to hard, Fat Clay (CH), with sand pockets
	22 - 27	Very stiff to hard, Sandy Lean Clay (CL)
	27 - 42	Stiff to hard, Fat Clay (CH), with slickensides

Boring	Depth (ft)	Description of Stratum
B-6 (cont.)	42 - 50	Dense, Silt (ML), with clay seams
B-7	0 - 0.7	Pavement: 8.5" concrete
	0.7 - 1.3	Base: 7.5" stabilized clay and gravel
	1.3 - 2	Fill: Fat Clay (CH)
	2 - 10	Stiff to very stiff, Fat Clay (CH)
	10 - 16	Very stiff to hard, Lean Clay w/Sand (CL)
	16 - 22	Stiff to very stiff, Fat Clay (CH), with sand pockets
	22 - 27	Hard, Lean Clay w/Sand (CL), with sand pockets
	27 - 45	Very stiff to hard, Fat Clay (CH), with slickensides
B-8	0 - 0.1	Pavement: 1" asphalt
	0.1 - 0.9	Base: 10" stabilized sand and shell
	0.9 - 8	Stiff to very stiff, Fat Clay (CH)
	8 - 26	Very stiff to hard, Lean Clay (CL)
	26 - 37	Stiff to hard, Lean Clay w/Sand (CL)
	37 - 40	Hard, Silty Clay (CL-ML), with siltstone fragments
B-9	0 - 0.2	Pavement: 2" asphalt
	0.1 - 0.9	Base: 9" stabilized sand and shell
	0.9 - 8	Stiff to very stiff, Fat Clay (CH), with slickensides
	8 - 18	Stiff to very stiff, Lean Clay w/Sand (CL)
	18 - 33	Very stiff to hard, Fat Clay w/Sand (CH), with slickensides
	33 - 45	Dense to very dense, Poorly Graded Sand w/Silt (SP-SM)
B-10	0 - 0.4	Pavement: 5" asphalt
	0.4 - 1	Base: 7" sand and gravel
	1 - 6	Fill: firm to very stiff, Fat Clay (CH)
	6 - 8	Very stiff, Fat Clay (CH), with siltstone fragments
	8 - 18	Very stiff to hard, Sandy Lean Clay (CL)
	18 - 33	Stiff to hard, Lean Clay w/Sand (CL)
	33 - 35	Sandy Lean Clay (CL), with abundant sand seams
	35 - 45	Dense, Silty Sand (SM)
B-11	0 - 0.2	Pavement: 2" asphalt
	0.1 - 0.9	Base: 9" stabilized sand and shell
	0.9 - 10	Firm to very stiff, Fat Clay (CH), with slickensides
	10 - 16	Very stiff to hard, Sandy Lean Clay (CL)
	16 - 22	Stiff to very stiff, Fat Clay (CH), with slickensides
	22 - 33	Very stiff to hard, Lean Clay (CL), with sand partings
	33 - 36	Sandy Lean Clay (CL), with abundant sand seams
	36 - 45	Very dense, Silty Sand (SM)
B-12	0 - 0.4	Pavement: 5.5" asphalt
	0.4 - 1.5	Base: 12.5" sand and gravel
	1.5 - 10	Stiff to very stiff, Fat Clay (CH)
	10 - 23	Very stiff to hard, Lean Clay w/Sand (CL)

ENGINEERING CORP.

Boring	Depth (ft)	Description of Stratum
B-12 (cont.)	23 - 35	Stiff to hard, Lean Clay (CL)

Subsurface Soil Properties: The subsurface clayey soils have low to very high plasticity, with liquid limits (LL) ranging from 24 to 82, and plasticity indices (PI) ranging from 6 to 53. The cohesive soils encountered are classified as "CL-ML", "CL", and "CH" type soils and granular soils were classified as "SP-SM", "SM", "SC", and "ML" in accordance with ASTM D 2487. High plasticity clays can undergo significant volume changes due to seasonal changes in moisture contents. "CH" soils undergo significant volume changes due to seasonal changes in soil moisture contents. "CL" type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes. Slickensides were encountered in the fat clays.

Groundwater Conditions: Groundwater was encountered at a depth of 20 to 43 feet below grade during drilling and was subsequently observed at a depth of 16.2 to 35.7 feet drilling was complete. Groundwater along the alignment may be pressurized. After completion of drilling, Borings B-6 and B-11 were converted to piezometers. Piezometer installation details are presented on Plates B-2 and B-3, in Appendix B. Detailed groundwater levels are summarized in Table 3. Piezometer plugging reports are presented in Appendix E.

Table 3. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth Encountered during Drilling (ft)	Groundwater Depth 15 min. After Drilling Completion (ft)	Groundwater Depth in Piezometer (ft)
B-5	1/18/13	40	20	16.2	-
B-6 (PZ-2)	1/18/13	50	43	31.1	26.8 (1/24/13) 28.7 (2/21/13)
B-7	1/21/13	45	Dry	33.7	-
B-8	1/21/13	40	Dry	35.7	-
B-9	1/21/13	45	33	29.3	-
B-10	1/22/13	45	33	29.8	-
B-11 (PZ-3)	1/22/13	45	35	29.3	27.7 (1/24/13) 24.7 (2/21/13)
B-12	1/22/13	35	27	28.3	-

The information in this report summarizes conditions found on the dates the borings were drilled. It should be noted that our groundwater observations are short-term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Hazardous Materials

No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

4.3 Subsurface Variations

It should be emphasized that: (i) at any given time, groundwater depths can vary from location to location, and (ii) at any given location, groundwater depths can change with time. Groundwater depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary away from and in between the boring locations.

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 continuously at intervals of 2 from the ground surface to a depth of 20 feet in the borings, then at intervals of 5 feet thereafter to the boring termination depths of 35 to 50 feet. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while a boring log shows some soil secondary features, it should not be assumed that the features are absent where not indicated on the boring logs.

5.0 <u>GEOTECHNICAL ENGINEERING RECOMMENDATIONS</u>

Based on drawings provided by HR Green, the proposed improvements include: (i) installation of 8 foot by 8 foot and 10 foot by 10 foot concrete box storm sewers by open cut method; (ii) installation of storm sewer manholes and junction boxes; and (iii) reconstruction of existing roadway pavement with new concrete pavement. The invert depth of the storm sewers along the alignment varies from 20.5 to 27.5 feet.

AVILES ENGINEERING CORP.

5.1 Geotechnical Parameters for Underground Utilities

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

5.2 Installation of Storm Sewers by Open-Cut Method

Storm sewers installed by open-cut methods should be designed and installed in accordance with Section 02317 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS).

5.2.1 <u>Loadings on Pipes</u>

Underground utilities support the weight of the soil and water above the crown, as well as roadway traffic and any structures that exist above the utilities.

<u>Earth Loads:</u> For underground utilities to be installed using open cut methods, the vertical soil load W_e can be calculated as the larger of the two values from Equations (1) and (3):

$$W_e = C_d \gamma B_d^2 \quad \text{............Equation (1)}$$

$$C_d = [1- e^{-2K\mu'(H/B_d)}]/(2K\mu') \quad \text{............Equation (2)}$$

$$W_e = \gamma B_c H \quad \text{............Equation (3)}$$

where: W_e = trench fill load, in pounds per linear foot (lb/ft);
C_d = trench load coefficient, see Plate C-2, in Appendix C;
γ = effective unit weight of soil over the conduit, in pounds per cubic foot (pcf);
B_d = trench width at top of the conduit < 1.5 B_c (ft);
B_c = outside diameter of the conduit (ft);
H = variable height of fill (ft);
when the height of fill above the top of the conduit H_c >2 B_d, H = H_h (height of fill above the middle of the conduit). When H_c < 2 B_d, H varies over the height of the conduit; and
$K\mu'$ = 0.1650 maximum for sand and gravel,
0.1500 maximum for saturated top soil,
0.1300 maximum for ordinary clay,

0.1100 maximum for saturated clay.

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

Traffic Loads: The vertical stress on top of an underground conduit, p_L (psf), resulting from traffic loads (from a H-20 or HS-20 truck) can be obtained from Plate C-3, in Appendix C. The live load on top of the underground conduit can be calculated from Equation (4):

$$W_L = p_L B_c \qquad \text{............Equation (4)}$$

where:
- W_L = live load on the top of the conduit (lb/ft);
- p_L = vertical stress (on the top of the conduit) resulting from traffic loads (psf);
- B_c = outside diameter of the conduit, (ft);

Lateral Loads: The lateral soil pressure p_l can be calculated from Equation (5); hydrostatic pressure should be added, if applicable.

$$p_l = 0.5\,(\gamma H_h + p_s) \qquad \text{............Equation (5)}$$

where:
- H_h = height of fill above the center of the conduit (ft);
- γ = effective unit weight of soil over the conduit (pcf);
- p_s = vertical pressure on conduit resulting from traffic and/or construction equipment (psf).

5.2.2 Trench Stability

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

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

Trenches 20 feet and Deeper: **OSHA requires that shoring or bracing for trenches 20 feet and deeper be specifically designed by a licensed professional engineer.**

Trenches Less than 20 Feet Deep: Trench excavations that are less than 20 feet deep may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, the general public, and adjacent structures, except for excavations which are less than 5 feet deep and verified by a competent person to have no cave-in potential. The excavation and trenching should be in accordance with Occupational Safety and Health Administration (OSHA), Safety and Health Regulations, 29 CFR, Part 1926. Recommended OSHA soil types for trench design for existing soils can be found on Plates C-1a through C-1c, in Appendix C. Fill soils are considered OSHA Class ‘C’; submerged cohesive soils should also be considered OSHA Class ‘C’, unless they are dewatered first.

Critical Height is defined as the height a slope will stand unsupported for a short time; in cohesive soils, it is used to estimate the maximum depth of open-cuts at given side slopes. Critical Height may be calculated based on the soil cohesion. Values for various slopes and cohesion are shown on Plate D-1, in Appendix D. Cautions listed below should be exercised in use of Critical Height applications:

1. No more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or soils that will slough when not laterally supported are encountered within the excavation depth.

2. If the soil at the surface is dry to the point where tension cracks occur, any water in the crack will increase the lateral pressure considerably. In addition, if tension cracks occur, no cohesion should be assumed for the soils within the depth of the crack. The depth of the first waler should not exceed the depth of the potential tension crack. Struts should be installed before lateral displacement occurs.

3. Shoring should be provided for excavations where limited space precludes adequate side slopes, e.g., where granular soils will not stand on stable slopes and/or for deep open cuts.

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

The maximum (steepest) allowable slopes for OSHA Soil Types for excavations less than 20 feet are presented on Plate D-2, in Appendix D.

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

Computation of Bracing Pressures: The following method can be used for calculating earth pressure against bracing for open cuts. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by Equation (6). The design soil parameters for trench bracing design are presented on Plates C-1a through C-1c, in Appendix C.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{...........Equation (6)}$$

where:

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

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

Bottom Stability: In open-cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate D-7, in Appendix D.

If the excavation extends below groundwater, and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 5 feet below the excavation in accordance with Section 01578 of the latest edition of the City of Houston Standard General Requirement (COHSGR).

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

Dewatering: AEC notes that the 20.5 to 27.5 feet invert depths provided by HR Green are fairly deep for an open cut trench excavation in a limited space environment. Table 4 presents the depth that granular soils and groundwater was encountered within the trench and/or pipe bedding zone of the borings along the alignment.

Table 4. Groundwater and Granular Soils within Trench Zone

Boring No.	Invert Depth near Boring (ft)	Granular Soil Strata Encountered in Trench Zone	Groundwater Level (ft)
B-5	27.5	(14'-16') SC	16.2

Note: (a) Approximate boring location.
(b) Groundwater level conservatively assumed to be at boring cave in depth.
(c) SC = clayey sand.

As indicated on Table 4, granular soils will be encountered within the trench zone of Boring B-5. Although granular soils were encountered in Borings B-6 through B-12, the depth of the granular soil strata is below the anticipated storm sewer invert depth. In addition, groundwater may be encountered within the trench zone near Boring B-5. Possible ground water control measures include: (i) deep wells with turbine or submersible pumps; (ii) multi-staged well points; or (iii) water-tight sheet pile cut-off walls. **Note that extended and/or excessive dewatering can result in differential settlement of existing adjacent structures as the groundwater table is lowered. Special care should be exercised to prevent a change of the groundwater level below structures when performing dewatering operations for the storm sewer installation. One option to reduce such risk includes using a sheet pile cutoff wall to minimize**

seepage into the excavation, combined with a series of monitoring and reinjection wells (to maintain the ground table) around the construction area. General groundwater control recommendations are presented in Section 6.2 of this report. The options for dewatering presented here are for reference purposes only; it is the Contractor's responsibility to take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation.

5.2.3 Bedding and Backfill

Trench excavation, pipe embedment material, and backfill for the proposed storm sewers should be in general accordance with Section 02317 of the latest edition of the COHSCS. Backfill should be placed in loose lifts not exceeding 8 inches and compacted to 95 percent of its ASTM D-698 (Standard Proctor) maximum dry density at a moisture content ranging between optimum and 3 percent above optimum.

5.3 Manholes and Junction Boxes

Based on the drawings provided by HR Green, storm sewer manholes and junction boxes will have an invert depth of 20.5 to 27.5 feet. Cast-in-place and pre-cast manhole construction should be in general accordance with Sections 02081 and 02082 of the latest edition of the COHSCS, respectively. The Contractor should be responsible for designing, constructing and maintaining safe excavations for the proposed manholes. Manhole open-cut excavations shall be in general accordance with Section 5.2.2 of this report. Geotechnical recommendations to guide design of manholes and junction boxes are presented below.

5.3.1 Allowable Bearing Capacity

We assume mat foundations will be used for the manholes and junction boxes. Based on soils encountered in our borings, a net allowable bearing capacity of 1,800 psf for dead loads and 2,700 psf for total loads, whichever is critical should be used for mat foundations of the proposed manholes. These values include a factor of safety of 3 for dead load and 2 for total load, respectively.

The net footing pressure may be determined by:

1. Summing the weight of the load applied to the foundation, the weight of the foundation and the weight of soil backfill placed above the foundation.
2. Subtracting the weight of soil excavated from the foundation.
3. Dividing the result of items 1 and 2 by the base area of the foundation.

5.3.2 Uplift Resistance

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 on Plates C-1a through C-1c, in Appendix C. Recommended design criteria for uplift resistance are shown on Plate D-8, in Appendix D.

5.3.3 Lateral Earth Pressures

Typically, there is no movement 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 manhole 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 Equation (7). The design soil parameters for earth pressure design are presented on Plates C-1a through C-1c, in Appendix C.

$p_0 = (q_s + \gamma h_1 + \gamma' h_2) K_0 + \gamma_w h_2$ ....Equation (7)

where,
p_0 = at-rest earth pressure, (psf);
q_s = uniform surcharge pressure, (psf);
γ, γ' = wet and buoyant unit weights of soil, (pcf);
h_1 = depth from ground surface to ground water table, (ft);
h_2 = $z-h_1$, depth from ground water table to point under consideration, (ft);
z = depth below ground surface, (ft);
K_0 = coefficient of at-rest earth pressure;
γ_w = unit weight of water, 62.4 pcf.

5.3.4 Manhole Backfill Material

Manhole and junction box bedding and backfill should be in accordance with the Sections 02316 and 02317 of the latest edition of the COHSCS.

5.4 Pavement Reconstruction

According to HR Green, the entirety of existing pavement along Genesee Street and Tuam Street within the project alignments will be replaced with new concrete pavement. Based on drawings provided by HR Green, Genesee Street between West Dallas Street and Tuam Street will have two lanes (one lane in each direction) and have a curb-to-curb width of 27 to 36 feet. Tuam Street between Genesee Street and Helena Street will have four lanes (two lanes in each direction) and have a curb-to-curb width of 52 feet. The new pavement will be placed at or near existing grade.

The pavement design recommendations developed below are in accordance with the "AASHTO Guide for Design of Pavement Structures," 1993 edition.

