AVILES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

69th STREET WASTEWATER TREATMENT PLANT IMPROVEMENTS HOUSTON, TEXAS

REPORTED TO

S&B INFRASTRUCTURE, LTD.

HOUSTON, TEXAS

by

Aviles Engineering Corporation

5790 Windfern

Houston, Texas 77041

713-895-7645

REPORT NO. G137-98

MARCH 1998

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AVILES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

69th STREET WASTEWATER
TREATMENT PLANT IMPROVEMENTS
HOUSTON, TEXAS

1.0 INTRODUCTION

The study reported herein is a Geotechnical Investigation of the subsurface conditions at the proposed Switchgear and Control Rooms, the roadway around the Sludge Plant, and the Oxygen line at the 69th Street Wastewater Treatment Plant (S&B Job No. U9380.65), located south of Clinton Drive, east of Turkey Street in Houston, Texas.

2.0 AUTHORIZATION

The investigation was authorized by Mr. Fred Flesch of S&B Infrastructure, Ltd. on February 3, 1998 upon acceptance of AEC's Proposal No. G98-01-14, dated February 2, 1998.

3.0 PURPOSE

The purpose of the investigation was to evaluate the subsurface soil conditions at the sites with particular reference to foundation support for the proposed Switchgear and Control Rooms; pavement design for roadway around the Sludge Plant; excavation trench safety, and bedding and backfill for storm sewer and oxygen line.

4.0 SUBSURFACE EXPLORATION

As requested, the subsurface exploration at the site consisted of drilling one undisturbed core boring to a depth of 50 feet at the proposed Switchgear and Control Rooms; 11 undisturbed core borings to a depth of 15 feet along the roadway around the Sludge Plant; and one undisturbed core boring to a depth of 15 feet below existing ground surface along the proposed alignment of the oxygen line. The approximate boring locations are shown on the attached Boring Location Plan, Plate 1.

The borings were of 3-inch nominal diameter. Undisturbed samples of cohesive soils were obtained from the borings by means of thin-wall, seamless steel Shelby Tube samplers. Granular soils were sampled with a two-inch split-barrel sampler in accordance with ASTM D-1586. Standard penetration resistance values were recorded as "Blows per Foot" and are shown on the boring logs. Undisturbed samples were extruded mechanically from the core barrels in the field, wrapped in aluminum foil, and sealed in plastic bags to reduce moisture loss and disturbance. The samples and cores were then placed in core boxes and transported to our laboratory for testing and further study.

5.0 LABORATORY TESTING

The samples were examined and classified in the laboratory by a geotechnical engineer. Laboratory tests were performed on selected soil samples to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. The test results are summarized on the boring logs on Plates 2 through 14. A key to the soil symbols in the boring logs is presented on Plate 15. The soil boring log profiles along the roadway at the Sludge Plant are included on Plates 16 and 17.

Strength properties of the soils were determined by unconfined compression and triaxial unconsolidated-undrained tests performed on selected undisturbed samples; these values are plotted on the boring logs as solid circles and triangles. Water content and dry unit weight were determined for each unconfined compression test sample. Water content determinations were also made on other samples to define the moisture profile at each boring location. Atterberg limits and percent finer than No. 200 sieve tests were also performed on selected samples to establish index properties and aid in the proper classification of the subsurface soils. The tests were performed in accordance with applicable ASTM Standards summarized on Plate 18.

6.0 SUBSURFACE CONDITIONS

Switchgear and Control Room (Boring B-1)

The boring log (Plate 2) indicates that the subsurface soils generally consist of fill, medium dense silty sand, hard sandy lean clay, and hard clay. The medium dense silty sand from about 2 to 22 feet below existing grade is probably backfill for the existing lift station foundation.

Results of our laboratory tests indicate that the cohesive soils are of moderately high to high plasticity with liquid limits (LL) ranging from 45 to 59 percent, and plasticity indices (PI) ranging from 30 to 38 percent. In accordance with the Unified Soil Classification System, these soils are classified as "CL" and "CH" type soils. "CL" soils generally do not undergo significant volume changes; however, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes due to seasonal changes in their moisture contents.

Groundwater was encountered at a depth of about 12 feet in the boring during our field investigation and subsequently rose to a depth of about 4 feet after 18 hours.

Sludge Plant Roadway (Borings B-2 through B-12)

The boring logs (Plates 3 through 13) indicate that the subsurface soils generally consist of fill, medium dense clayey sand, stiff to hard sandy lean clays, very loose to medium dense silty sand, loose to medium dense poorly graded sand, and stiff to hard clays.

Results of our laboratory tests indicate that the cohesive soils are of low to high plasticity with LL ranging from 23 to 56 percent, and PI ranging from 9 to 38 percent.

Groundwater was encountered at depths ranging from about 5 to 13 feet in Borings B-3, B-6, B-7, B-8, B-9, and B-11 during our field investigation. Groundwater was measured at depths ranging from about 6½ to 11½ feet, about 30 minutes after drilling in Borings B-3, B-4, B-6, B-7, B-8, B-9, and B-11. Groundwater was not encountered in the remaining borings.

Oxygen Line (Boring B-13)

The boring log (Plate 14) indicates that the subsurface soils generally consist of about 4 feet of fill, (sandy clay and medium dense silty sand) underlain by firm to hard sandy lean clay and medium dense silty sand to the termination depth of the boring.

Results of our laboratory tests indicate that the cohesive soils are of moderately high to high plasticity with liquid limit about 41 percent, and plasticity index (PI) about 28 percent.

Groundwater was encountered at a depth of about 13 feet in the boring during our field investigation. Groundwater subsequently dropped to a depth of about 13.5 feet after a half hour in the boring.

7.0 SUBSURFACE VARIATIONS

The information contained in this report summarizes the conditions encountered on the dates the borings were drilled. The depth to the static groundwater table and subsurface soil moisture contents will vary with seasonal and environmental variations such as frequency and magnitude of rainfall and future construction activities that may alter the surface and drainage characteristics of the site. In cohesive soils, fluctuations in groundwater depth occur over a longer period than in granular soils.

An accurate evaluation of the steady groundwater table requires long term measurements of monitoring wells and/or piezometers. It is not possible to accurately predict the level of groundwater that might occur based upon short-term exploration. The installation of monitoring wells/piezometers was beyond the scope of this investigation. We recommend that AEC be notified immediately if the groundwater depth changes significantly from that mentioned in this report.

8.0 DESIGN CRITERIA AND RECOMMENDATIONS

The information provided to us indicates that the 69th Wastewater Treatment Plant improvements will consist of (1) a proposed Switchgear and Control Room, (2) roadway around the Sludge Plant, and (3) installation of the oxygen line and storm sewer.

8.1 Switchgear and Control Room Foundations

8.1.1 Foundation Type and Depth

According to the drawings provided to us, the proposed Switchgear and Control Room will be located adjacent to the existing Lift Station which is founded about 42 feet below existing grade. Although the proposed Switchgear and Control Room building will be lightly loaded, the magnitude of the machinery dynamic loads are not known. Also, the adequacy of the existing lift station walls has not been evaluated. Mr. Fred Flesch of S&B Infrastructure, Ltd., requested foundation recommendations for drilled footings supported at or below the bottom of the lift station. These recommendations are presented below. In addition, we have presented recommendations for spread footings as an option.

8.1.2 Straight Drilled Shaft Footings

The Switchgear and Control Room can be supported on drilled footings as proposed. The silty sand layer from 2 to 22 feet below existing grade and the sand pockets and silt seams encountered in the sandy lean clays and clays indicate a potential for sloughing or caving-in of the side walls during construction. We therefore recommend the use of straight drilled shaft footings. Casing or slurry will be required to mitigate potential sloughing and caving of the excavation sidewalls.

We used the Reese and O'Neill method for drill shaft analysis. In this analysis, skin friction in the top 5 feet of soil is neglected due to moisture fluctuation and construction disturbance. The total allowable compressive axial bearing capacity of a drilled shaft is the sum of the allowable end bearing and allowable skin friction capacity, which are obtained by dividing their ultimate capacities with a factor of safety (F.S. = 2.0 for skin friction, F.S. = 3.0 for end bearing). The total allowable compressive axial capacity curves for 18 and 24-inch diameter drilled shafts are presented on Plate 19.

8.1.3 Spread Footings

Alternatively, the structural loads can also be supported on shallow spread footings founded 4 feet below lowest adjacent existing grade into the medium dense silty sand. We recommend net allowable bearing pressures of 2,000 for dead loads and 3,000 for total loads. Lateral loads imposed on the existing lift station wall should be estimated and the potential impact on the wall evaluated if this option is selected.

Footing excavations should be checked by qualified geotechnical professionals to evaluate the adequacy of the foundation soils. If footing excavations expose loose soils, we recommend they be compacted to at least 95 percent of the ASTM D-698 maximum dry density. Any ponded water in drilled footings should be removed prior to concrete placement.

8.1.4 Drilled Shaft Construction

Groundwater was encountered in the boring at a depth of about 4 feet below existing grade during our field investigation. Footing excavations should be dewatered or the concrete should be placed using the tremie method. We recommend that guidelines presented in "Drilled Shafts: Construction Procedures and Design Methods" by Reese and O'Neill (U.S. Department of Transportation, Federal Highway Administration Publication No. FHWA-HI-88-042, 1988) be followed during construction.

Each footing excavation should be monitored by a qualified Owner's Representative prior to placing concrete, to check that (a) the footing excavation has been constructed to the specified dimensions at the recommended depth and formation, (b) excessive cuttings and any soft-compressible materials and ponded water have been removed from the bottom of the excavation.

Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the state of stress and possible caving in the foundation soils. No footings should be poured without the prior approval of the Owner's Representative.

8.1.5 Estimated Foundation Settlement

Based on the soil conditions encountered and the anticipated structural loads, we estimate that footings designed and constructed as recommended will experience total consolidation settlements on the order of 1 inch, and differential settlements in the order of 0.5 inch.

8.1.6 Estimated Potential Vertical Rise

No moisture-induced vertical movements (also known as Potential Vertical Rise or PVR) at the existing ground surface are anticipated at this location since the upper 22 feet of soil is silty sand.

8.1.7 Floor Slab

The slab-on-grade floor can be supported on the medium dense to dense silty sand. Because of the possibility that this soil is old fill, we recommend that a minimum 1 feet of the existing silty sand be removed from the floor slab area and the exposed surface be proof-rolled in accordance with Item 216 of the "Standard Specifications for Construction of Highways, Streets, and Bridges", published by the Texas Department of Transportation (1993 Edition). The intent of the proof-rolling is to identify soft or weak areas; care should be taken to limit the number of passes as this could result in overstressing the existing lift station wall, and wetting the subgrade soils due to the shallow groundwater. Any soft or weak areas should be removed and replaced either with select fill. We recommend that subgrade preparation and fill placement extend at least 5 feet beyond the edge of the new building footprint. The excavated soils may be used as fill provided they are free of deleterious materials. The fill should be placed in loose lifts not exceeding 8 inches in thickness. If hand-tamping equipment is used, the loose lift thickness should be reduced to no more than 4 inches. The fill should be compacted to a minimum 95% of the ASTM D-698 maximum dry unit weight and at a moisture content near optimum.

The concrete slab may be doweled into the grade beams provided the connections are designed to withstand the order of magnitude of differential movements estimated in this report. A vapor barrier consisting of 6 mil polyethylene may be placed between the select fill and the concrete slab.

8.1.8 Site Grading

The following measures should be undertaken to reduce the possibility of changes in the moisture content of the soils under the floor slab.

1. Design final grading to provide site drainage away from structures.
2. Prevent ponding of water or excessive drying of excavated surfaces.
3. Design roof drains to discharge into paved areas, or, if discharged onto ground, at least 6 feet away from structures.
4. To the extent possible, do not place utility lines beneath the structure.
5. Locate landscaping away from structures and pavements.

8.2 Oxygen Line and Storm Sewer Excavation

The proposed 12-inch diameter oxygen line, and 42-inch (or less) diameter storm sewer line will most likely be installed in open cut trenches.

8.2.1 Geotechnical Parameters

A summary of the recommended geotechnical parameters is presented in Table 1. These values are based on the results of field and laboratory test data as well as our experience. It should also be noted that because of the nature of the soil stratigraphy, parameters at locations away from the borings may vary substantially from values reported in the table.

TABLE 1 SUMMARY OF SELECTED GEOTECHNICAL PARAMETERS

PARAMETER	COHESIVE SOILS (SANDY LEAN CLAY & CLAY)	COHESIONLESS SOILS (SILTY SAND, CLAYEY SAND, AND SAND)
Saturated Unit Weight, γ (pcf)	125	122
Buoyant Unit Weight, γ' (pcf)	62.5	60
Angle of Internal Friction, δ (deg.)	0	30
Undrained Shear Strength, C_u (psf)	1,000	0
Coef. Of Active Earth Pressure, K_a	0.5	0.33
Coef. Of Passive Earth Pressure, K_p	1.0	3.0

Note: Parameters at a specific location may vary from the value reported in this table and consideration should be given to reducing shear strength values in design to account for soil micro structure and/or secondary features, where appropriate.

8.2.2 Trench Stability

The contractor should be responsible for all safety during construction. The recommendations presented herein are intended to guide the contractor in his design. In areas where open cut trenches are to be used, all support systems should be designed to meet OSHA Guidelines. OSHA design methods are based on a relationship between soil type, unconfined compressive strength, groundwater, and prior soil disturbance. The OSHA soil classification for excavating trench and shoring is presented in Table 2.

TABLE 2 OSHA SOIL CLASSIFICATION FOR EXCAVATING, TRENCHING AND SHORING

DEPTH (ft)	B-2	B-3	B-4	B-5	B-6	B-7	B-8	B-9	B-10	B-11	B-12	B-13
0-2	C	C	B	C	B	B	C	C	C	C	C	B
2-4	C	B	B	C	B	B	B	C	C	C	B	C
4-6	B	B	C	B	B	C	C	C	C	C	B	B
6-8	B	B	C	B	B	B	C	C	C	C	B	B
8-10	B	B	C	B	B	C	C	C	B	C	B	B
10-12	B	B	C	B	B	C	C	C	B	C	B	B
12-14	B	C	C	B	B	C	C	C	B	C	B	C
14-15	B	C	B	B	B	B	C	C	B	C	B	C

In the zones of cohesive soils, a Critical Height (used to determine maximum depth of open cut at given side slopes) may be calculated based on the cohesion of the soil. Critical Heights for various slopes and cohesion values are illustrated on Plate 20. Critical Height is defined as the height a slope will stand unsupported for a short time.

Several cautions should be exercised in the use of Critical Height applications:

1 No more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or unsuitable cohesive soils are encountered within the excavation depth.

2. If the soil at the surface is dry to the point where tension cracks occur, any water in the 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 wale 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, i.e. where 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 Occupational Safety and Health Administration (OSHA) requirements. Plate 21 presents the steepest allowable slopes in Soil Types A, B, and C for excavations less than 20 feet.

If limited space is available for the required open trench side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated in Plate 22.

The allowable side slope for temporary excavations in cohesive soils is 1(V): 1(H) or flatter, depending on the soil conditions during construction. Bracing schemes and methods for calculating bracing stress are discussed below. The contractor should be responsible for designing, constructing and maintaining safe and stable excavations.

8.2.3 Computation of Bracing Pressures

A logarithmic spiral method is used for calculating earth pressure against bracing for open cuts. This concept is illustrated on Plate 23. For practical application the pressure can be obtained by the following relationship:

$$P_a = 1.1\ P_A$$

Where:	P_a	=	Maximum Pressure (psf)
	1.1	=	Dimensionless Coefficient
	P_A	=	Active Pressure (psf)
	P_A	=	$K_a\ \gamma\ D$
Where:	γ	=	Soil Density (pcf)
	D	=	Depth of Soil (ft)
	K_a	=	Coefficient of Active Earth Pressure (Plate 24)

Then, the design load for struts in open cut in sand is obtained by the following relationship:

$$L = 0.8\ P_a \cos \delta$$

Where:	L	=	Design Load (psf)
	0.8	=	Dimensionless Coefficient
	$\tan \delta$	=	2/3 tan (angle of wall friction)
example:	if ϕ	=	41°, δ = 30.2° (dense to very dense)
example:	if ϕ	=	30°, δ = 21.1° (loose to medium)

The above parameters should be used in calculating bracing pressure within zones of clay as previously described. Recommended pressure distribution for the design of struts in open cuts for clay and sand are illustrated on Plates 25 and 26. The distribution assumes level backfill and no surcharge loads adjacent to or near the walls. Due to the possibility of the groundwater table rising is above the bottom of excavation, we recommend that hydrostatic pressure be included in the design.

8.2.4 Groundwater Control

Dewatering and groundwater control should be the Contractor's responsibilities. The comments and suggestions in this report regarding excavations and groundwater are for informational purposes only, and may be used to review the Contractor's proposed construction procedures. The following dewatering criteria is presented for the contractor's guidance.

8.2.5 Shallow Open Cuts

Seepage in the clay will probably be low. Seepage influx will be primarily from sand and silt seams and layers, and fissures. Gravity drainage with sumps can be effective in removing seepage water in these clay zones. Greater seepage and slope stability problems may be experienced in sands, which are at or below the observed groundwater depth. When groundwater cannot be handled by sumps, dewatering procedures such as well points or wells should be used in advance of excavation.

Generally, the groundwater depth should be lowered at least 3 feet below the excavation bottom to be able to work on a firm surface. Extended and/or extensive dewatering can result in settlement of existing structures in the vicinity. If well points are installed, the contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity.

In open cuts, the possibility of the bottom heave must be considered, because of the removal of the weight of excavated soil. In clays and sandy lean clays, heave normally does not occur unless the ratio of critical height to depth of cut approaches one. In silty clay, heave should not occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. This can be mitigated if a well point system is used to dewater the area.

8.2.6 Pipe Bedding and Backfill

Bedding and backfill for sewers should be constructed in accordance with the City of Houston, Department of Public Works, Standard Construction Specifications (COH Specs.) For Wastewater Collection System, Waterlines, Storm Draining, and Street Paving, Sections 02316, 02317 and 02320. Based on the our borings, we anticipate the soils within the bedding zone for the oxygen line will primarily consist of silty sand and sandy lean clay fill and firm to hard sandy lean clay; the soils within the bedding zone for sludge plant roadway will primarily consist of fill, medium dense silty sand, stiff to hard sandy lean clays, very loose to medium dense silty sand, loose to medium dense poorly graded sand, and stiff to hard fat clays. Accordingly, Drawing No. 02317-01 should be used for bedding and trench zone backfill for dry or wet stable trench.

The backfill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of the ASTM D-698 maximum dry density and at a moisture content within 2% of optimum moisture content.

8.2.7 Soil Stiffness Categories

Soil stiffness categories for pipe design are presented in Table 3. These values are based on the results of field and laboratory test data as well as our experience.

TABLE 3 SOIL STIFFNESS CATEGORIES[1] FOR PIPE DESIGN

BORING NO.	SOIL STIFFNESS CATEGORIES	SOIL DESCRIPTION	MINIMUM TYPICAL E'n, psi (long-term)
B-2	D	varies	600
B-3	D	varies	600
B-4	E	varies	300
B-5	D	sandy lean clays	600
B-6	C	clays	1,000
B-7	E	varies	300
B-8	E	varies	300
B-9	E	varies	300
B-10	D	varies	600
B-11	E	varies	300
B-12	D	sandy lean clays	600
B-13	D	varies	600

8.3 Pavement Section

8.3.1 Rigid Pavement Design

The proposed pavement surface will be Portland Cement Concrete. The design procedure for determining concrete pavement slab thickness for rigid pavements is based on the 1993 AASHTO Guide for Design of Pavement Structures which was originally developed from the AASHTO Road Test. The following parameters were used in the design.