5.4.1 Estimation of Traffic Loading

According to the Houston Regional Traffic Count Map (published by the Texas Transportation Institute), the Texas Department of Transportation (TxDOT) 24 hour Traffic Volume on Tuam Street between Albany Street and Helena Street was 6,070 vehicles per day (vpd) in 2001 and 4,250 vpd in 2006; the growth rate from 2001 to 2006 was a decrease of 30.0 percent. The TxDOT 24 hour Traffic Volume on Tuam Street between Baldwin Street and Bagby Street was 5,630 vpd in 2001 and 4,310 vpd in 2006; the growth rate from 2001 to 2006 was a decrease of 23.4 percent. Conversely, according to the COH's Traffic Counts Website, the 24 hour Traffic Volume on Tuam Street between Bagby Street to Brazos Street was 2,242 vpd in 2012 and 3,148 vpd in 2013, with a resulting growth rate from 2012 to 2013 of 40.4 percent.

AEC selected the available traffic data along Tuam Street between Bagby Street to Brazos Street for pavement design purposes, since the traffic growth rate for this section is positive (even though the traffic count location was taken two blocks away from the project alignment). Based on this data, AEC projected a traffic count of 4,054 vpd in 2014. However, given that an annual growth rate of 40 percent is unrealistic, AEC instead assumed a growth rate of 5 percent over a design life of 25 years (provided by HR Green). This growth rate was selected by AEC as a weighted average, considering that available 5 year growth rates in the area varied from -49 to 144 percent.

Traffic data was not available for Genesee Street. Based on AEC's site visit, traffic volume along Genesee Street between West Dallas Street and Tuam Street is fairly low and is mostly residential. AEC understands that a traffic count was not performed along the Genesee Street alignment, since the alignment between West Dallas Street and Tuam Street does not have any traffic signals and the street will have minimal traffic.

Traffic design information such as traffic volume, types of vehicles, percentage of heavy trucks, and traffic volume growth rate for the pavement was not available when this report was prepared.

<u>Estimate Anticipated Traffic Loads:</u> We first estimated traffic loads by estimating the number of repetitions of an 18-kip Equivalent Single-Axle Load (ESAL) over the project alignment. Pavement design is based on the anticipated design number of 18-kip ESAL the pavement is subjected to during its design life. The equation to calculate the number of 18-kip ESAL repetitions to use for pavement design is presented in Equation (14). Assumptions made by AEC to estimate 18-kip ESAL repetitions are presented on Table 5. According to HR Green, the pavement will have a design life of 25 years.

$$\text{18-kip ESAL} = (ADT)(T)(T_f)(D)(L)(G)(Y)(365) \qquad \text{...........Equation (8)}$$

where:
ESAL = 18-kip Equivalent Single-Axle Load repetitions;
ADT = Average Daily Traffic, vehicles per day;
T = Percent of heavy trucks;
T_f = Truck factor (vehicles with 5 or more axles);

D	= Directional factor;
L	= Lane factor;
G	= Growth factor;
Y	= Design life, in years.

Table 5. Parameters for Estimation of Traffic Loads along Tuam Street

Parameters	Values
2014 Average Daily Traffic (ADT), Projected	4,054 vpd (both directions combined)
Percent Heavy Trucks (T)	3% (assumed)
Truck factor (T_f)	4.0 (assumed)
Directional factor (D)	0.5 (2 lane in each direction)
Lane factor (L)	1.0 (2 lane in each direction)
Total Growth Rate Factor (G)	1.84 (5.0% annual growth rate over 25 years, assumed)
Design life (Y)	25 years
Estimated 18-kip ESALs	4,084,000

AEC notes that calculated number of 18-kip ESAL repetitions is highly sensitive to parameters such as percent heavy trucks, truck factor, and annual growth rate in pavement design. Differences between assumed and actual traffic parameters can have significant effects on overall pavement thickness design and ultimate roadway performance. AEC should be notified if different traffic loads or design parameters are required for pavement design at the site, so that our analysis can be updated accordingly.

5.4.2 Rigid Pavement

According to Section 10.05 of the COH Infrastructure Design Manual, residential roadways with concrete pavement width less than 27 feet from curb to curb must have a minimum concrete thickness of 6 inches and a minimum stabilized subgrade thickness of 6 inches, while residential roadways with concrete pavement width greater than 27 feet from curb to curb must have a minimum thickness of 7 inches and a minimum stabilized subgrade thickness of 6 inches. Major thoroughfares must have a minimum thickness of 8 inches and a minimum stabilized subgrade thickness of 8 inches.

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

Overall Standard Deviation (S_0)	0.35
Initial Serviceability (P_0)	4.5
Terminal Serviceability (P_t)	2.5
Reliability Level (R)	95%
Overall Drainage Coefficient (C_d)	1.2
Load Transfer Coefficient (J)	3.2
Loss of Support Category (LS)	1.0
Roadbed Soil Resilient Modulus (M_R)	4,500 psi
Elastic Modulus (E_{sb}) of Stabilized Soils	20,000 psi
Composite Effective Modulus of Subgrade Reaction (k)	91 pci
Mean Concrete Modulus of Rupture (S'_c)	600 psi (at 28 days)
Concrete Elastic Modulus (E_c)	3.37×10^6 psi

Pavement design was performed using the DARWin v3.0 computer program. Pavement sections for Genesee Street and Tuam Street are presented on Table 6. DARWin analysis results are presented on Plates F-1 through F-3, in Appendix F. Even though the minimum subgrade thickness required for Genesee Street is 6 inches (according to the COH Infrastructure Design Manual), AEC increased the subgrade thickness to 8 inches due to the presence of high to very high-plasticity soils encountered in our borings along the project alignment.

Table 6. Recommended Rigid Pavement Sections

Pavement Layer	Genesee Street between West Dallas and Tuam	Tuam Street between Genesee and Helena
Portland Cement Concrete	7*	9
Lime-stabilized Subgrade	8	8

Note: (*) Minimum thickness required by City of Houston Infrastructure Design Manual.

Given the above design parameters, the concrete pavement section for Genesee Street should sustain 1,076,142 repetitions of 18-kip ESALs and the pavement section for Tuam Street should sustain 4,834,811 repetitions of 18-kip ESALs. AEC should be notified if different standards or constants are required for pavement design at the site, so that our recommendations can be updated accordingly.

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

5.4.3 Reinforcing Steel

Reinforcing steel should be in accordance with Section 02751 of the latest edition of the COHSCS. Reinforcing steel is required to control pavement cracks, deflections across pavement joints and resist warping stresses in rigid pavements. The cross-sectional area of steel (A_s) required per foot of slab width can be calculated as follows (for both longitudinal and transverse steel).

$$A_s = FLW/(2f_s) \qquad \text{...........Equation (8)}$$

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

5.4.4 Pavement Subgrade Preparation

Existing pavement and base should be demolished in accordance with Section 02221 of the latest edition of the COHSCS. Subgrade preparation should extend a minimum of 2 feet beyond the paved area perimeters. After demolition of existing pavement and base, we recommend that a competent soil technician inspect the exposed subgrade to determine if there are any unsuitable soils or other deleterious materials. Excavate and dispose of unsuitable soils and other deleterious materials which will not consolidate; the excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths. The exposed soils should be proof-rolled in accordance with Item 216 of the 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.

Scarify the top 8 inches of the exposed subgrade and stabilize with at least 7 percent hydrated lime by dry soil weight. Lime stabilization shall be performed in accordance with Section 02336 of the latest edition of the COHSCS. The percentage of lime required for stabilization is a preliminary estimate for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. The stabilized soils should be compacted to 95 percent of their ASTM D 698 (Standard Proctor) dry density at a moisture content ranging from optimum to 3 percent above optimum.

5.5 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 hydrated lime. Excavated clay soils should be stabilized with at least 6 percent hydrated lime by dry soil weight. Lime stabilization shall be performed in accordance with Section 02336 of the latest edition of the 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 pavement, 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 pavement construction) 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

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

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures. Groundwater control should be in general accordance with Section 01578 of the latest edition of the COHSGR.

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

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

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, possible ground water control measures include: (i) deep wells with turbine or submersible pumps; (ii) multi-staged well points; or (iii) water-tight sheet pile cut-off walls. Generally, the groundwater depth should be lowered at least 5 feet below the excavation bottom (in accordance with Section 01578 of the latest edition of the COHSGR) to be able to work on a firm surface when water-bearing granular soils are encountered.

Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist him in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

Note that extended and/or excessive dewatering can result in differential settlement of existing adjacent structures as the groundwater table is lowered. Special care should be exercised to prevent a change of the groundwater level below structures when performing dewatering operations for the storm sewer installation. One option to reduce such risk includes using a sheet pile cutoff wall to minimize seepage into the excavation, combined with a series of monitoring and reinjection wells (to maintain the ground table) around the construction area.

For open cut construction in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. Guidelines for evaluating bottom stability are presented in Section 5.2.2 of this report.

Sheet Piling: Temporary water-tight sheet piling can installed to support excavations and also to control groundwater seepage into the excavations. Design soil parameters for sheet pile design are presented on Plates C-1a through C-1c, in Appendix C. AEC recommends that the sheet pile design consider both short-term and long-term parameters; whichever is critical should be used for design. The determination of the pressures exerted on the sheet piles by the retained soils shall consider active earth pressure, hydrostatic pressure, and uniform surcharge (including construction equipment, soil stockpiles, and traffic load, whichever surcharge is more critical). Sheet pile design should be based on the following considerations:

(1) Ground water elevation at the top of the ground surface on the retained side;
(2) Ground water elevation 5 feet below the bottom of the access shaft excavation (assuming dewatering operations using deep wells);
(3) Neglect cohesion for active pressure determination, Equation (6) in Section 5.2.2 of this report;
(4) The design retained height should extend from the ground surface to the water line tunnel invert depth;
(5) A 300 psf uniform surcharge pressure from construction equipment or soil stockpiles should be considered at the top of the sheet piles; loose soil stockpiles during access shaft construction should be limited to 3 foot high or less;
(6) Use a Factor of Safety of 2.0 for passive earth pressure in front of (i.e. the shaft side) the sheet piles.

Design, construction, and monitoring of sheet piles should be performed by qualified personnel who are experienced in this operation. Sheet piles should be driven in pairs, and proper construction controls provided to maintain alignment along the wall and prevent outward leaning of the sheet piles. Construction of the sheet piles should be in accordance with the latest edition of the COHSCS, or equivalent standard, such as Item 407 of the 2004 TxDOT Standard Specifications.

6.3 Construction Monitoring

Pavement construction and subgrade preparation, as well as excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

6.4 Monitoring of Existing Structures

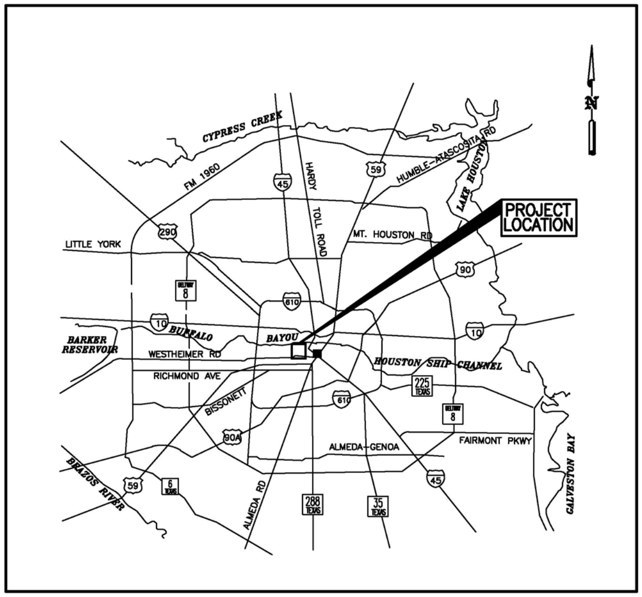
Existing structures in the vicinity of the proposed alignment should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel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

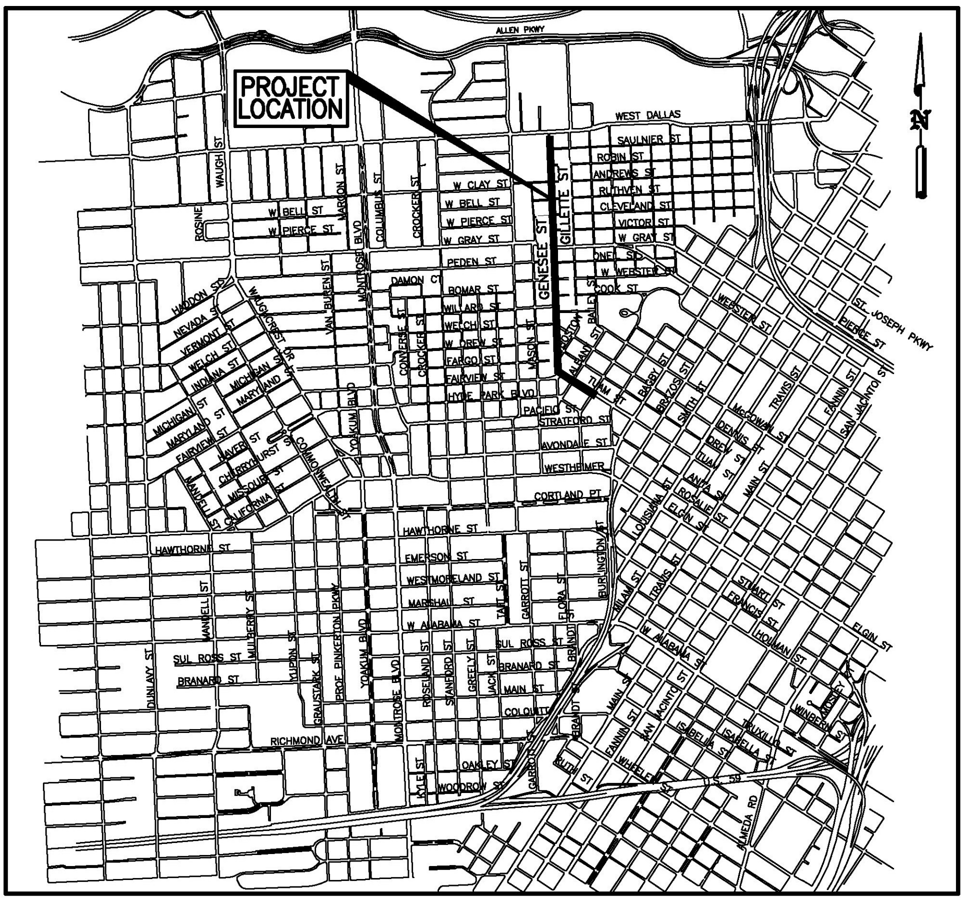
7.0 LIMITATIONS

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

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

APPENDIX A





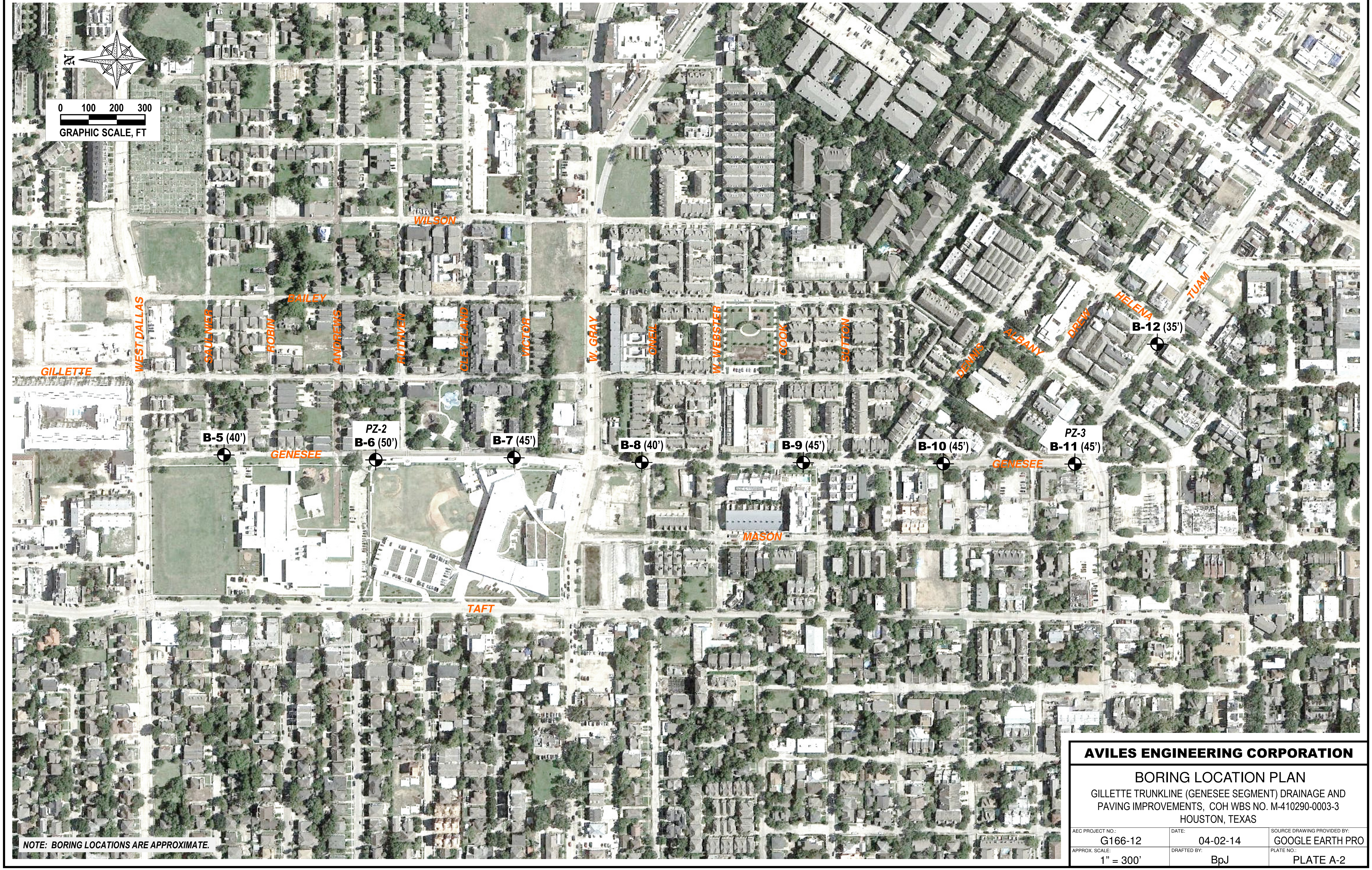
AVILES ENGINEERING CORPORATION

VICINITY MAP

GILLETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS, COH WBS NO. M-410290-0003-3

HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G166-12	03-31-14	HR GREEN, INC.
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
N.T.S.	BpJ	PLATE A-1



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

GILLETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS, COH WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G166-12	04-02-14	GOOGLE EARTH PRO
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
1" = 300'	BpJ	PLATE A-2

PROJECT: **Montrose and Midtown Storm Sewers**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-5**

DATE **1/18/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (ft):
Easting: 3116041.398
Northing: 13842263.3
Elevation: 50.974

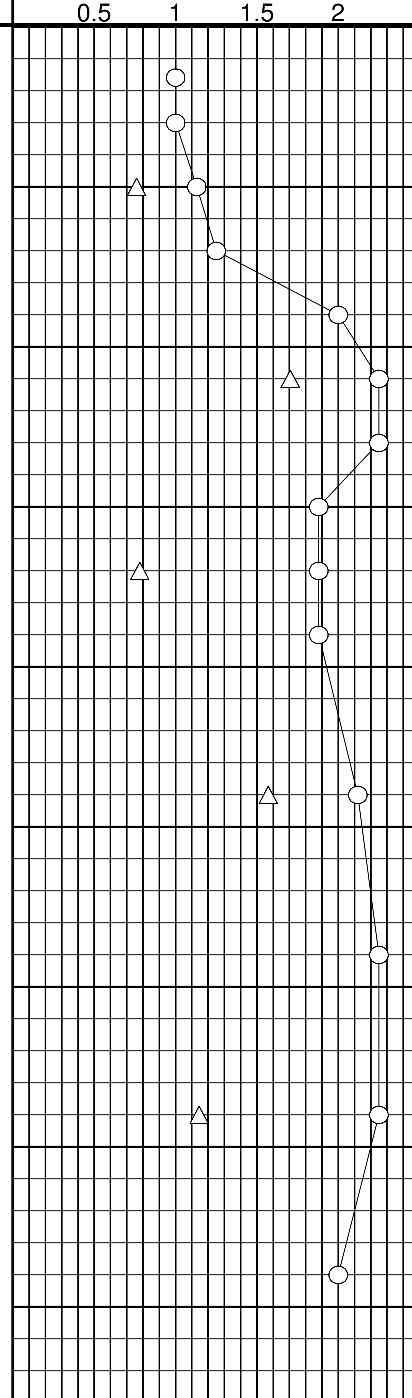
DEPTH IN FEET	DESCRIPTION
0	Pavement: 4" asphalt
	Base: 10" sand and shell
	Stiff to very stiff, tan and gray Fat Clay w/Sand (CH), with ferrous stains -tan 2'-4' -with calcareous nodules 4'-6'
6	Very stiff to hard, light gray, reddish brown, and tan Lean Clay (CL), with calcareous nodules and ferrous stains -with siltstone fragments 12'-14'
14	Light gray and tan Clayey Sand (SC)
16	Stiff to hard, reddish brown and light gray Fat Clay (CH), with slickensides -with calcareous nodules 16'-18' -tan and light gray, with ferrous stains 23'-25' -with siltstone fragments 33'-40'
40	Termination Depth = 40 feet.

Approx. Depth (ft)	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
1.5	29					
3	20		81	61	19	42
5	23	106				
7	22					
9	17		89	48	15	33
11	13	124				
13	15					
15	17		33	27	13	14
17	33	89				
19	35					
24	16	116	90	53	15	38
29	25					
34	30	97				
39	23		100	70	26	44

S.P.T. BLOWS / FT.

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **20** FEET WHILE DRILLING
WATER LEVEL AT **16.2** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-6**

DATE **1/18/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

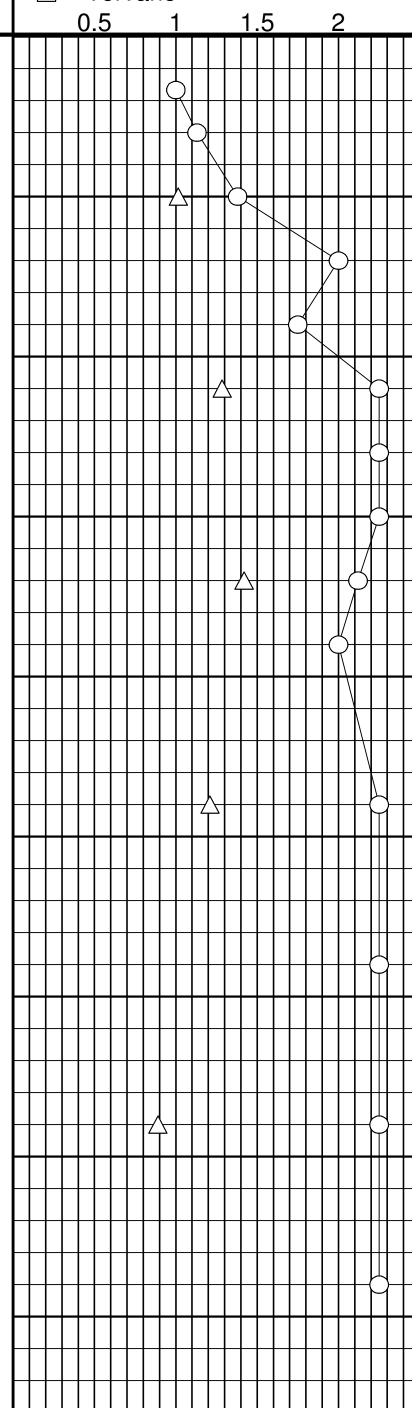
Survey Coordinates (ft):
Easting: 3116048.393
Northing: 13841728
Elevation: 50.804

DEPTH IN FEET	DESCRIPTION
0	Pavement: 8" concrete
	Base: 8" stabilized clay and shell
	Fill: dark gray Fat Clay (CH), with roots
	Very stiff, tan and light gray Fat Clay (CH), with slickensides -tan, with siltstone fragments and calcareous nodules 4'-6'
6	Very stiff, reddish brown and light gray Lean Clay w/Sand (CL), with silt seams, siltstone fragments, and abundant calcareous nodules
12	Very stiff, reddish brown and light gray Fat Clay (CH), with siltstone fragments, calcareous nodules, and ferrous stains
	Very stiff to hard, light gray and tan Lean Clay (CL), with ferrous stains -light gray and reddish brown, with slickensides, siltstone fragments, and calcareous nodules
18	Very stiff to hard, reddish brown and light gray Fat Clay (CH), with sand pockets
24	Very stiff to hard, light gray and tan Sandy Lean Clay (CL), with calcareous nodules and ferrous stains
30	Stiff to hard, reddish brown and light gray Fat Clay (CH), with slickensides -with siltstone fragments 28'-30' -with calcareous nodules 33'-40' -with siltstone fragments 38'-40'
36	
42	

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	28		88	71	22	49
	24					
	24	103				
			83	47	16	31
	20					
	31					
	16	117				
			93	45	13	32
	16					
	18					
	22	107				
			92	71	24	47
	34					
	16	118				
			97	70	27	43
	27					
	25	104				
			99	56	20	36
	20					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **45** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **43** FEET WHILE DRILLING
WATER LEVEL AT **31.1** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

PROJECT NO. G166-12 **PLATE A-4**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-6**

DATE **1/18/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Dense, reddish brown Silt (ML), with clay seams, wet -with siltstone fragments 43'-45'	48	21			96			
48				47	24						
			Termination Depth = 50 feet.								
54											
60											
66											
72											
78											
84											

BORING DRILLED TO **45** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **43** FEET WHILE DRILLING

WATER LEVEL AT **31.1** FEET AFTER **1/4 HR**

DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

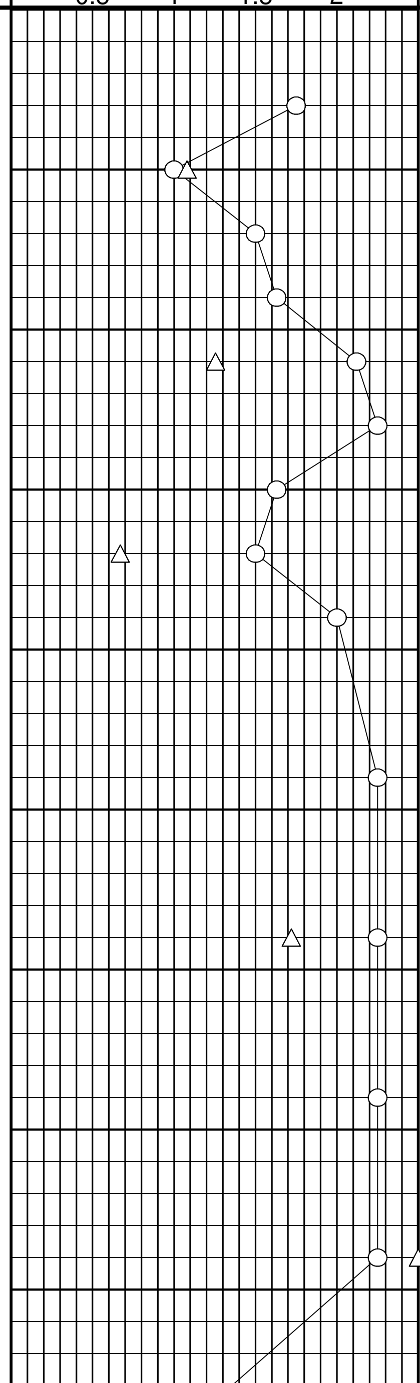
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-7**

DATE **1/21/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3116076.618
Northing: 13841238.32
Elevation: 50.977

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 8.5" concrete							
Base: 7.5" stabilized sand and gravel		20					
Fill: dark gray Fat Clay (CH), with ferrous stains		18		91	50	16	34
Stiff to very stiff, dark gray Fat Clay (CH), with ferrous stains -tan and light gray 4'-8' -with siltstone fragments and abundant calcareous nodules 6'-10' -reddish brown and light gray 8'-10		23	105				
		25		88	51	19	32
		24					
Very stiff to hard, light gray and tan Lean Clay w/Sand (CL), with ferrous stains -with sand pockets 12'-14'		16	119				
		13		70	32	13	19
		14					
Stiff to very stiff, light gray Fat Clay (CH), with sand pockets -light gray, tan, and reddish brown 18'-20'		21	105				
		27					
Hard, light gray and tan Lean Clay w/Sand (CL), with sand pockets		15		83	39	14	25
Very stiff to hard, reddish brown and light gray Fat Clay (CH), with slickensides		25	104				
-with siltstone fragments 33'-45', and calcareous nodules 33'-40'		23		99	60	16	44
		19	112				



DEPTH IN FEET: 0, 6, 12, 18, 24, 30, 36, 42

BORING DRILLED TO **45** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING
WATER LEVEL AT **33.7** FEET AFTER **DRILLING**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

PROJECT NO. G166-12 **PLATE A-5**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-7**

DATE **1/21/13** 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 (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Fat Clay (CH) (cont.) -with silt seams 43'-45'		17		○	93	60	17	43
			Termination Depth = 45 feet.								
48											
54											
60											
66											
72											
78											
84											

BORING DRILLED TO **45** FEET WITHOUT DRILLING FLUID

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

WATER LEVEL AT **33.7** FEET AFTER **DRILLING**

DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-8**

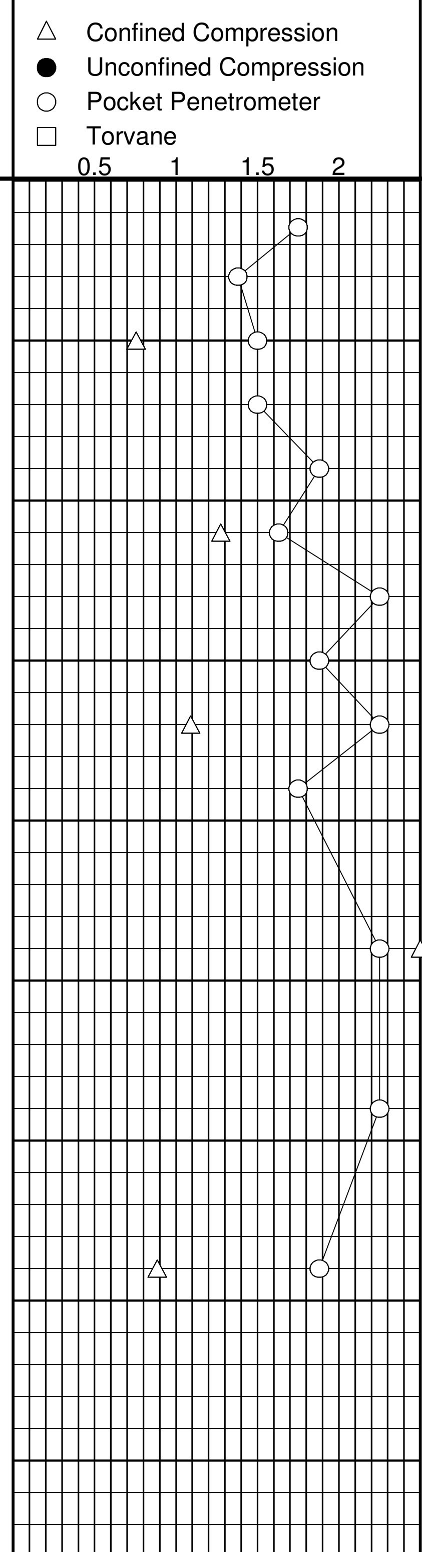
DATE **1/21/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (ft):
Easting: 3116078.065
Northing: 13840790.34
Elevation: 49.527

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 1" asphalt							
	Base: 10" stabilized sand and shell		24		91	65	21	44
	Stiff to very stiff, dark gray Fat Clay (CH), with ferrous stains		21					
	-gray and tan 2'-6'							
	-with siltstone fragments 4'-8', and calcareous nodules 4'-6'		21	107				
6	-dark brown and light gray, with silt partings 6'-8'		21		97	52	19	33
	Very stiff to hard, tan and gray Lean Clay (CL), with ferrous stains		17					
			18	116				
12	-gray, tan, and reddish brown 12'-14'		14		86	37	13	24
	-light gray 14'-16'		20					
	-with slickensides 16'-18'		19	108				
18	-brown, light gray, and tan, with silt and clay pockets 18'-20'		22		97	47	19	28
24	-light gray, tan, and reddish brown 23'-25'		17	117				
	Stiff to hard, light gray and brown Lean Clay w/Sand (CL)							
	-with sand pockets 28'-30'		20		84	44	16	28
30								
	-reddish brown and light gray, with silt seams, siltstone fragments, and calcareous nodules 33'-35'		22	107				
36								
	Hard, reddish brown and light gray Silty Clay (CL-ML), with siltstone fragments and calcareous nodules	50/4"	20		91	32	26	6
	Termination Depth = 40 feet.							
42								