1) Design 18-kips Equivalent Single-Axle Load (ESAL) for 20 year, ESAL = 1,000,000
2) Modulus of Elasticity of Concrete, $E_c = 3.42 \times 10^6$ psi
3) Resilient Modulus of Subgrade Soil, M_R = 2,000 psi
4) Mean Concrete Modulus of Rupture, S'_c = 600 psi (at 28 days)
5) Joint Transfer Coefficient, J = 3.0
6) Drainage Coefficient C_d = 1.0
7) Overall Standard Deviation, S_o = 0.35
8) Reliability, R = 95%
9) Initial Serviceability p_0 = 4.2
10) Terminal Serviceability p_t = 2.5

Based on the results obtained using the AASHTOWARE™ DARWin™ 3.0 program, we recommend the following concrete pavement section:

TABLE 4 RECOMMENDED PAVEMENT SECTION

PAVEMENT SECTION	SECTION THICKNESS (inches)
Portland Cement Concrete	8
Option 1: 4% Lime + 8% Fly Ash Treated Subgrade	6
Option 2: 6% Cement Treated Subgrade	6

8.3.2 Subgrade Preparation

Initially, the existing fill should be removed and stockpiled and the exposed surface proof-rolled as recommended for the floor slab. Any weak or dry soils encountered should be replaced with select fill as recommended earlier. The pavement subgrade soils exhibit moderate plasticity. We recommend that the top 6 inches of sandy lean clay or clayey sand subgrade be stabilized with at least 4 percent hydrated lime and 8 percent fly ash (or 4 percent cement); and the top 6 inches fat clay, where they exist (near Boring B-4), be stabilized with at least 6 percent hydrated lime. The actual percentage of lime and fly ash, or cement should be confirmed by laboratory testing prior to construction.

Lime and fly ash, or cement stabilized soils should be placed in 8-inch loose lifts and compacted to at least 95 percent of the ASTM D-698 maximum dry density at a moisture content within 2 percent of optimum. Lime and fly ash stabilization should be in accordance with TxDOT Item 260. Cement stabilization should be in accordance with TxDOT Item 275.

8.3.3 Reinforcing Steel Requirements

The required longitudinal and transverse rebars for the concrete pavement were computed using:

$$A_s = \frac{FLW}{2f_s} \qquad \frac{1.8\,(25')\,100.5}{2\,(45000)} = .05\ \text{in}^2/\text{ft}$$

30" = .08 in²/ft

where:
- A_s = Cross-sectional area of steel per foot width of slab, sq. in.,
- F = Coefficient of resistance between slab and subgrade = 1.8,
- L = Distance between free transverse joints or free longitudinal edges, feet,
- W = Weight of pavement per foot width of slab, lb/sq. Ft., and
- f_s = Allowable working stress in the steel, psi; use 45,000 psi for Grade 60 steel.

Based on the above, the required reinforcing steel for the concrete pavement is as follows:

TABLE 5 REINFORCEMENT STEEL BAR SIZE AND SPACING

PAVEMENT THICKNESS (INCHES)	SLAB LENGTH (feet)	REINFORCEMENT BAR SIZE AND SPACING (CENTER TO CENTER) (inches)
8	80	#4 @ 15.5

The size and spacing of dowels to be provided at expansion joints for the pavement is presented below:

TABLE 6 DOWEL SIZE AND SPACING

PAVEMENT THICKNESS (inches)	DOWEL SIZE AND SPACING (inches)		
	DIAMETER	LENGTH	SPACING
8	1	18	12

9.0 SITE PREPARATION

To reduce construction problems which may develop if attempts are made to work the surface materials following prolonged periods of rainfall, we recommend that prior to starting any work at the site, proper construction drainage be provided to maintain a relatively dry construction site.

10.0 CONSTRUCTION CONSIDERATIONS

10.1 Trench Safety

The contractor should be responsible for the safety of the workers and other personnel, as well as equipment. Trenches should be adequately shored or provided with adequate side slopes according to the OSHA requirements. In addition, design, construction, and maintenance of the trenching and shoring should be performed by qualified personnel under experienced supervision.

10.2 Hazardous Materials

The field investigation did not indicate the presence of hazardous materials. However, the possibility of existence of hazardous materials at the proposed site should not be discounted. If signs of hazardous materials are observed (fumes, odors, discolored soils or other materials, etc.), the person in-charge should be notified immediately and work should be halted until the site has been remediated, or verified as safe by a qualified hazardous waste specialist before resuming construction.

11.0 INFLUENCE OF EXCAVATION ON EXISTING STRUCTURES

11.1 Protection of Existing Structures

The contractor should be responsible for protecting any existing structures in the vicinity which could be adversely affected by the proposed construction. If excavation is performed adjacent to or near existing structures, the excavation should be adequately shored to reduce lateral soil movements that could result in settlement or other distress to the existing structures. The excavation should not extend below the foundation elevation of existing structures, unless adequate shoring is provided.

11.2 Monitoring

Structures located close to the proposed structures and alignments should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded. This can be accomplished by conducting a pre-construction survey, taking photographs and/or video film, and documenting existing elevations, cracks, settlements, and other existing distress in the structures. The monitoring should include establishment of elevation monitor stations, crack gauges, and inclinometers, as required. The monitoring should be performed before, periodically during, and after construction. The data should be reviewed by qualified engineers in a timely manner to evaluate the impact on existing structures and develop plans to mitigate the impact, should it be necessary.

12.0 DESIGN REVIEW

AEC should be retained to review the design and construction plans and specifications prior to release to make certain that the geotechnical recommendations and design criteria presented herein have been properly interpreted.

13.0 CONSTRUCTION MONITORING

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

14.0 GENERAL

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are a true representation of the soils encountered at the specific boring locations on the dates of drilling and represent the stratigraphy as encountered during drilling of the subject property. 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.

15.0 CLOSING REMARKS

AEC appreciates the opportunity to be of service on this project and looks forward to our continuing association on this and future projects. We are interested in providing materials testing services during the construction phase of this project.

AVILES ENGINEERING CORPORATION

Shou Ting Hu, M.S.C.E., E.I.T.
Project Manager

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March 26, 1998

STH/AAJ

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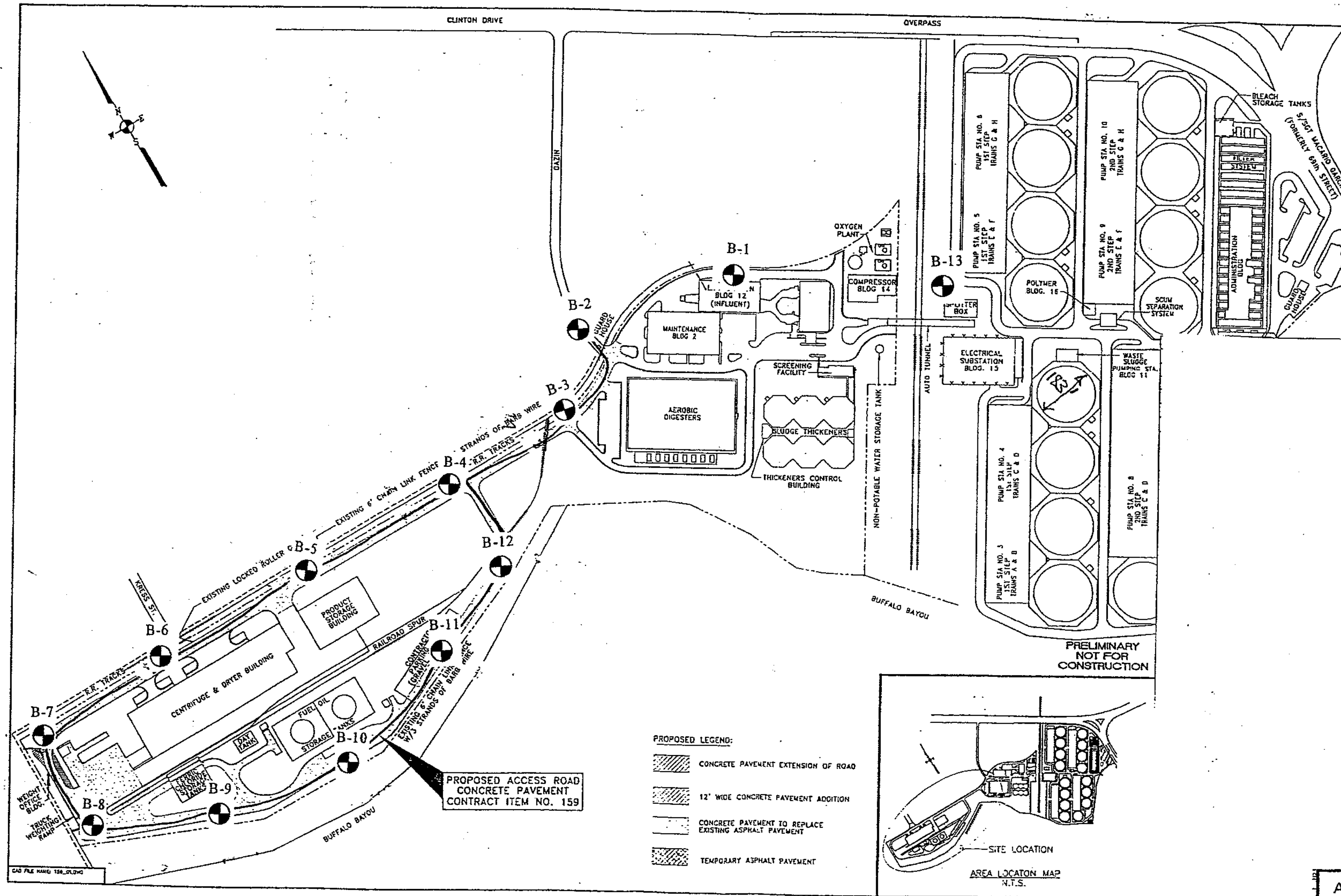
AVILES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS



AVILES ENGINEERING CORPORATION	G137-98
BORING LOCATION PLAN 69th STREET WWTP IMPROVEMENTS HOUSTON, TEXAS	
MARCH 1998	PLATE 1

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **69th STREET WWTP IMPROVEMENTS** BORING **B-1**

DATE **FEBRUARY 23, 1998** TYPE **3" CORE** LOCATION **SEE PLAN**

Shear strength, TSF: ○ Pocket Penetrometer; △ Confined Compression; ● Unconfined Compression; □ Torvane

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Pavement, 2-inch asphalt, 5.5-inch crushed limestone base		20					
	Fill, tan silty sand (SM)					41		
4	Medium dense to dense tan silty sand (SM), possible fill w/ trace calcareous nodules	13	14					
		23	12					
	- gray	25	17					
8	- light gray and tan	32	13			14		
12								
		42	22					
16								
		28	24			7		
20								
24	Hard tan and light gray sandy lean clay (CL) w/ sand pockets, ferrous & calcareous nodules		18	112			45	14
28	Hard tan and light gray clay (CH) w/ silt seams, calcareous & ferrous nodules		17	114	2.545		51	21
32	- reddish brown and light gray		19	112	2.58			
36								
40			24				59	21

BORING DRILLED TO 50 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 12 FEET WHILE DRILLING

WATER LEVEL AT 4 FEET AFTER 18 HOURS

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS BORING B-1

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

SHEAR STRENGTH, TSF: ○ POCKET PENETROMETER; △ CONFINED COMPRESSION; ● UNCONFINED COMPRESSION; □ TORVANE (scale 0.5, 1, 1.5, 2)

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
42										
					21	107				
46										
			- w/ silt pockets		22					
50			Bottom @ 50'							
54										
58										
62										
66										
70										
74										
78										
82										

BORING DRILLED TO 50 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 12 FEET WHILE DRILLING

WATER LEVEL AT 4 FEET AFTER 18 HOURS

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS

BORING B-2

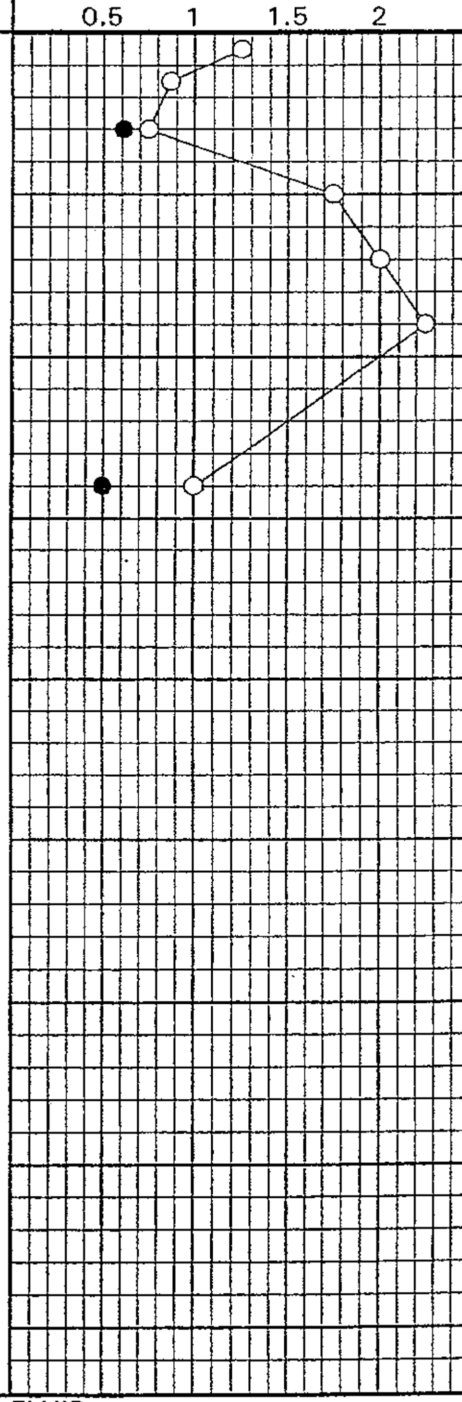
DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

Elevation:

DEPTH IN FEET	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0–4	Fill, tan clayey sand (SC) w/ roots, gravel, ferrous nodules		12			24	13
			13		37		
			15	111			
4–13	Fill, very stiff to hard tan and light gray sandy lean clay (CL) w/ sand pockets and seams, ferrous nodules		15			27	12
			18				
			12				
13–15	Stiff tan and light gray sandy lean clay (CL) w/ trace calcareous & ferrous nodules, silty sand seams		17	110			
	Bottom @ 15'						

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



Depth scale: 0, 4, 8, 12, 16, 20, 24, 28, 32, 36, 40

BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT ____ FEET WHILE DRILLING
WATER LEVEL AT ____ FEET AFTER ____ HOURS
DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

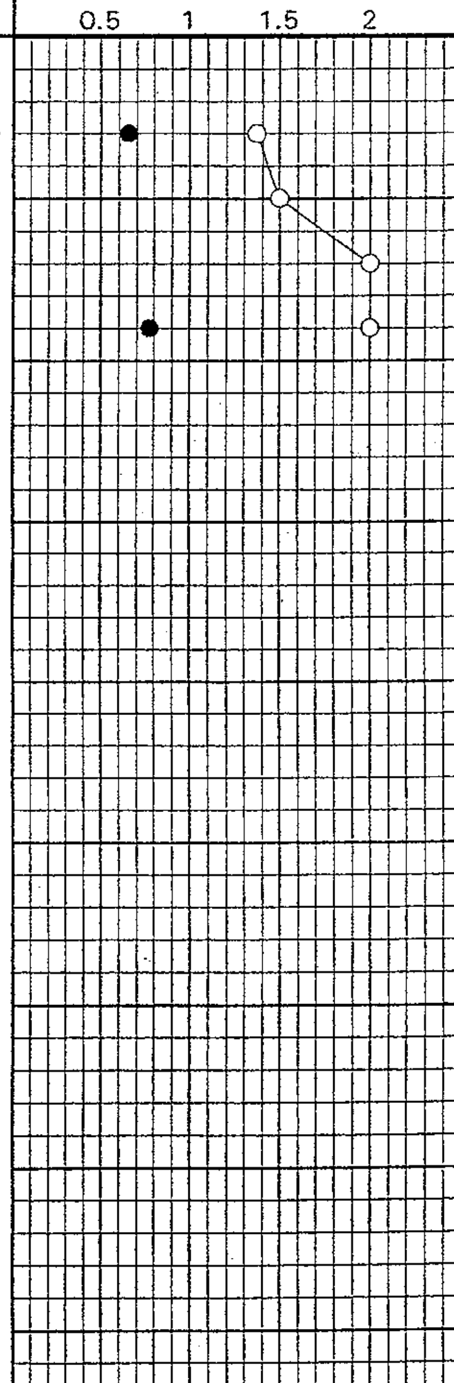
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS BORING B-3

DATE FEBRUARY 25, 1998 TYPE 3" CORE LOCATION SEE PLAN

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Fill, tan and light gray clayey sand (SC) w/ roots, gravel, ferrous nodules		17			24	14
	Very stiff tan and light gray sandy lean clay (CL) w/ ferrous nodules		16	110			
4			16				
	- w/ sand pockets and seams		15			23	14
8	- caved @ 9.5 feet		18	110			
12	Loose light gray silty sand (SM)	9	17				
16	Bottom @ 15'						

SHEAR STRENGTH, TSF
○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 6 FEET WHILE DRILLING
WATER LEVEL AT 9.5 FEET AFTER 0.5 HOURS
DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. [illegible]137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS   BORING B-4

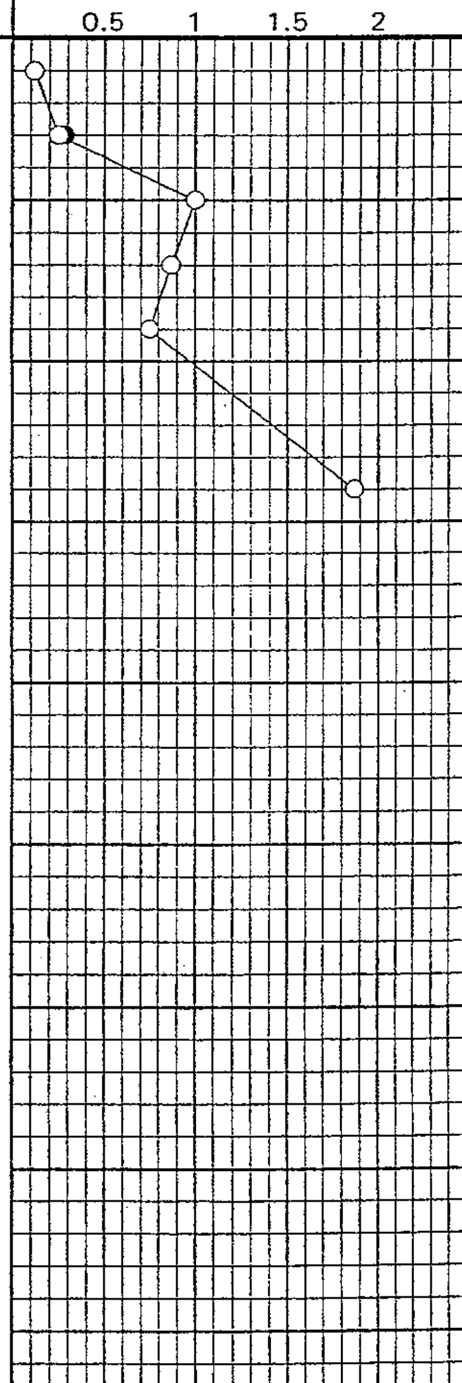
DATE FEBRUARY 25, 1998   TYPE 3" CORE   LOCATION SEE PLAN

Elevation:

DEPTH IN FEET	SYMBOL / CORES	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0–2	Fill / core	Fill, soft light gray and reddish brown fat clay (CH) w/ roots, sand pockets, ferrous nodules		27			56	16
2–6	Fill / core	Fill, firm light gray and brown sandy lean clay (CL), w/ sand seams		28	96			
				19				
6–8	Fill / core	Fill, light gray and tan clayey sand (SC) w/ ferrous nodules		18				
8–10	Fill / core	Fill, stiff light gray and tan sandy lean clay (CL) w/ clayey sand seams, wood		22			26	15
10–13	Fill	Light gray and tan silty sand (SM) - caved @ 10 feet						
13–15	Clay / core	Very stiff brown and light gray sandy lean clay (CL) w/ silt stone, calcareous nodules		12			32	16
15		Bottom @ 15'						