SHEAR STRENGTH, TSF



BORING DRILLED TO **40** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING
WATER LEVEL AT **35.7** FEET AFTER **DRILLING**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

PROJECT: **Montrose and Midtown Storm Sewers**

ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-9**

DATE **1/21/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

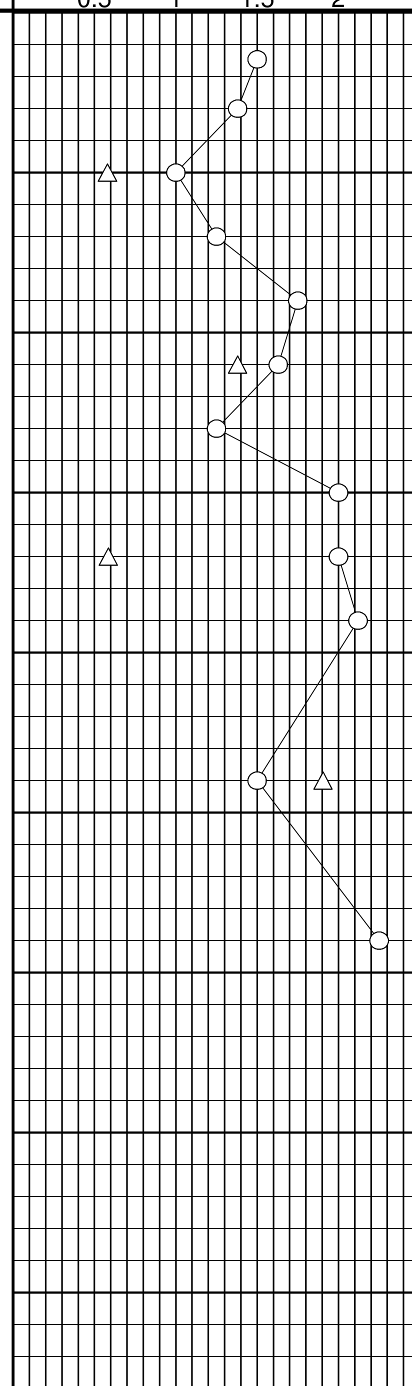
DESCRIPTION

Survey Coordinates (ft):
Easting: 3116100.037
Northing: 13840217.05
Elevation: 49.457

Description	S.P.T. Blows / Ft.	Moisture Content, %	Dry Density, PCF	-200 Mesh	Liquid Limit	Plastic Limit	Plasticity Index
Pavement: 2" asphalt							
Base: 9" stabilized sand with shell		24					
Stiff to very stiff, dark gray Fat Clay (CH), with slickensides and ferrous stains		24		93	62	18	44
-tan and gray 4'-6'		32	92				
-reddish brown and light gray, with siltstone fragments and calcareous nodules 6'-8'		26					
Stiff to very stiff, light gray and tan Lean Clay w/Sand (CL), with ferrous nodules		19		75	32	12	20
-with calcareous nodules 8'-10'							
-with sand partings and pockets 10'-16'		19	115				
		17					
		17		80	43	14	29
		21	106				
Very stiff to hard, reddish brown and light gray Fat Clay w/Sand (CH), with slickensides and ferrous stains		33					
-with sandy clay seams 23'-25'		15	116	79	65	23	42
		15					
Dense to very dense, tan and light gray Poorly Graded Sand w/Silt (SP-SM), wet	34	21					
-with siltstone fragments 38'-45'	69	22		10			

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DEPTH IN FEET: 0, 6, 12, 18, 24, 30, 36, 42

BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **33** FEET WHILE DRILLING
WATER LEVEL AT **29.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

PROJECT NO. G166-12 **PLATE A-7**

ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-9**

DATE **1/21/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane) 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Poorly Graded Sand w/Silt (SP-SM) (cont.) -tan 43'-45'	77	26						
			Termination Depth = 45 feet.								
48											
54											
60											
66											
72											
78											
84											

BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **33** FEET WHILE DRILLING
WATER LEVEL AT **29.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-10**

DATE **1/22/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

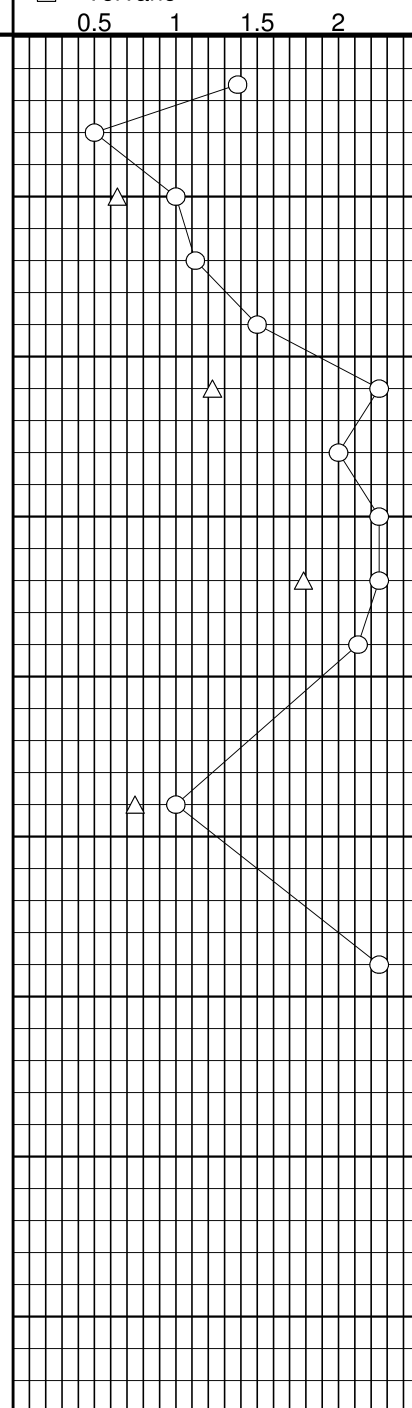
Survey Coordinates (ft):
Easting: 3116114.454
Northing: 13839715.18
Elevation: 49.433

Depth (ft)	Description
0	Pavement: 5" asphalt
	Base: 7" sand and gravel
	Fill: firm to very stiff, dark gray Fat Clay (CH), with ferrous stains -with shell 1'-4' -with gravel and sand seams 2'-4'
6	Very stiff, tan and light gray Fat Clay (CH), with siltstone fragments, calcareous nodules, and ferrous stains
	Very stiff to hard, light gray and tan Sandy Lean Clay (CL), with ferrous stains -light gray, tan, and reddish brown, with sand pockets 12'-14' -with siltstone fragments 14'-16', and calcareous nodules 14'-18' -light gray 16'-18'
18	Stiff to hard, light gray, tan, and brown Lean Clay w/Sand (CL) -with fat clay pockets 18'-20' -light gray 23'-25' -light gray and tan, with ferrous stains 28'-30'
	Light gray and tan Sandy Lean Clay (CL), with abundant sand seams
	Dense, light gray and tan Silty Sand (SM), wet

Depth (approx., ft)	S.P.T. Blows / Ft.	Moisture Content, %	Dry Density, PCF	-200 Mesh	Liquid Limit	Plastic Limit	Plasticity Index
1		30					
2.5		31		94	62	19	43
5		30	95				
7		33					
8.5		15		64	32	13	19
11		15	121				
13		15					
14.5		14		63	35	11	24
17		16	119				
19		32					
24		15	117	75	26	12	14
29		15					
34		21		60	37	15	27
39	48	22					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **33** FEET WHILE DRILLING
WATER LEVEL AT **29.8** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-10**

DATE **1/22/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane) 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Silty Sand (SM) (cont.)	36	23			13			
			Termination Depth = 45 feet.								
48											
54											
60											
66											
72											
78											
84											

BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **33** FEET WHILE DRILLING

WATER LEVEL AT **29.8** FEET AFTER **1/4 HR**

DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-11**

DATE **1/22/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

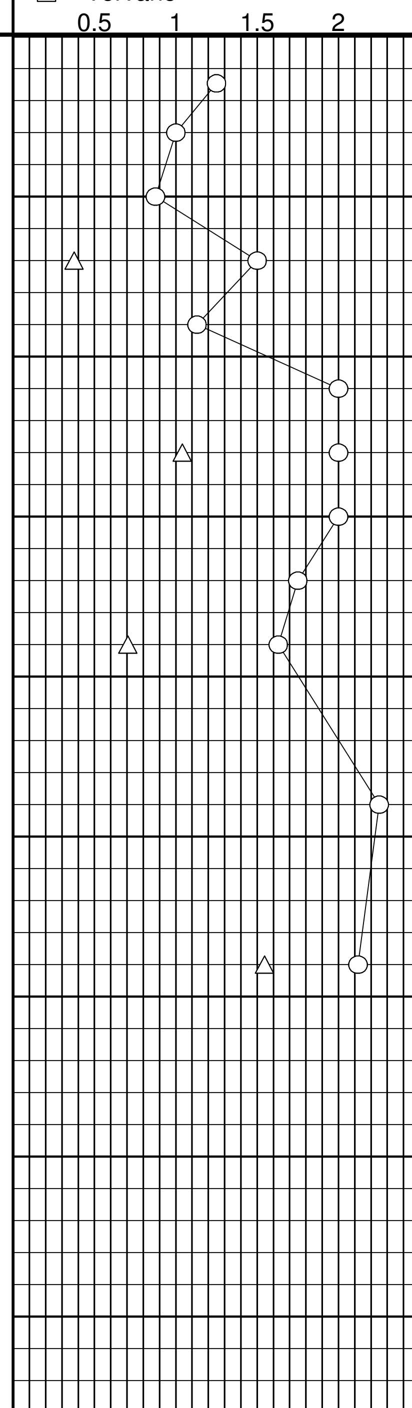
Survey Coordinates (ft):
Easting: 3116130.681
Northing: 13839251.11
Elevation: 48.398

DEPTH IN FEET	DESCRIPTION
0	Pavement: 2" asphalt
	Base: 9" stabilized sand and shell
	Firm to very stiff, dark gray Fat Clay (CH), with slickensides and ferrous stains -gray and tan 4'-6' -brown and gray 6'-8', with calcareous nodules 6'-10' -reddish brown and light gray, with siltstone fragments 8'-10'
6	
	Very stiff to hard, tan and gray Sandy Lean Clay (CL), with ferrous stains -with sand pockets 14'-16'
12	
	Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with slickensides -with calcareous nodules 16'-18'
18	
	Very stiff to hard, light gray and tan Lean Clay (CL), with sand partings and ferrous stains
24	
30	
	Light gray and tan Sandy Lean Clay (CL), with abundant sand seams
36	Very dense, tan Silty Sand (SM), wet -with siltstone fragments 38'-40'
42	

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	27					
	27		94	73	27	46
	33					
	33	91				
	32					
	15		64	36	14	22
	15	118				
	13					
	33		96	82	30	52
	32	94				
	14		89	41	14	27
	20	107				
	10		66	24	14	10
50/3"	19		15			

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **35** FEET WHILE DRILLING
WATER LEVEL AT **29.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Montrose and Midtown Storm Sewers** BORING **B-11**

DATE **1/22/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (△ Confined Compression, ● Unconfined Compression, ○ Pocket Penetrometer, □ Torvane; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Silty Sand (SM) (cont.) -tan and light gray 43'-45'	50/5"	34						
			Termination Depth = 45 feet.								
48											
54											
60											
66											
72											
78											
84											

BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **35** FEET WHILE DRILLING
WATER LEVEL AT **29.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

PROJECT: **Montrose and Midtown Storm Sewers**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-12**

DATE **1/22/13** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

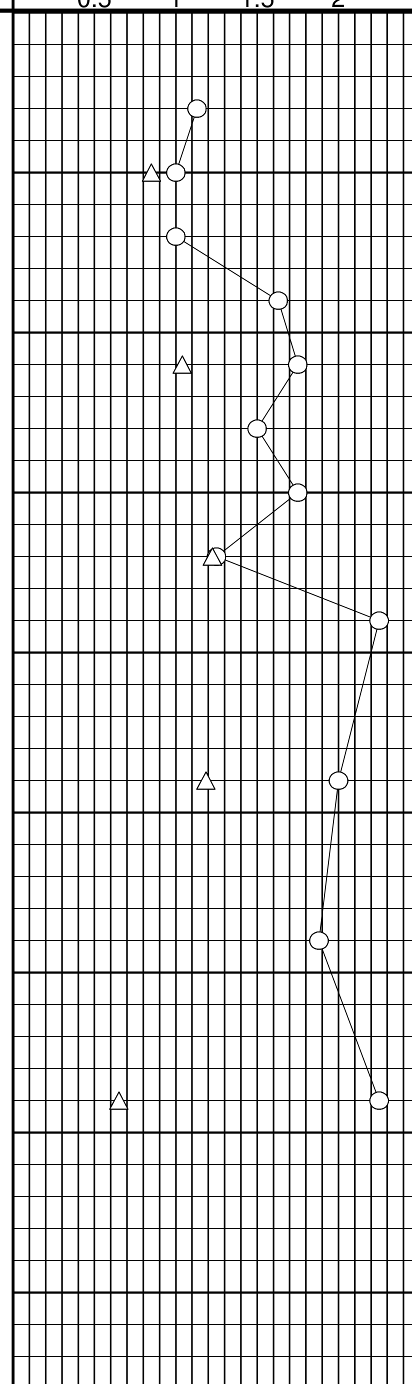
DESCRIPTION

Survey Coordinates (ft):
Easting: 3116588.52
Northing: 13838966.08
Elevation: 50.18

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

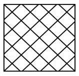
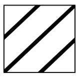
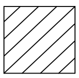
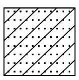
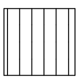
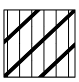
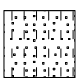
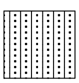
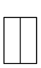
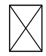
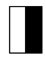
DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 5.5" asphalt		10					
Base: 12.5" sand and gravel							
Stiff to very stiff, dark gray Fat Clay (CH), with ferrous stains				92	77	24	53
		30					
		28	96				
-tan and gray 6'-8', with calcareous nodules 6'-10'		28					
-reddish brown and light gray, with siltstone fragments 8'-10'		29		99	62	23	39
Very stiff to hard, brown, reddish brown, and light gray Lean Clay w/Sand (CL), with ferrous stains		17	110				
-with slickensides, fat clay seams, siltstone fragments, and calcareous nodules 10'-12'		17					
-light gray and tan 12'-14'		15		71	29	13	16
-gray and tan 14'-16'							
-tan, reddish brown, and gray, with siltstone fragments and sand pockets 16'-18'		17	116				
-gray and tan 18'-20'		18					
Stiff to hard, light gray, tan, and reddish brown Lean Clay (CL), with ferrous stains		15	116				
		16		87	40	13	27
-reddish brown and light gray, with fat clay seams and siltstone fragments 33'-35'		21	110				
Termination Depth = 35 feet.							

DEPTH IN FEET: 0, 6, 12, 18, 24, 30, 36, 42



BORING DRILLED TO **35** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **27** FEET WHILE DRILLING
WATER LEVEL AT **28.3** FEET AFTER **1/4 HR**
DRILLED BY **V&S** CHECKED BY **WW** LOGGED BY **RJM**

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Paving
	Fill
	High plasticity clay
	Low plasticity clay
	Clayey sand
	Silt
	Silty low plasticity clay
	Poorly graded sand with silt
	Silty sand
Misc. Symbols	
	Water table depth during drilling
	Subsequent water table depth
	Pocket Penetrometer
	Confined Compression
Soil Samplers	
	Rock core
	Undisturbed thin wall Shelby tube
	Standard penetration test
	Auger

AYLES ENGINEERING CORP.