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



Depth scale: 0, 4, 8, 12, 16, 20, 24, 28, 32, 36, 40

BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT ____ FEET WHILE DRILLING

WATER LEVEL AT 10 FEET AFTER 0.5 HOURS

DRILLED BY JH DRILLING   CHECKED BY SH   LOGGED BY JH DRILLING

PLATE 5

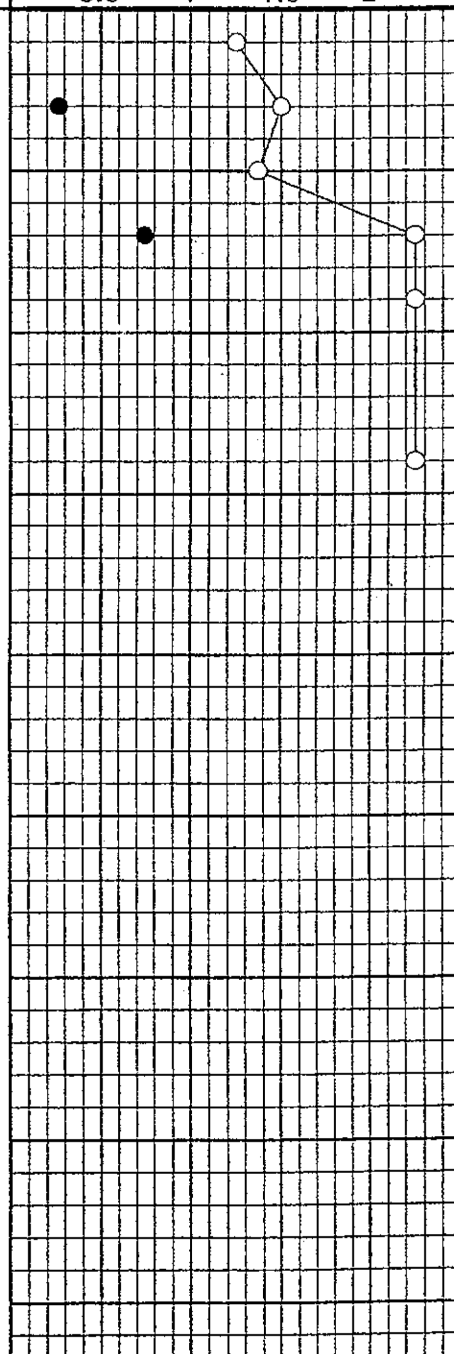
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **69th STREET WWTP IMPROVEMENTS** BORING **B-5**

DATE **FEBRUARY 25, 1998** TYPE **3" CORE** LOCATION **SEE PLAN**

SHEAR STRENGTH, TSF: ○ POCKET PENETROMETER; △ CONFINED COMPRESSION; ● UNCONFINED COMPRESSION; □ TORVANE (scale 0.5, 1, 1.5, 2)

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0			Fill, very stiff light gray, tan and reddish brown sandy lean clay (CL) w/ roots, sand pockets, ferrous nodules		31				
					36	87		28	12
4			- w/ sand seams		12				
			Hard light gray and brown sandy lean clay (CL) w/ ferrous & calcareous nodules		17	111		38	13
8					15				
12			- light gray and tan		16				
16			Bottom @ 15'						
20									
24									
28									
32									
36									
40									



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT ____ FEET WHILE DRILLING

WATER LEVEL AT ____ FEET AFTER ____ HOURS

DRILLED BY **JH DRILLING** CHECKED BY **SH** LOGGED BY **JH DRILLING**

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

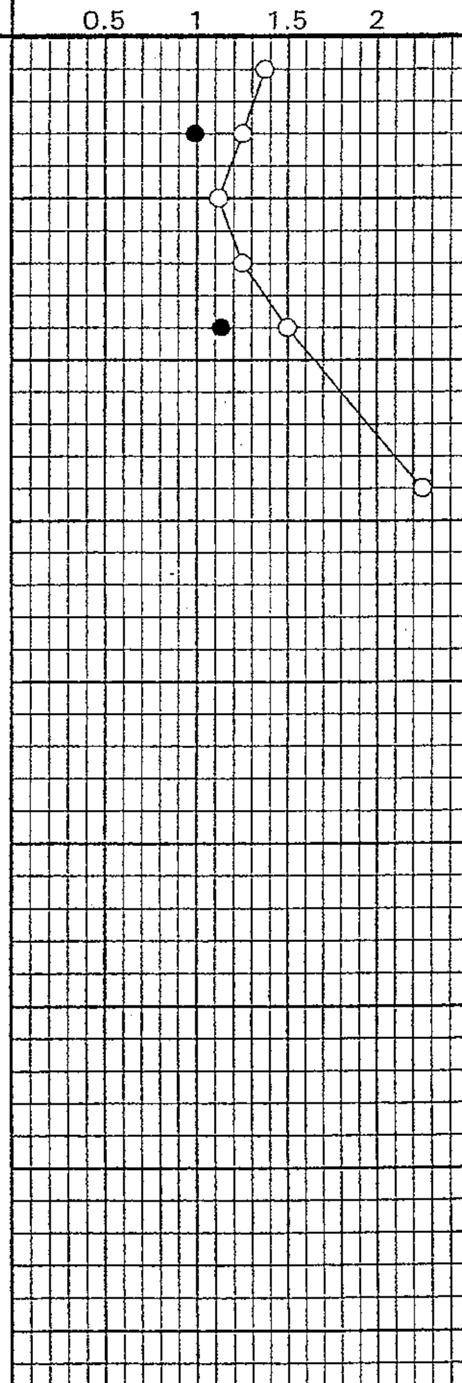
PROJECT: 69th STREET WWTP IMPROVEMENTS BORING B-6

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Very stiff light gray and tan sandy lean clay (CL) w/ ferrous nodules		17			39	13
			17	112			
4	- w/ sand pockets		16				
	- w/ sand seams		18			41	17
8			23	103			
	- w/ silty sand seams @ 12 to 13 feet						
12							
	Hard reddish brown and light gray clay (CH) w/ silt seams and pockets, silt stones, calcareous nodules, ferrous stains		20				
16	Bottom @ 15'						
20							
24							
28							
32							
36							
40							

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 13 FEET WHILE DRILLING ▽

WATER LEVEL AT 11.5 FEET AFTER 0.5 HOURS ▼

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PLATE 7

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS

BORING B-7

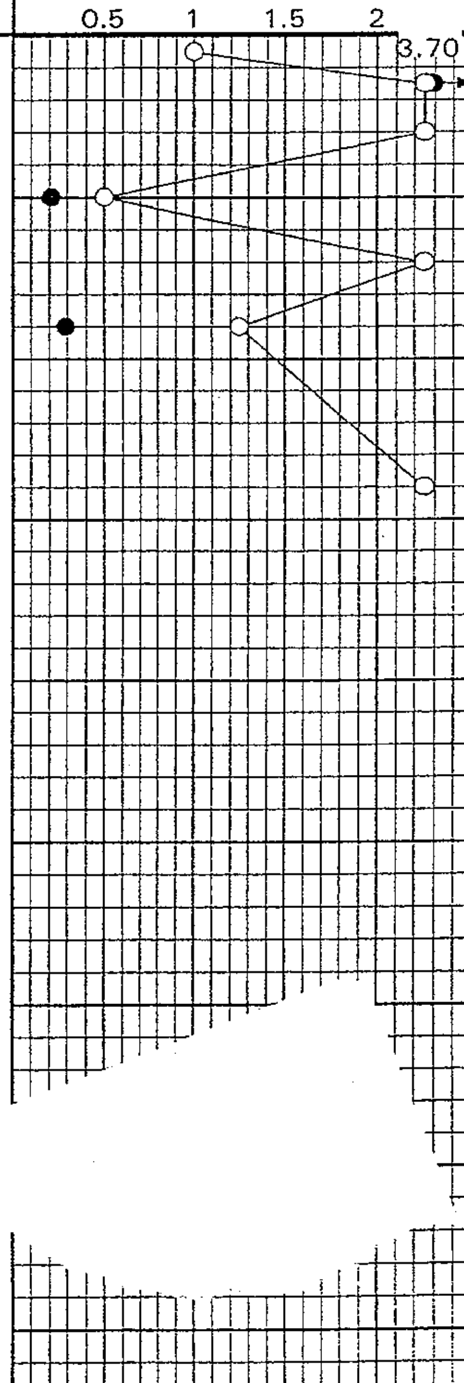
DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

Elevation:

DEPTH IN FEET	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Stiff to hard light gray and tan sandy lean clay (CL) w/ roots, ferrous nodules, vertical sand seams		16				
			8	115		28	10
			9				
4	- w/ sand layers, water puddle @ 5 feet		19	106			
			16				
8			17	111		25	10
	Tan and light gray silty sand (SM)						
12	Hard reddish brown and light gray clay (CH) w/ silt stones, calcarous & ferrous nodules, ferrous stains		28				
16	Bottom @ 15'						
20							
24							
28							
32							
36							
40							

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 10 FEET WHILE DRILLING
WATER LEVEL AT 6.5 FEET AFTER 0.5 HOURS
DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS BORING B-8

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

Elevation:

DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF (○ Pocket Penetrometer, △ Confined Compression, ● Unconfined Compression, □ Torvane)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
Pavement: 2-inch asphalt, 4-inch crushed limestone base, 6-inch light gray sand							
Fill, stiff brown and light gray sandy lean clay (CL) w/ silty sand seams, ferrous & calcareous nodules		13	113	● ≈0.4; ○ ≈1.0		28	10
		15		○ ≈1.0			
					25		
	3	18					
Very loose to loose gray silty sand (SM) - light gray and tan	4 (water encountered while drilling)	21					
	7 (water level after 0.5 hours)	19					
- w/ reddish brown and light gray fat clay seams @ 12 to 13 feet					21		
	8	18					
Bottom @ 15'							