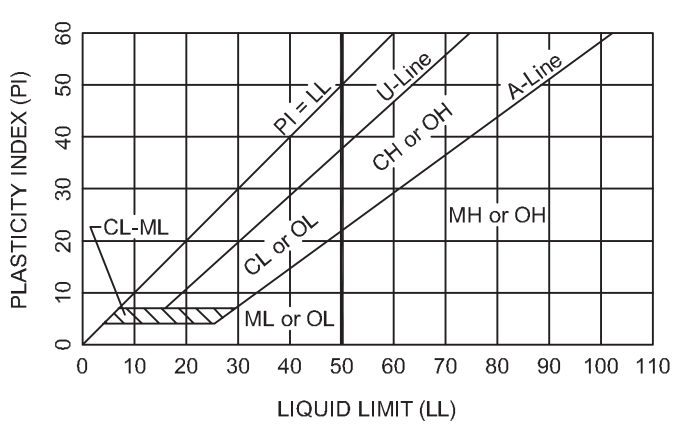
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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



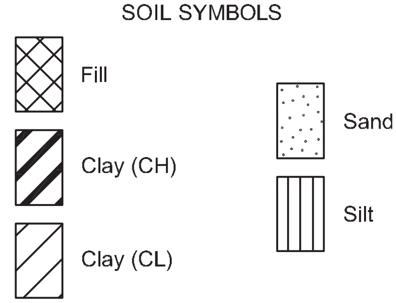
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

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



AVLES 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.

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

SUMMARY OF LABORATORY TEST RESULTS

BORING NO.	DEPTH	WATER CONTENT (%)	DRY DENSITY (pcf)	SHEAR STRENGTH (tsf)		ATTERBERG LIMITS			PERCENT PASSING NO. 200 (%)
				UNCONFINED COMPRESSION TEST	UU TEST (confining pressure in psi)	LL (%)	PL (%)	PI (%)	
B-5	0-2	29							
	2-4	20				61	19	42	80.6
	4-6	23	106.3		1.52 (3)				
	6-8	22							
	8-10	17				48	15	33	88.4
	10-12	13	123.5		3.41 (7)				
	12-14	15							
	14-16	17				27	13	14	32.6
	16-18	33	88.9		1.56 (11)				
	18-20	35							
	23-25	16	115.8		3.14 (14)	53	15	38	90.2
	28-30	25							
	33-35	30	96.6		2.29 (17)				
	38-40	23				70	26	44	99.5
B-6	0-2	28				71	22	49	87.6
	2-4	24							
	4-6	24	103.0		2.03 (3)				
	6-8	20				47	16	31	83.4
	8-10	31							
	10-12	16	116.9		2.57 (7)				
	12-14	16				45	13	32	82.5
	14-16	18							
	16-18	22	106.8		2.84 (11)				
	18-20	34				71	24	47	91.8
	23-25	16	117.9		2.42 (16)				
	28-30	27				70	27	43	96.9
	33-35	25	104.1		1.78 (23)				
	38-40	20				56	20	36	99.2
	43-45	21							95.6
	48-50	24							

SUMMARY OF LABORATORY TEST RESULTS

BORING NO.	DEPTH	WATER CONTENT (%)	DRY DENSITY (pcf)	SHEAR STRENGTH (tsf)		ATTERBERG LIMITS			PERCENT PASSING NO. 200 (%)
				UNCONFINED COMPRESSION TEST	UU TEST (confining pressure in psi)	LL (%)	PL (%)	PI (%)	
B-7	0-2	20							
	2-4	18				50	16	34	90.7
	4-6	23	105.0		2.16 (3)				
	6-8	25							
	8-10	24				51	19	32	87.5
	10-12	16	118.5		2.51 (7)				
	12-14	13							
	14-16	14				32	13	19	70.3
	16-18	21	105.4		1.34 (11)				
	18-20	27							
	23-25	15				39	14	25	83.0
	28-30	25	104.3		3.44 (19)				
	33-35	23				60	16	44	99.1
	38-40	19	112.0		5.09 (26)				
	43-45	17				60	17	43	92.8
B-8	0-2	24				65	21	44	90.7
	2-4	21							
	4-6	21	107.2		1.51 (3)				
	6-8	21				52	19	33	97.4
	8-10	17							
	10-12	18	115.7		2.55 (7)				
	12-14	14				37	13	24	85.6
	14-16	20							
	16-18	19	108.2		2.18 (11)				
	18-20	22				47	19	28	96.8
	23-25	17	117.1		5.01 (16)				
	28-30	20				44	16	28	84.2
	33-35	22	106.8		1.77 (23)				
	38-40	20				32	26	6	91.4

SUMMARY OF LABORATORY TEST RESULTS

BORING NO.	DEPTH	WATER CONTENT (%)	DRY DENSITY (pcf)	SHEAR STRENGTH (tsf)		ATTERBERG LIMITS			PERCENT PASSING NO. 200 (%)
				UNCONFINED COMPRESSION TEST	UU TEST (confining pressure in psi)	LL (%)	PL (%)	PI (%)	
B-9	0-2	24							
	2-4	24				62	18	44	92.8
	4-6	32	92.0		1.16 (3)				
	6-8	26							
	8-10	19				32	12	20	75.4
	10-12	19	114.7		2.76 (7)				
	12-14	17							
	14-16	17				43	14	29	79.5
	16-18	21	105.8		1.17 (11)				
	18-20	33							
	23-25	15	116.1		3.81 (16)	65	23	42	79.0
	28-30	15							
	33-35	21							
	38-40	22							10.4
	43-45	26							
B-10	0-2	30							
	2-4	31				62	19	43	94.1
	4-6	30	94.7		1.28 (3)				
	6-8	33							
	8-10	15				32	13	19	64.0
	10-12	15	120.9		2.45 (7)				
	12-14	15							
	14-16	14				35	11	24	63.3
	16-18	16	119.0		3.57 (11)				
	18-20	32							
	23-25	15	116.7		1.5 (16)	26	12	14	74.7
	28-30	15							
	33-35	21				37	15	22	60.4
	38-40	22							
	43-45	23							13.0

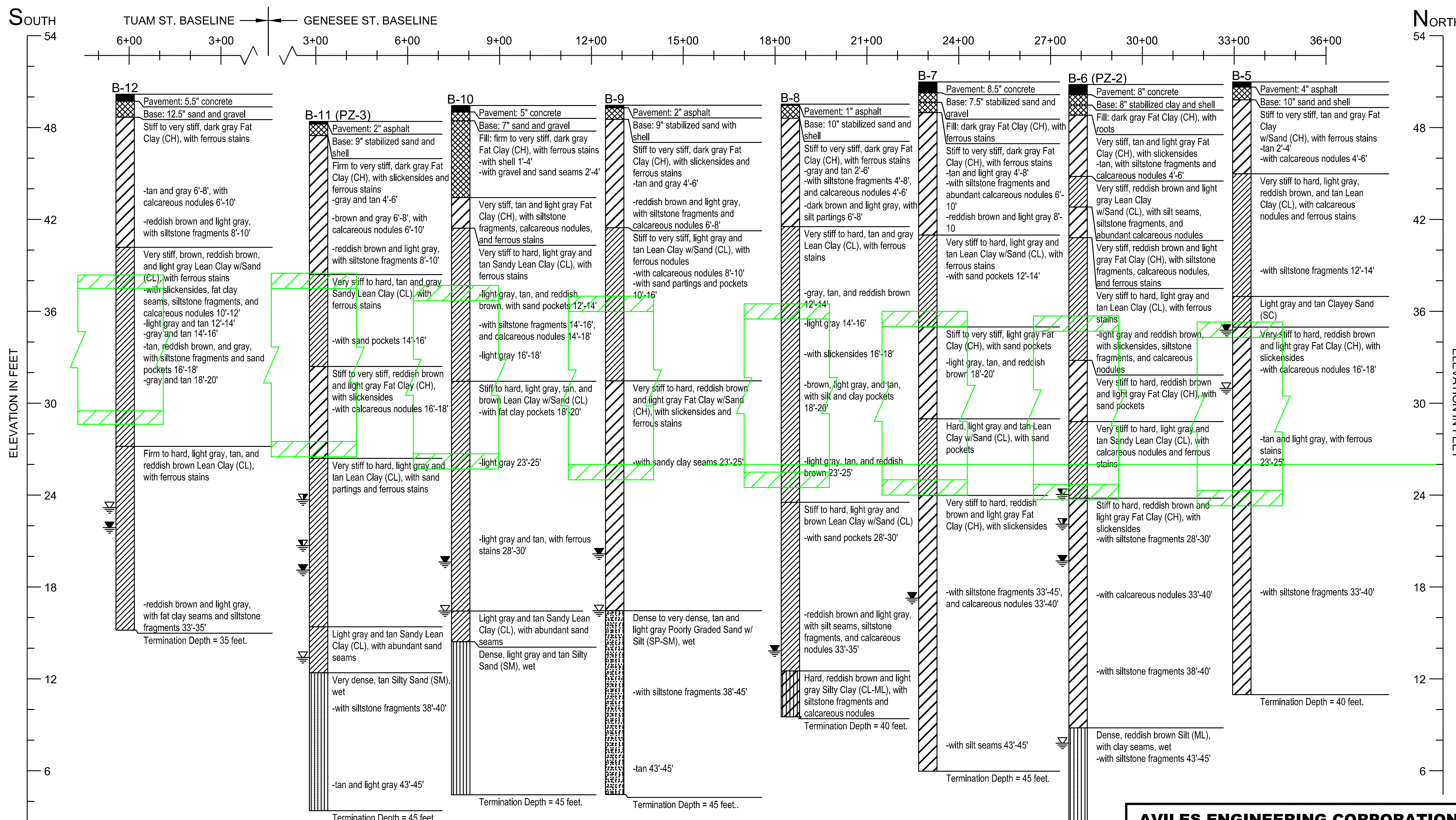
SUMMARY OF LABORATORY TEST RESULTS

BORING NO.	DEPTH	WATER CONTENT (%)	DRY DENSITY (pcf)	SHEAR STRENGTH (tsf)		ATTERBERG LIMITS			PERCENT PASSING NO. 200 (%)
				UNCONFINED COMPRESSION TEST	UU TEST (confining pressure in psi)	LL (%)	PL (%)	PI (%)	
B-11	0-2	27							
	2-4	27							
	4-6	33				73	27	46	93.7
	6-8	33	90.9		.75 (5)				
	8-10	32							
	10-12	15				36	14	22	64.4
	12-14	15	118.0		2.08 (9)				
	14-16	13							
	16-18	33				82	30	52	96.1
	18-20	32	94.0		1.41 (13)				
	23-25	14				41	14	27	88.7
	28-30	20	107.0		3.09 (19)				
	33-35	10				24	14	10	66.3
	38-40	19							15.5
	43-45	34							
B-12	0-2	10							
	2-4	30				77	24	53	92.1
	4-6	28	95.6		1.7 (3)				
	6-8	28							
	8-10	29				62	23	39	99.4
	10-12	17	110.2		2.08 (7)				
	12-14	17							
	14-16	15				29	13	16	70.7
	16-18	17	116.4		2.45 (11)				
	18-20	18							
	23-25	15	116.3		2.37 (16)				
	28-30	16				40	13	27	86.9
	33-35	21	109.9		1.30 (20)				

AVILES
ENGINEERING CORP.

APPENDIX B

Plate B-1	Generalized Soil Profile
Plates B-2 and B-3	Piezometer Installation Details



LEGEND:

Paving	Fill	Low plasticity clay	High plasticity clay	Clayey sand	Silty sand	Silt	Silty low plasticity clay	Poorly graded sand with silt	8'x8' and 10'x10' Storm Sewer RCB

- Depth of groundwater encountered duringdrilling
- Depth of groundwater 15 min. after initial encounter
- Depth of groundwater on 01–24–13 (piezometer)
- Depth of groundwater on 01–24–13 (piezometer)

NOTES:

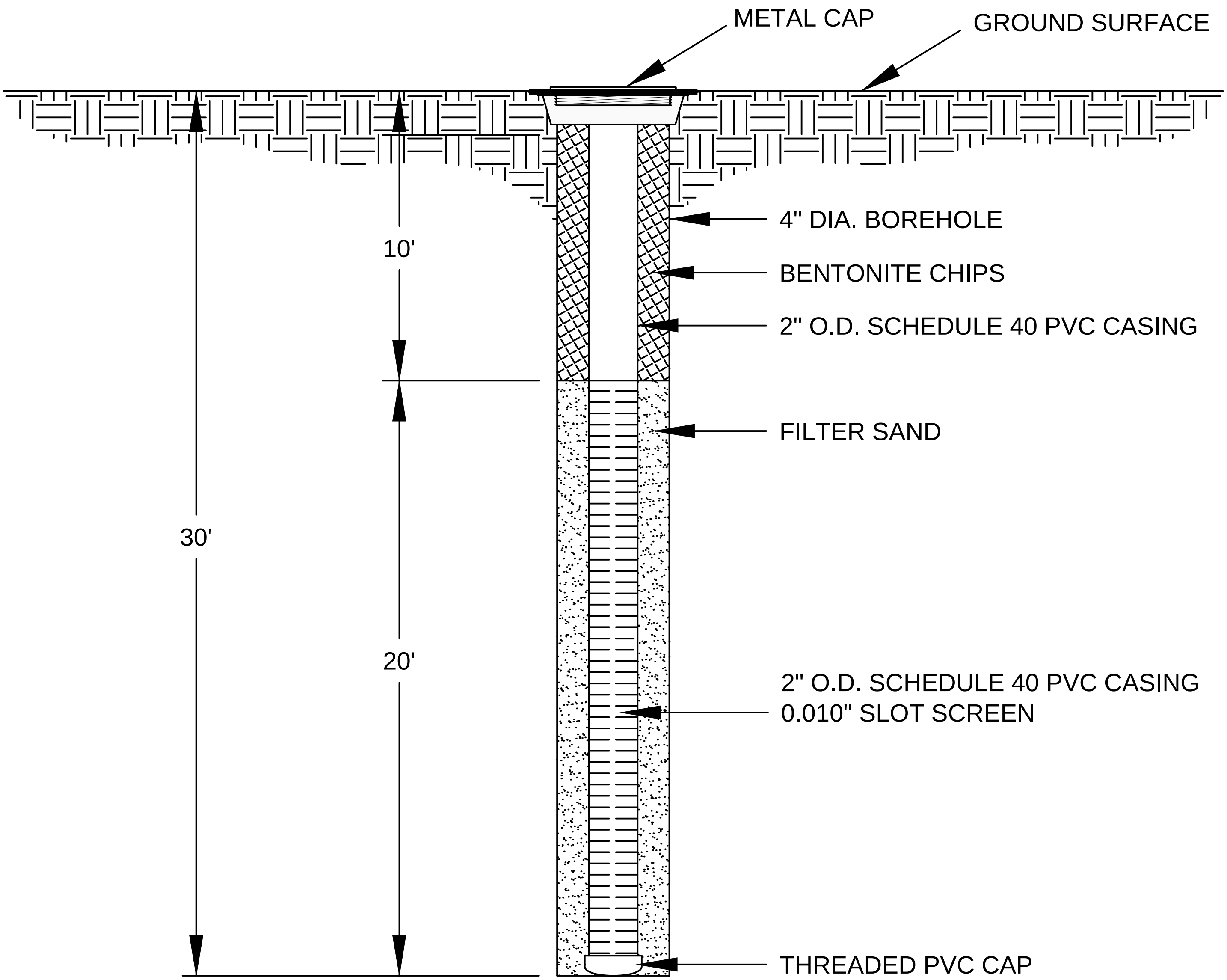
1. *SOIL STRATIGRAPHY AND SECONDARY SOIL STRUCTURE (SUCH AS SEAMS, LAYERS, OR POCKETS OF SANDS, SILTS, SLICKENSIDES, AND FISSURES) THAT ARE DIFFERENT FROM WHAT WERE IDENTIFIED IN THE ACTUAL BORINGS MAY EXIST AWAY FROM THESE BORINGS.*

AVILES ENGINEERING CORPORATION

GENERALIZED SOIL PROFILE

GILLETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS, COH WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO.: G166-12	DATE: 04-02-14	SOURCE DRAWING PROVIDED BY: AVILES ENGINEERING CORP.
VERTICAL SCALE: 1" = 6' HORIZONTAL SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-1