Depth in feet scale: 0, 4, 8, 12, 16, 20, 24, 28, 32, 36, 40

BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 7 FEET WHILE DRILLING
WATER LEVEL AT 9 FEET AFTER 0.5 HOURS
DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS

BORING B-9

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

SHEAR STRENGTH, TSF: ○ POCKET PENETROMETER; △ CONFINED COMPRESSION; ● UNCONFINED COMPRESSION; □ TORVANE (scale 0.5, 1, 1.5, 2)

Elevation:

DEPTH IN FEET	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Pavement: 2-inch asphalt, 6-inch crushed limestone base		14 6		○ 2		30	12
	Fill, very stiff gray sandy lean clay (CL) w/ shell, calcareous nodules, sand layers	8	26			36		
4	Fill, very loose to loose light gray and tan silty sand (SM) w/ ferrous nodules	4	28					
	- gray	11	17					
8	Loose to medium dense light gray and tan poorly graded sand (SP) - tan	8	17			11		
12	- light gray	5	22					
16	Bottom @ 15'							

BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 10 FEET WHILE DRILLING ▽

WATER LEVEL AT 9 FEET AFTER 0.5 HOURS ▼

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

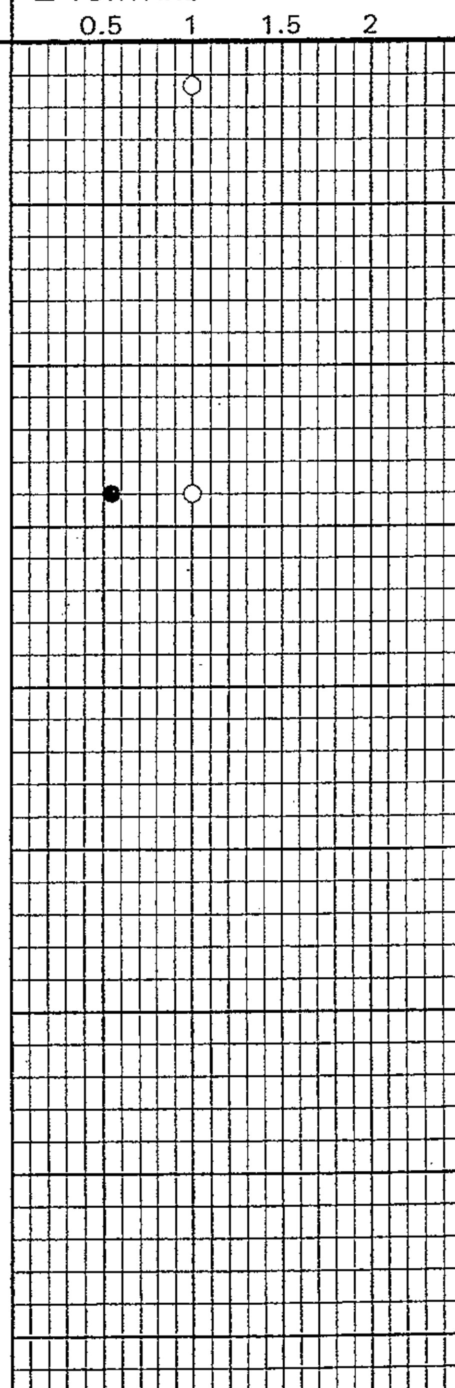
PROJECT: 69th STREET WWTP IMPROVEMENTS
BORING B-10

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Pavement: 2-inch asphalt, 6-inch crushed limestone base		12 15		31	19	15
	Fill, light gray and tan clayey sand (SC) w/ gravel	17	29				
4	Fill, medium dense light gray silty sand (SM) w/ ferrous nodules, clayey sand pockets	10	20		38		
	- w/ shell, gravel	14	11				
8	Fill, stiff reddish brown and light gray clay (CH) w/ sand seams, gravel, shells, ferrous nodules	11	18			60	21
12			27	101			
	Bottom @ 15'						
16							
20							
24							
28							
32							
36							
40							

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT ____ FEET WHILE DRILLING
WATER LEVEL AT ____ FEET AFTER ____ HOURS
DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PROJECT NO. 9137-98

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS BORING B-11

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

SHEAR STRENGTH, TSF: ○ POCKET PENETROMETER; △ CONFINED COMPRESSION; ● UNCONFINED COMPRESSION; □ TORVANE (scale 0.5, 1, 1.5, 2)

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0–2	Pavement: 3-inch asphalt, 6-inch crushed limestone base		14		32	27	15
			11				
2–4	Fill, light gray and brown clayey sand (SC) w/ ferrous nodules	18	16		23		
4–13	Fill, very loose to medium dense light gray silty sand (SM) w/ ferrous nodules	8	23				
	- gray	8	37				
	- dark gray	4	26		27		
	- caved @ 11.5 feet						
13–15	Medium dense brown silty sand (SM)	12	12				
15	Bottom @ 15'						

BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 5 FEET WHILE DRILLING

WATER LEVEL AT 11.5 FEET AFTER 0.5 HOURS

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PLATE 12

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: 69th STREET WWTP IMPROVEMENTS

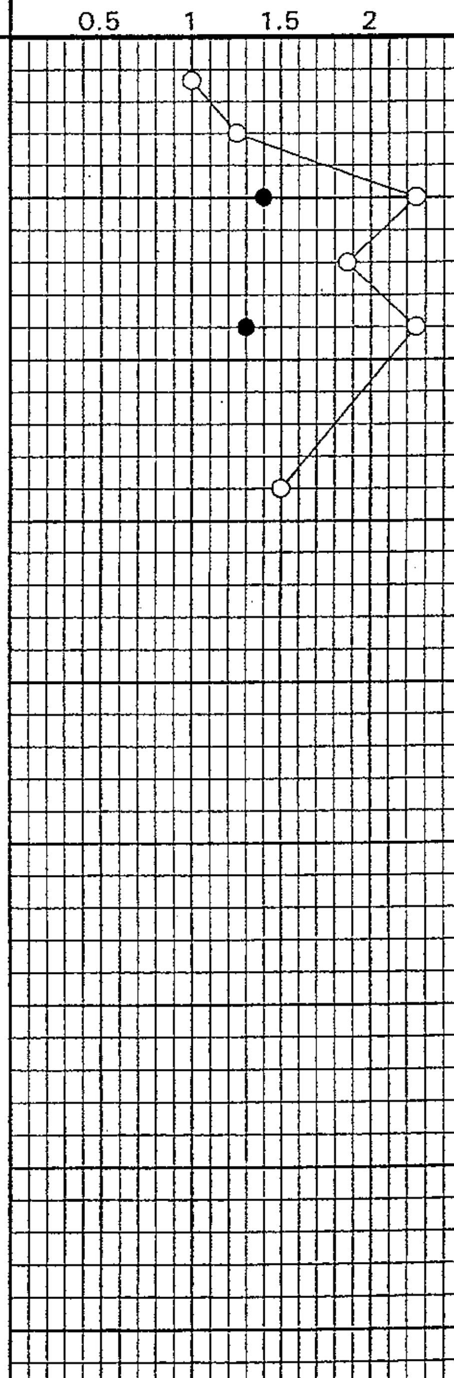
BORING B-12

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION Elevation:	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0			Pavement, 2-inch asphalts, 6-inch crushed limestone base		13			27	13
			Fill, stiff to hard tan and light gray sandy lean clay (CL) w/ silty sand pockets and layers, ferrous nodules		14				
					17				
4					12	120			
					16			35	13
8					14	119			
12					13				
16			Bottom @ 15'						
20									
24									
28									
32									
36									
40									

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT ____ FEET WHILE DRILLING

WATER LEVEL AT ____ FEET AFTER ____ HOURS

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

PLATE 13

PROJECT NO. G137-98

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

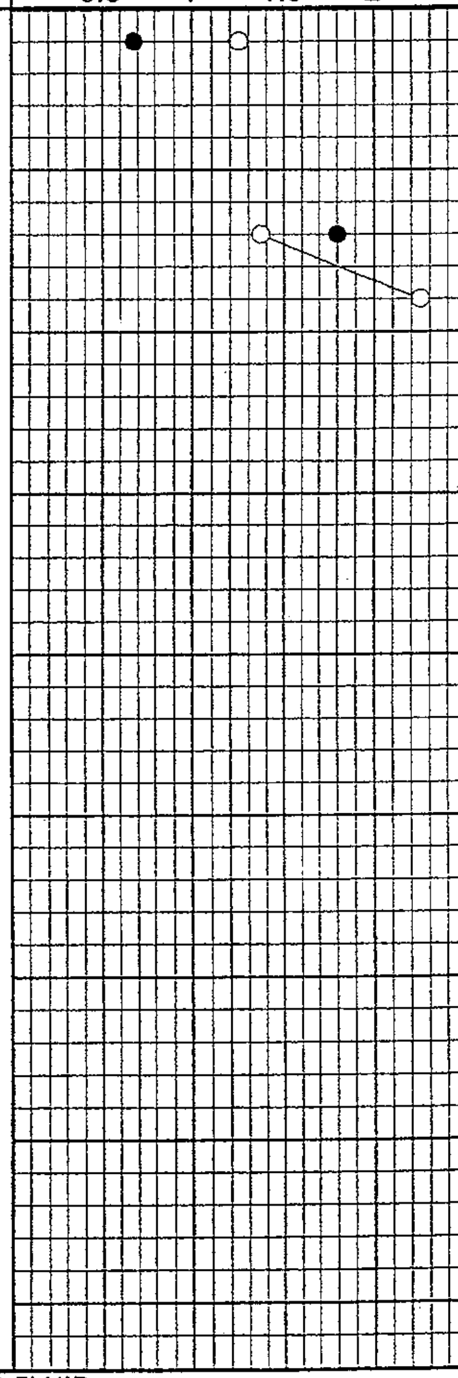
PROJECT: 69th STREET WWTP IMPROVEMENTS

BORING B-13

DATE FEBRUARY 23, 1998 TYPE 3" CORE LOCATION SEE PLAN

DEPTH IN FEET	DESCRIPTION (Elevation:)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT
0	Fill, very stiff tan gray sandy lean clay (CL) w/ silty sand pockets , ferrous nodules		12	119		21	14
	Fill, medium dense tan silty sand (SM)	12	11		27		
4	Firm to hard light gray and tan sandy lean clay (CL) w/ sand pockets, calcareous and ferrous nodules	4	23				
			17	112		41	13
8			18				
12	Medium dense light gray silty sand (SM)	12	22		19		
	Bottom @ 15'						