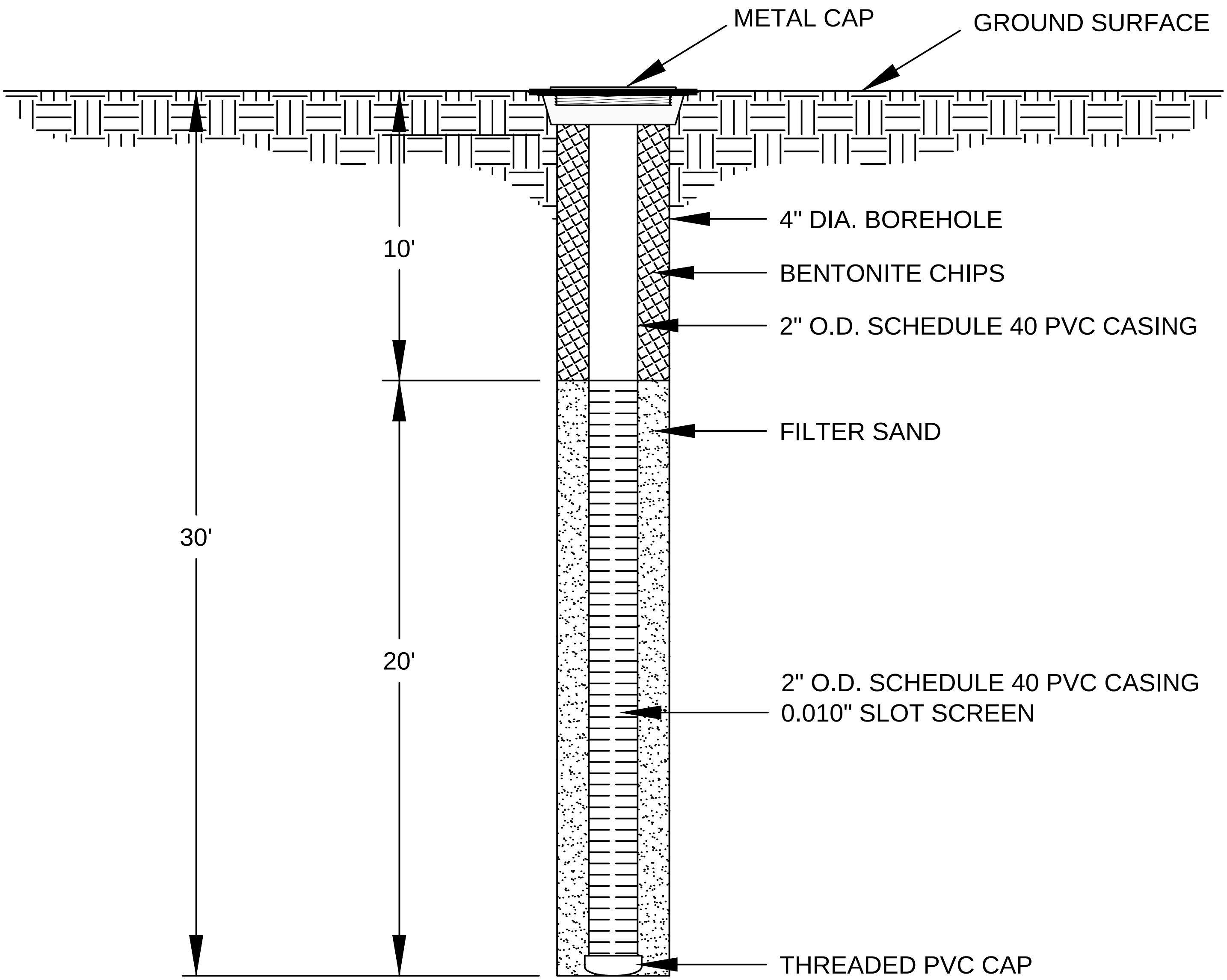
GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
26.76 FT	1/24/13
28.70 FT	2/21/13

AVILES ENGINEERING CORPORATION

PIEZOMETER INSTALLATION DETAILS
BORING B-6 (PZ-2)

GILLETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS, COH WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO. :	DATE:	SOURCE DWG. BY:
G166-12	04-02-14	AVILES ENGINEERING CORP.
SCALE:	DRAWN BY:	PLATE NO. :
N.T.S.	BpJ	PLATE B-2



GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
27.70 FT	1/24/13
24.70 FT	2/21/13

AVILES ENGINEERING CORPORATION

PIEZOMETER INSTALLATION DETAILS BORING B-11 (PZ-3)

GILLETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS, COH WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO. :	DATE:	SOURCE DWG. BY:
G166-12	04-02-14	AVILES ENGINEERING CORP.
SCALE:	DRAWN BY:	PLATE NO. :
N.T.S.	BpJ	PLATE B-3

AVILES ENGINEERING CORP.

APPENDIX C

Plates C-1a thru C-1c	Recommended Geotechnical Design Parameters
Plate C-2	Load Coefficients for Pipe Loading
Plate C-3	Live Loads on Pipe Crossing Under Roadway

G166-12 GILETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS
SOIL PARAMETERS FOR UNDERGROUND UTILITIES

Boring	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term					Long-Term				
						C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p
B-5	0-1	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	1-8	Stiff to very stiff CH/CL	130	68	B	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	8-14	Very stiff to hard CL	140	78	B	3400	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89
	14-16	SC	120	58	C	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
	16-20	Stiff CH	118	56	C*	1600	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	20-30	Very stiff to hard CH	135	73	n/a	3100	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76
	30-40	Very stiff to hard CH	126	64	n/a	2300	0	1.00	1.00	1.00	225	16	0.57	0.72	1.76
B-6	0-2	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	2-10	Very stiff CH/CL	128	66	B	2000	0	1.00	1.00	1.00	200	16	0.57	0.72	1.76
	10-18	Very stiff to hard CL	130	68	B	2600	0	1.00	1.00	1.00	250	18	0.53	0.69	1.89
	18-27	Very stiff to hard CH/CL	130	68	B (16-20)	2400	0	1.00	1.00	1.00	225	16	0.57	0.72	1.76
	27-42	Stiff to hard CH	130	68	n/a	1800	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	42-50	Dense ML	125	63	n/a	0	32	0.31	0.47	3.25	0	32	0.31	0.47	3.25
B-7	0-2	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	2-10	Stiff to very stiff CH	129	67	B	2000	0	1.00	1.00	1.00	200	16	0.57	0.72	1.76
	10-16	Very stiff to hard CL	138	76	B	2500	0	1.00	1.00	1.00	250	18	0.53	0.69	1.89
	16-22	Stiff to very stiff CH	127	65	B (16-20)	1300	0	1.00	1.00	1.00	125	16	0.57	0.72	1.76
	22-40	Very stiff to hard CL/CH	130	68	n/a	3400	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76
	40-45	Very stiff CH	125	63	n/a	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
B-8	0-1	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	1-8	Stiff to very stiff CH	130	68	B	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	8-14	Very stiff to hard CL	137	75	B	2500	0	1.00	1.00	1.00	250	18	0.53	0.69	1.89
	14-20	Very stiff to hard CL	129	67	B	2200	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	20-30	Hard CL	137	75	n/a	3600	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89
	30-37	Stiff to very stiff CL	131	69	n/a	1800	0	1.00	1.00	1.00	175	18	0.53	0.69	1.89
	37-40	Hard CL-ML	125	63	n/a	2000	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89

G166-12 GILETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS
SOIL PARAMETERS FOR UNDERGROUND UTILITIES

Boring	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term					Long-Term				
						C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p
B-9	0-1	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	1-8	Stiff to very stiff CH	121	59	B	1200	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	8-16	Very stiff to hard CL	137	75	B	2400	0	1.00	1.00	1.00	225	18	0.53	0.69	1.89
	16-20	Stiff to hard CL/CH	128	66	B	1200	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	20-33	Very stiff to hard CH	133	71	n/a	3000	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76
	33-38	Dense SP-SM	125	63	n/a	0	32	0.31	0.47	3.25	0	32	0.31	0.47	3.25
	38-45	Very dense SP-SM	125	63	n/a	0	34	0.28	0.44	3.54	0	34	0.28	0.44	3.54
B-10	0-1	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	1-6	Fill: firm to very stiff CH	124	62	C	1000	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	6-8	Very stiff CH	124	62	B	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	8-14	Very stiff to hard CL	139	77	B	2400	0	1.00	1.00	1.00	225	18	0.53	0.69	1.89
	14-18	Very stiff to hard CL	138	76	B	3600	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89
	18-35	Stiff to hard CL	135	73	B (18-20)	1500	0	1.00	1.00	1.00	150	18	0.53	0.69	1.89
	35-45	Dense SM	125	63	n/a	0	32	0.31	0.47	3.25	0	32	0.31	0.47	3.25
B-11	0-1	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	1-10	Firm to very stiff CH	121	59	B	800	0	1.00	1.00	1.00	75	16	0.57	0.72	1.76
	10-16	Very stiff to hard CL	136	74	B	2100	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	16-22	Stiff to very stiff CH	124	62	B (16-20)	1400	0	1.00	1.00	1.00	125	16	0.57	0.72	1.76
	22-36	Very stiff to hard CL	128	66	n/a	3100	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89
	36-45	Very dense SM	125	63	n/a	0	34	0.28	0.44	3.54	0	34	0.28	0.44	3.54
B-12	0-2	Pavement/Base	120	58	n/a	0	0	1.00	1.00	1.00	0	0	1.00	1.00	1.00
	2-10	Stiff to very stiff CH	123	61	B	1700	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	10-16	Very stiff CL	129	67	B	2100	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	16-30	Very stiff to hard CL	133	71	B (16-20)	2400	0	1.00	1.00	1.00	225	18	0.53	0.69	1.89
	30-35	Stiff to hard CL	133	71	n/a	1300	0	1.00	1.00	1.00	125	18	0.53	0.69	1.89

G166-12 GILETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS
SOIL PARAMETERS FOR UNDERGROUND UTILITIES

Boring	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term					Long-Term				
						C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p

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

(2) C = Soil ultimate cohesion for short term (upper limit of 3,600 psf for design purposes), φ = Soil friction angle for short term;

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

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

(5) CL = Lean Clay, CH = Fat Clay, CL-ML = Silty Clay; SC = Clayey Sand; SM = Silty Sand; ML = Silt; SC-SM = Silty Clayey Sand; GC = Clayey Gravel
SP-SM = Poorly Graded Sand with Silt; SP-SC = Poorly Graded Sand with Clay;

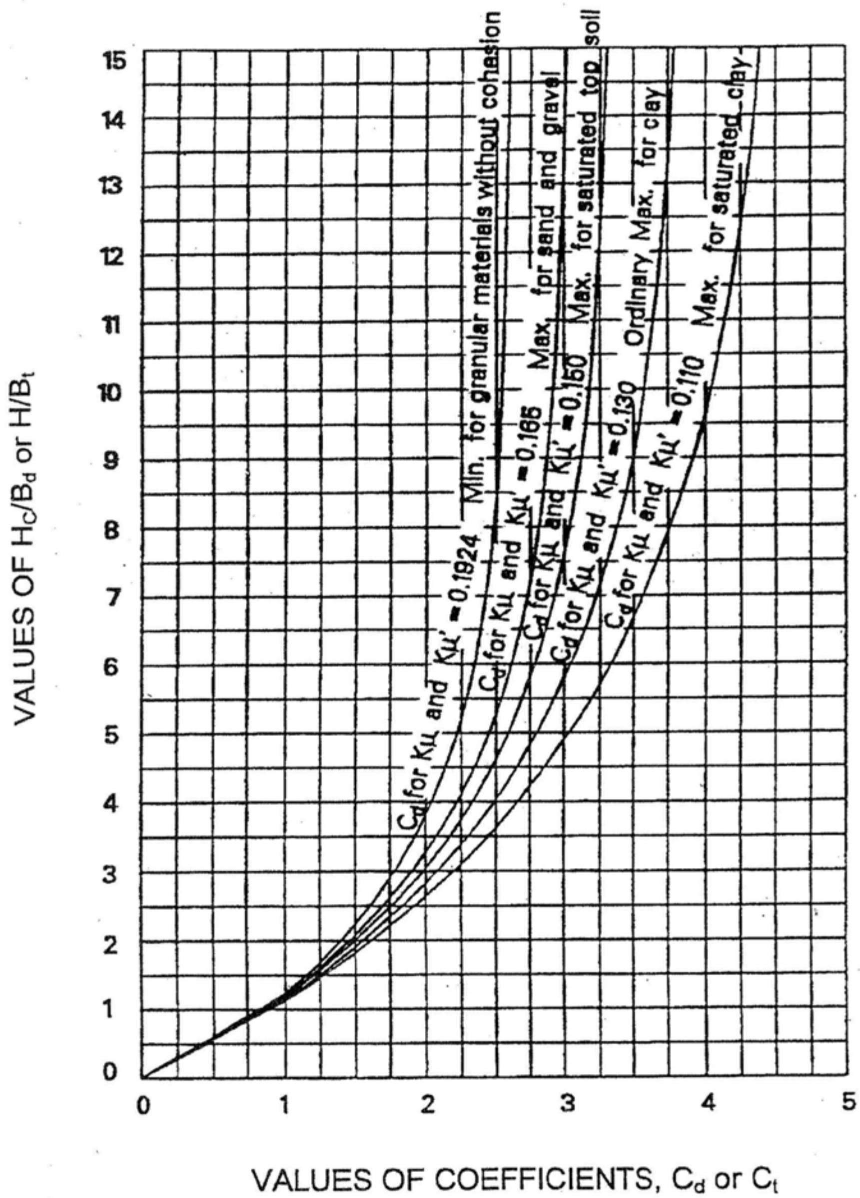
(6) OSHA Soil Types for soils in the top 20 feet below grade:

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

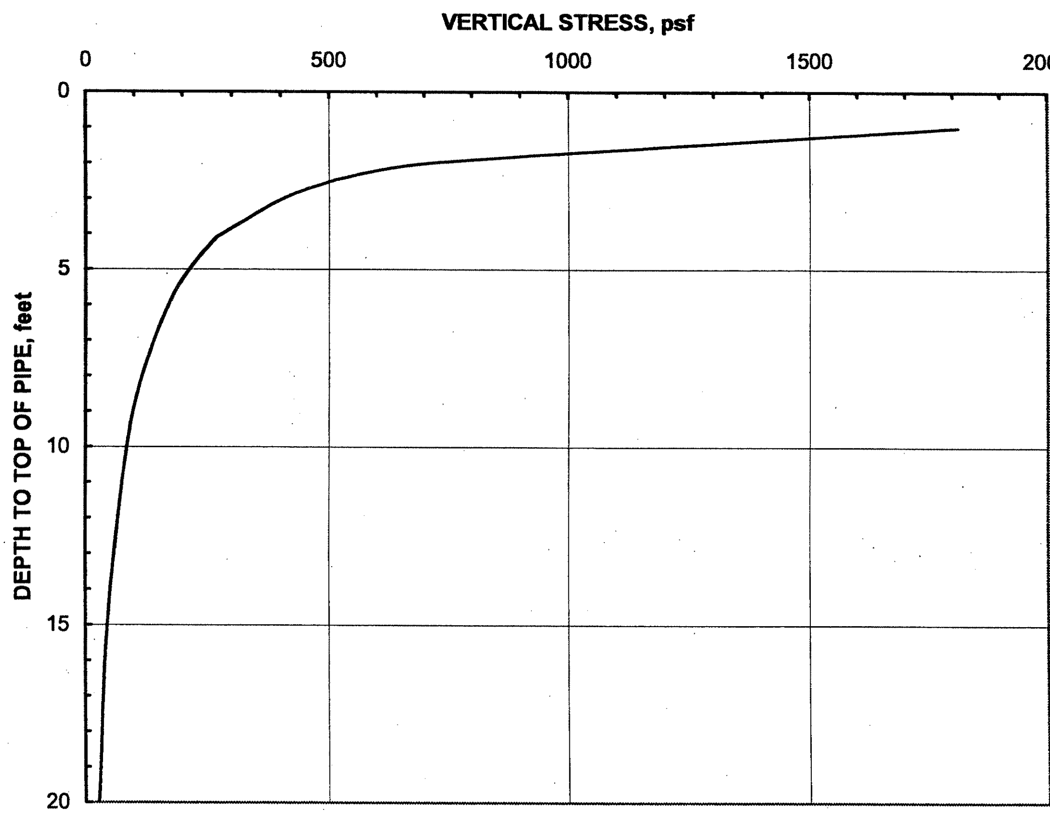
B: cohesive soils with qu = 0.5 tsf or greater

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

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



Reference: US Army Corps of Engineers Engineering Manual, EM 1110-2-2909, Oct. 31, 1997, Figure 2-5.