SHEAR STRENGTH, TSF: O POCKET PENETROMETER; Δ CONFINED COMPRESSION; ● UNCONFINED COMPRESSION; □ TORVANE (scale 0.5, 1, 1.5, 2)



BORING DRILLED TO 15 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 13 FEET WHILE DRILLING

WATER LEVEL AT 13.5 FEET AFTER 0.5 HOURS

DRILLED BY JH DRILLING CHECKED BY SH LOGGED BY JH DRILLING

KEY TO SYMBOLS

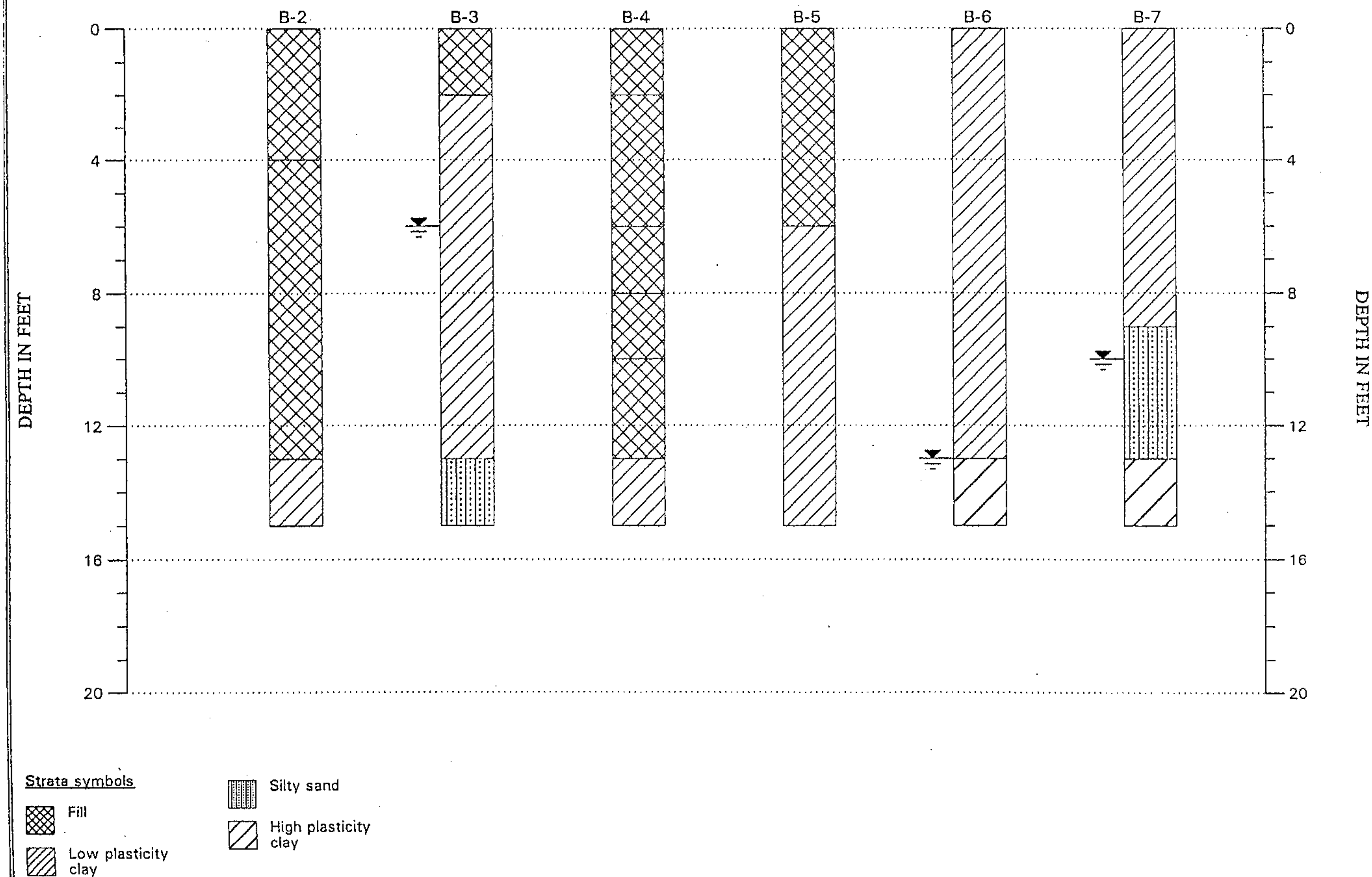
Symbol	Description
Strata symbols	
	Pavement material (or generic rock)
	Fill
	Silty sand
	Low plasticity clay
	High plasticity clay
	Poorly graded sand
Misc. Symbols	
	Water table during drilling
	Water table at boring completion

Symbol	Description
□	Torvane
●	Unconfined Compression
△	Confined Compression
○	Hand Penetrometer
Soil Samplers	
	Rock core
■	Shelby tube sampler
⊠	Standard penetration test

Notes:

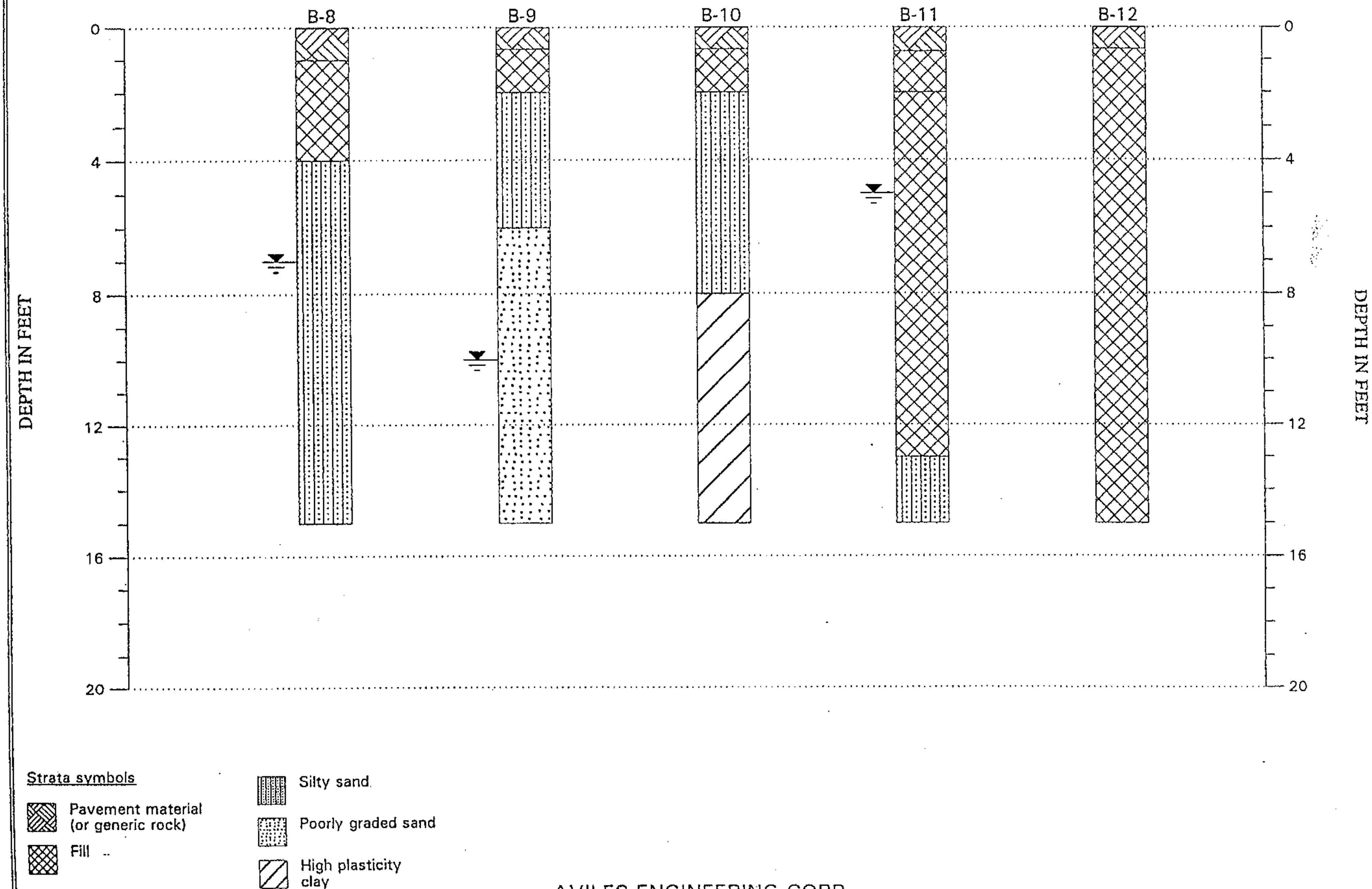
1. Exploratory borings were drilled on FEBRUARY 23, 1998 using a 4-inch diameter continuous flight power auger.
2. No free water was encountered at the time of drilling or when re-checked the following day.
3. Boring locations were taped from existing features and elevations extrapolated from the final design schematic plan.
4. These logs are subject to the limitations, conclusions, and recommendations in this report.
5. Results of tests conducted on samples recovered are reported on the logs.

LOG OF BORINGS
69th STREET WWTP IMPROVEMENTS



AVILES ENGINEERING CORP.