LIVE LOADS ON PIPE CROSSING UNDER ROADWAY

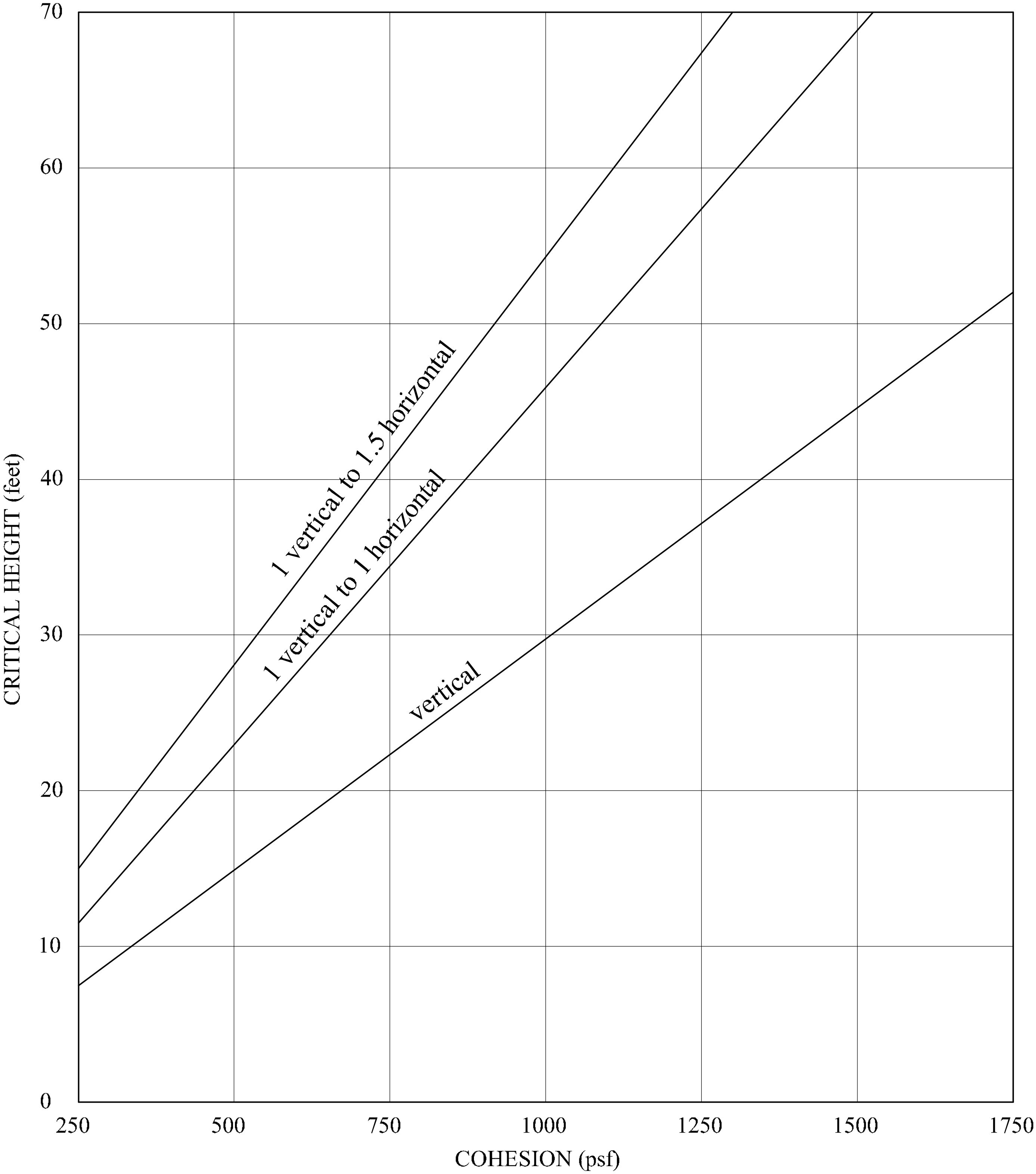
Note: 1. The vertical stress was estimated using AASHTO HS20 truck axle loadings on paved surfaces (Reference: ASCE 15-98, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standandard Installations").
2. Single truck passing.

AVILES ENGINEERING CORP.

APPENDIX D

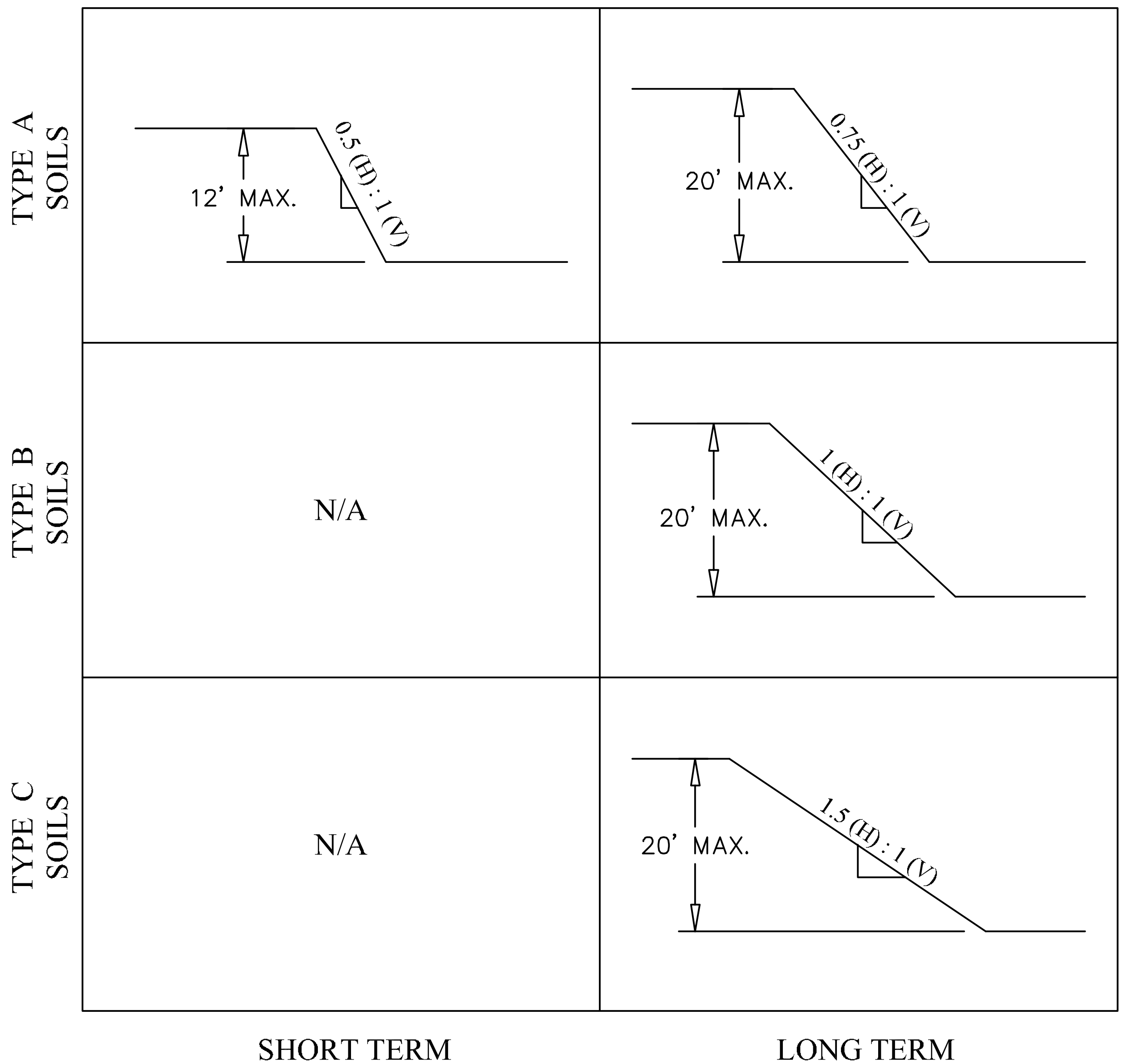
ENGINEERING CORP.

Critical Heights of Cut Slopes in Nonfissured Clays



Note: The charts are calculated based on NAVFAC DM7.1, Page 7.1-319, assuming the critical circles are toe circles, and wet unit weight of soils = 125pcf.

MAXIMUM ALLOWABLE SLOPES



NOTES:

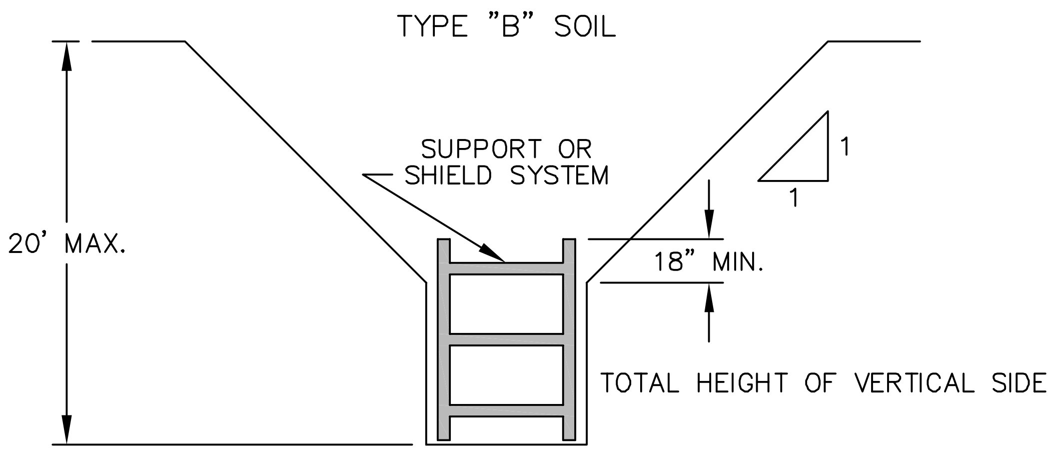
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

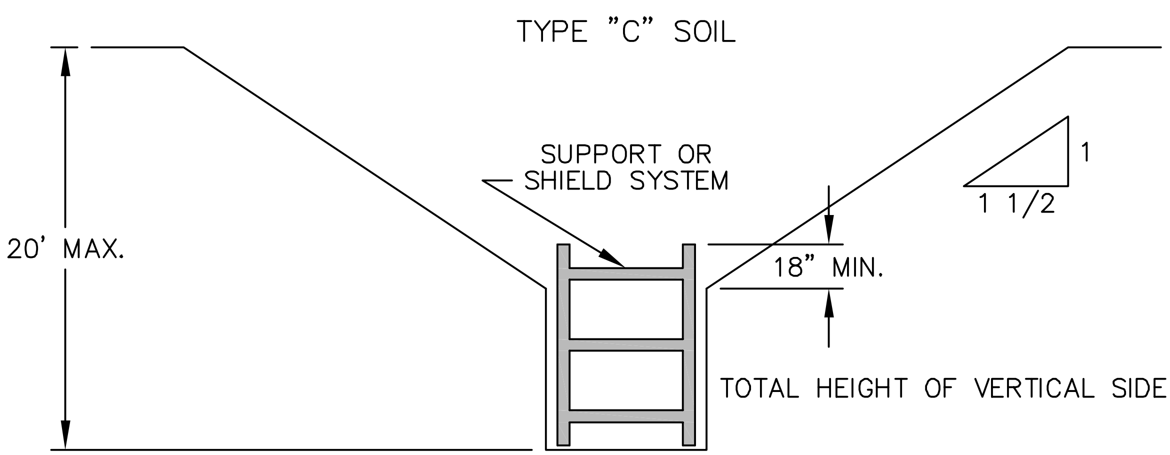
(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

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

ENGINEERING CORP.

A COMBINATION OF BRACING AND OPEN CUTS



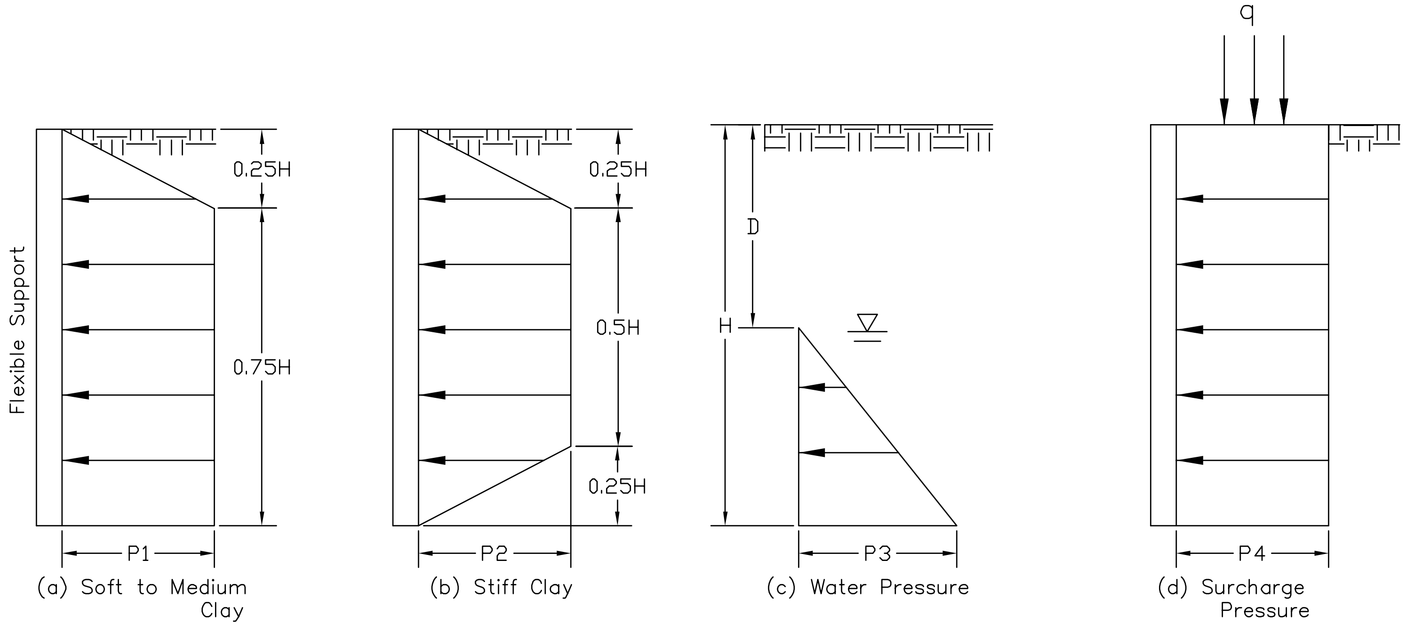


Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

AVILES ENGINEERING CORP.

LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



Empirical Pressure Distributions

Where:

H = Total excavation depth, feet

D = Depth to water table, feet

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

P2 = Lateral earth pressure = 0.4γH, psf

P3 = Water pressure = γ_w (H-D), psf

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

γ = Effective unit weight of soil, pcf

γ_w = Unit weight of water, pcf

C = Drained shear strength or cohesion, psf

K_a = Coefficient of active earth pressure

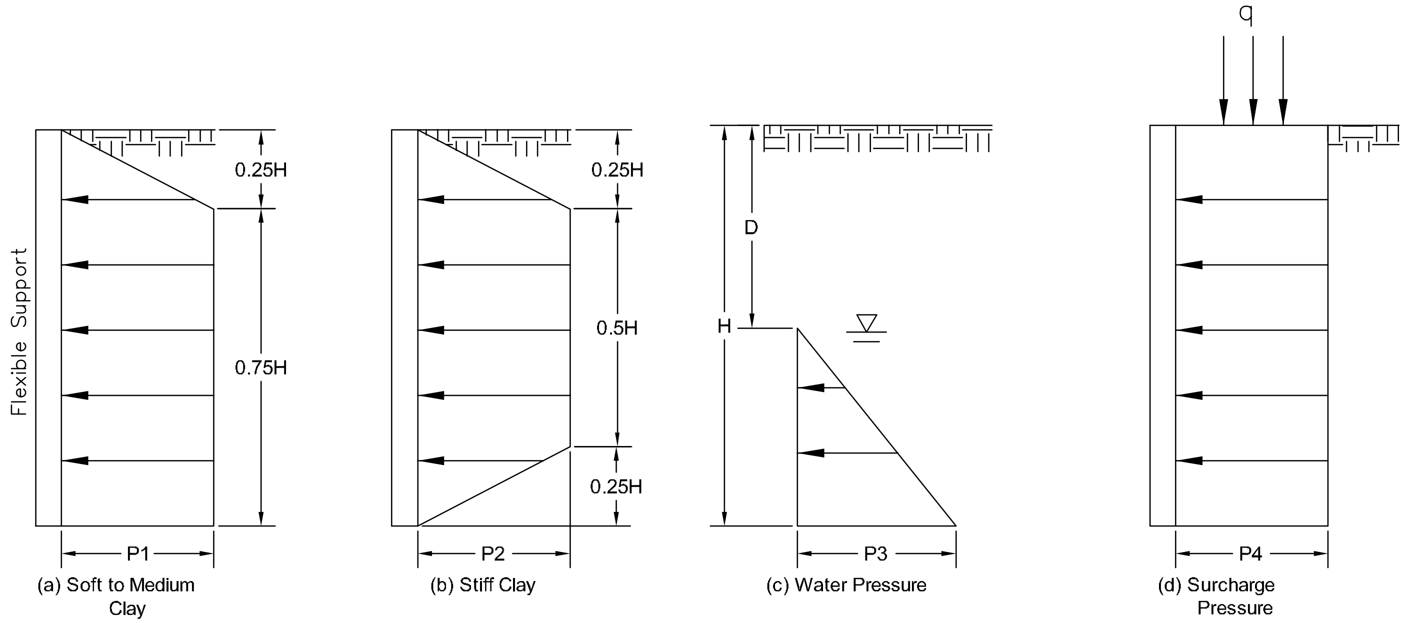
Notes:

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

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

ENGINEERING CORP.

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



Empirical Pressure Distributions

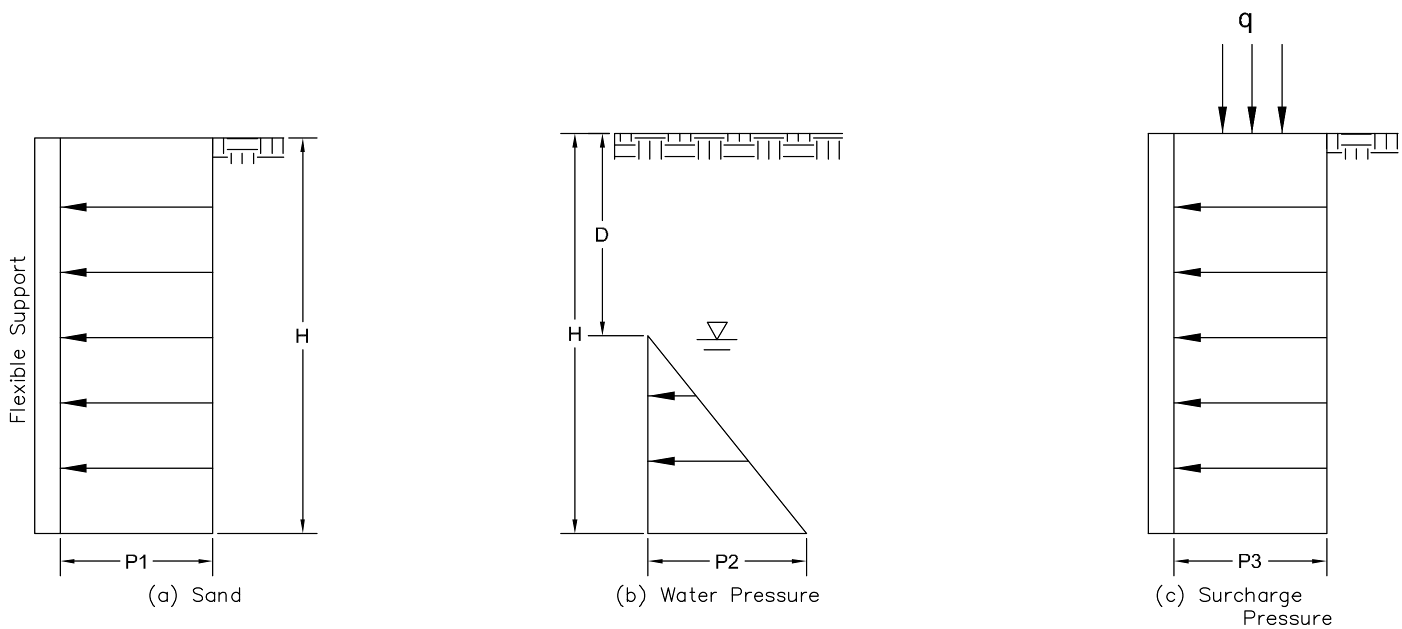
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H\text{-}4S_u$, psf
P2 = Lateral earth pressure = $0.2\gamma H$, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight 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.

ENGINEERING CORP.

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



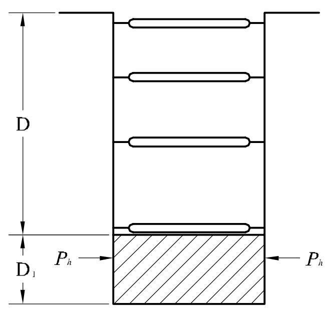
Empirical Pressure Distributions

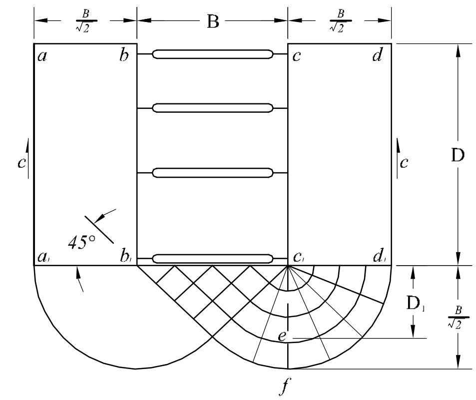
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65^{*}\gamma H K_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a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

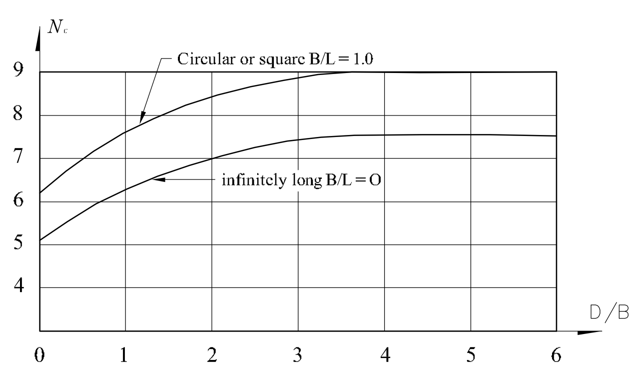
$$\textit{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \quad ; D_1 \geq 5\ ft.$$

Pressure on buried length, P_h:

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

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

where; B = width of excavation

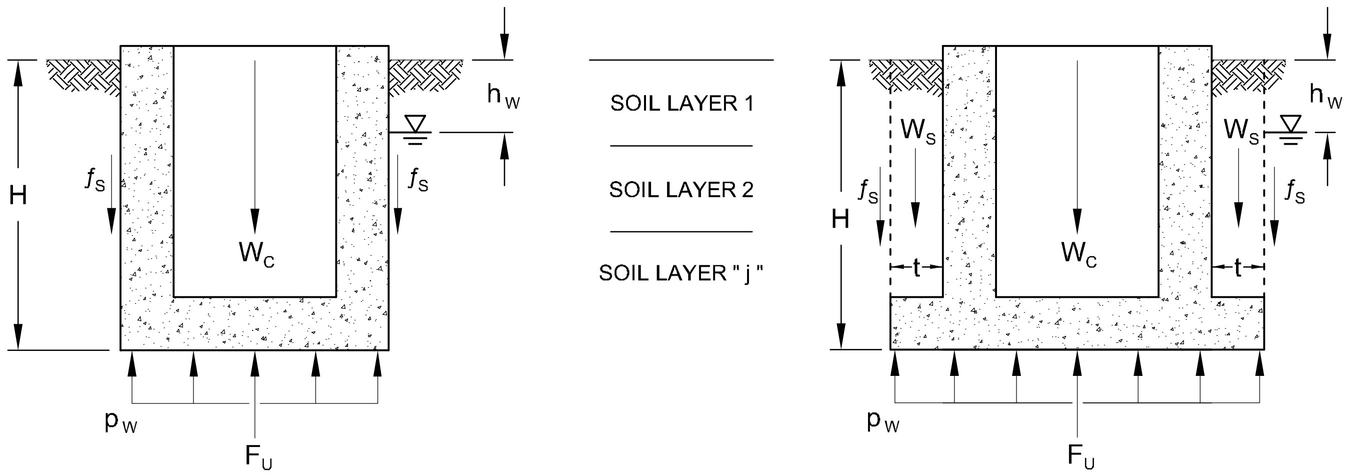


N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)

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

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

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

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

(b)

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

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

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

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

Where:

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

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

Reference:

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

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

APPENDIX E

Plates E-1 and E-2 Well Plugging Reports

Texas Department of License and Regulation

Water Well Driller/Pump Installer Program
P.O. Box 12157 Austin, Texas 78711 (512)463-7880 FAX (512)463-8616
Email address: water.well@license.state.tx.us

This form must be completed and filed with the department within 30 days following the plugging of the well.

PLUGGING REPORT

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER

Name	Address	City	State	Zip
City of Houston Geo Dept	611 Walker Floor 14	Houston	Tx	77002

2) WELL LOCATION

County	Physical Address	City	State	Zip
Harris	1200 Genesee	Houston	Tx	77019

3) Owner's Well No. 2 | Long. ° ' " | Lat. ° ' " | Grid # 65-13-9

4) Type of Well ☐ Water ☒ Monitor ☐ Injection ☐ De-Watering

5) N↑

Drill, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale gridded map available from Texas Natural Resource Information Service. The location of the well should be denoted within the grid by placing a corresponding dot in the square to the right. The legal description is optional.

B) HISTORICAL DATA ON WELL TO BE PLUGGED (if available)

6) Driller Edward Van Antwerp | License No. 3003M

7) Drilled 01/18/2013 | 8) Diameter of hole 4 inches | 9) Total depth of well 50 feet.

C. CURRENT PLUGGING DATA

10) Date well plugged 2/26/2013

12) Name of Driller/Pump Installer or Well Owner performing the plugging Edward Van Antwerp

License No. 3003M

13) CASING AND CEMENTING DATA RELATIVE TO THE PLUGGING OPERATIONS.

CASING LEFT IN WELL

DIAMETER (inches)	FROM (feet)	TO (feet)
2	0	50

CEMENT/BENTONITE PLUG(S) PLACED IN WELL

FROM (feet)	TO (feet)	SACKS
0	50	3

11) REMOVE ALL REMOVEABLE CASING

Please check box beside the method of plugging used

☒ Tremmie pipe cement from bottom to top.

☐ Tremmie pipe bentonite from bottom to 2 feet from surface, cement top 2 feet.

☐ Pour in 3/8 bentonite chips when standing water in well is less than 100 feet in depth, cement top 2 feet.

☐ Large diameter well filled with clay material from top to bottom.

COMMENTS

1) tried to pull well material
2) top piece of pvc broke off
3) grouted well material in place

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that all of the statements herein are true and correct. I understand that failure to complete items 1 through 13 will result in the report(s) being returned for completion and resubmitted.

Company or Individual's Name (type or print) Van and Sons Driling Service, Inc

Address 319 John Alber | City Houston | State Tx | Zip 77076

Signature [signature] Licensed Driller/Pump Installer | Date 3/5/2013 | Signature Apprentice | Date / /

PLATE E-1

Texas Department of License and Regulation

Water Well Driller/Pump Installer Program

P.O. Box 12157 Austin, Texas 78711 (512)463-7880 FAX (512)463-8616

Email address: water.well@license.state.tx.us

This form must be completed and filed with the department within 30 days following the plugging of the well.

PLUGGING REPORT

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER

Name	Address	City	State	Zip
City of Houston Geo Dept	611 Walker Floor 14	Houston	Tx	77002

2) WELL LOCATION

County	Physical Address	City	State	Zip
Harris	Tuam at Genesee	Houston	Tx	77019

3) Owner's Well No. 3 | Long. ° ' " | Lat. ° ' " | Grid # 65-13-9

4) Type of Well ☐ Water ☒ Monitor ☐ Injection ☐ De-Watering

5) N↑

Drill, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale gridded map available from Texas Natural Resource Information Service. The location of the well should be denoted within the grid by placing a corresponding dot in the square to the right. The legal description is optional.

B) HISTORICAL DATA ON WELL TO BE PLUGGED (if available)

6) Driller: Edward Van Antwerp | License No. 3003M

7) Drilled 01/22/2013 | 8) Diameter of hole 4 inches | 9) Total depth of well 50 feet.

C. CURRENT PLUGGING DATA

10) Date well plugged 2/26/2013

12) Name of Driller/Pump Installer or Well Owner performing the plugging: Edward Van Antwerp

License No. 3003M

13) CASING AND CEMENTING DATA RELATIVE TO THE PLUGGING OPERATIONS.

CASING LEFT IN WELL

DIAMETER (inches)	FROM (feet)	TO (feet)
2	0	45

CEMENT/BENTONITE PLUG(S) PLACED IN WELL

FROM (feet)	TO (feet)	SACKS
0	45	3

11) REMOVE ALL REMOVEABLE CASING

Please check box beside the method of plugging used

☒ Tremmie pipe cement from bottom to top.

☐ Tremmie pipe bentonite from bottom to 2 feet from surface, cement top 2 feet.

☐ Pour in 3/8 bentonite chips when standing water in well is less than 100 feet in depth, cement top 2 feet.

☐ Large diameter well filled with clay material from top to bottom.

COMMENTS

1) tried to pull well material

2) top piece of pvc broke off

3) grouted well material in place

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that all of the statements herein are true and correct. I understand that failure to complete items 1 through 13 will result in the report(s) being returned for completion and resubmitted.

Company or Individual's Name (type or print): Van and Sons Driling Service, Inc

Address 319 John Alber | City Houston | State Tx | Zip 77076

Signature [signature] Licensed Driller/Pump Installer | 3/5/2013 Date | Signature Apprentice | / / Date

AVILES ENGINEERING CORP.

APPENDIX F

Plates F-1 thru F-3 DARWin v3.0 Analysis Results

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Microsoft

Rigid Structural Design Module

Rigid Structural Design

Pavement Type	JPCP
Slab Thickness for Performance Period Traffic	7 in
Initial Serviceability	4.5
Terminal Serviceability	2.5
28-day Mean PCC Modulus of Rupture	600 psi
28-day Mean Elastic Modulus of Slab	3,360,000 psi
Mean Effective k-value	91 psi/in
Reliability Level	95 %
Overall Standard Deviation	0.35
Load Transfer Coefficient, J	3.2
Overall Drainage Coefficient, Cd	1.2
18-kip ESALs Over Initial Performance Period	1,076,142

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Microsoft

Rigid Structural Design Module

Rigid Structural Design

Pavement Type	JPCP
18-kip ESALs Over Initial Performance Period	4,084,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
28-day Mean PCC Modulus of Rupture	600 psi
28-day Mean Elastic Modulus of Slab	3,360,000 psi
Mean Effective k-value	91 psi/in
Reliability Level	95 %
Overall Standard Deviation	0.35
Load Transfer Coefficient, J	3.2
Overall Drainage Coefficient, Cd	1.2
Calculated Design Thickness	8.76 in

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Microsoft

Rigid Structural Design Module

Rigid Structural Design

Pavement Type	JPCP
Slab Thickness for Performance Period Traffic	9 in
Initial Serviceability	4.5
Terminal Serviceability	2.5
28-day Mean PCC Modulus of Rupture	600 psi
28-day Mean Elastic Modulus of Slab	3,360,000 psi
Mean Effective k-value	91 psi/in
Reliability Level	95 %
Overall Standard Deviation	0.35
Load Transfer Coefficient, J	3.2
Overall Drainage Coefficient, Cd	1.2
18-kip ESALs Over Initial Performance Period	4,834,811

PLATE F-3