LOG OF BORINGS
69th STREET WWTP IMPROVEMENTS

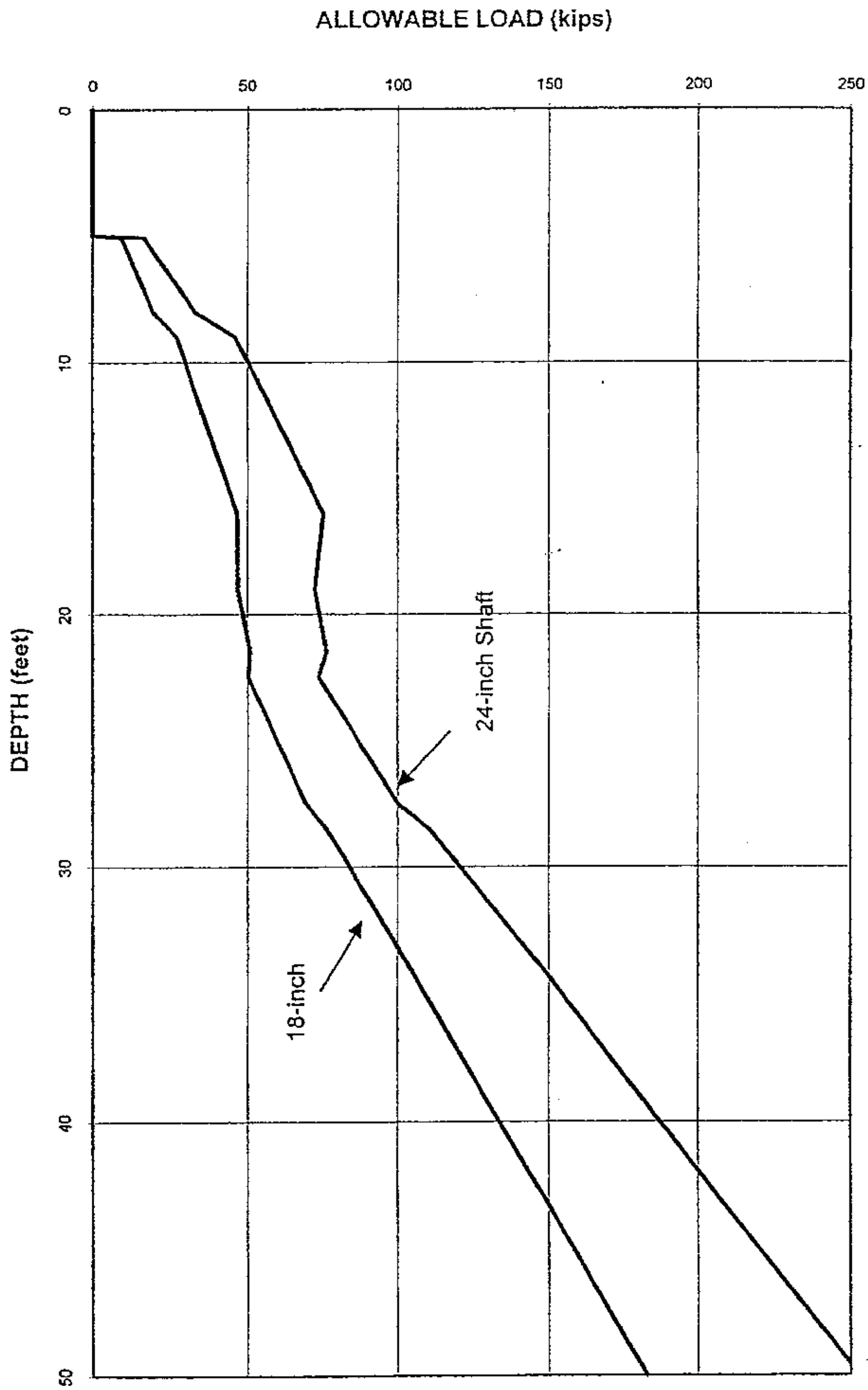


AVILES ENGINEERING CORP.

AVILES
ENGINEERING CORP.

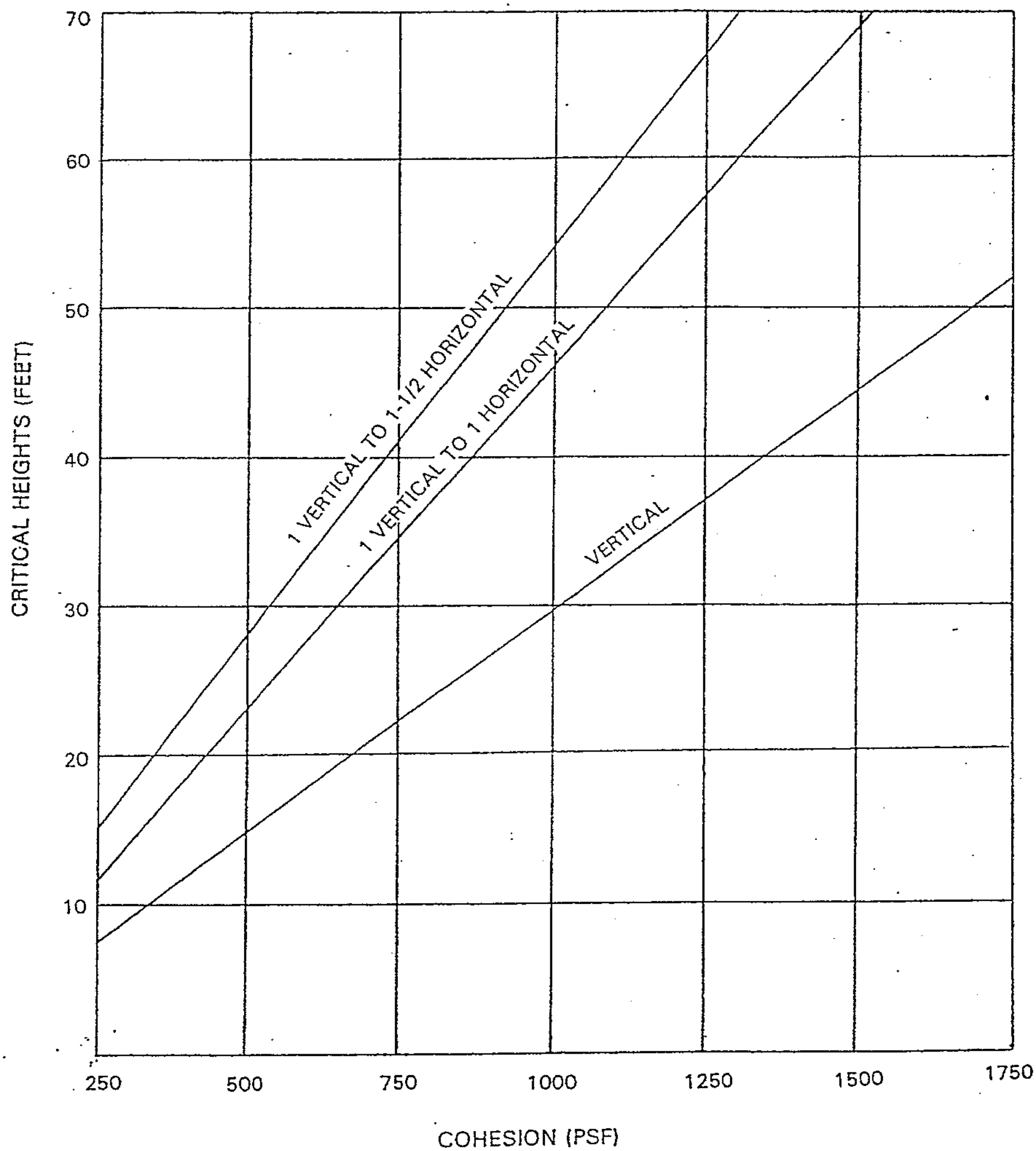
ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part I)
Hydrometer Analysis	D 422	Tex-110-E (Part II)
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
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

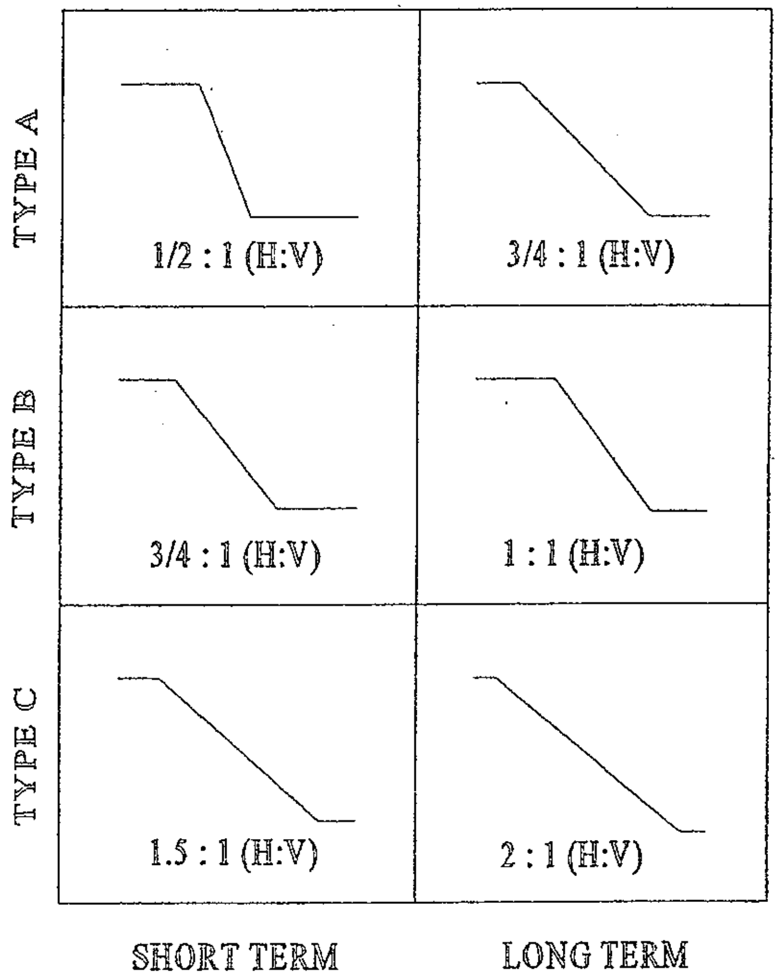


ALLOWABLE LOAD VS DEPTH FOR STRAIGHT SIDE DRILLED SHAFT
(G137-98, SWITCHGEAR AND CONTROL ROOM)
(Reese & O'Neill Method)

AVILES ENGINEERING CORP.



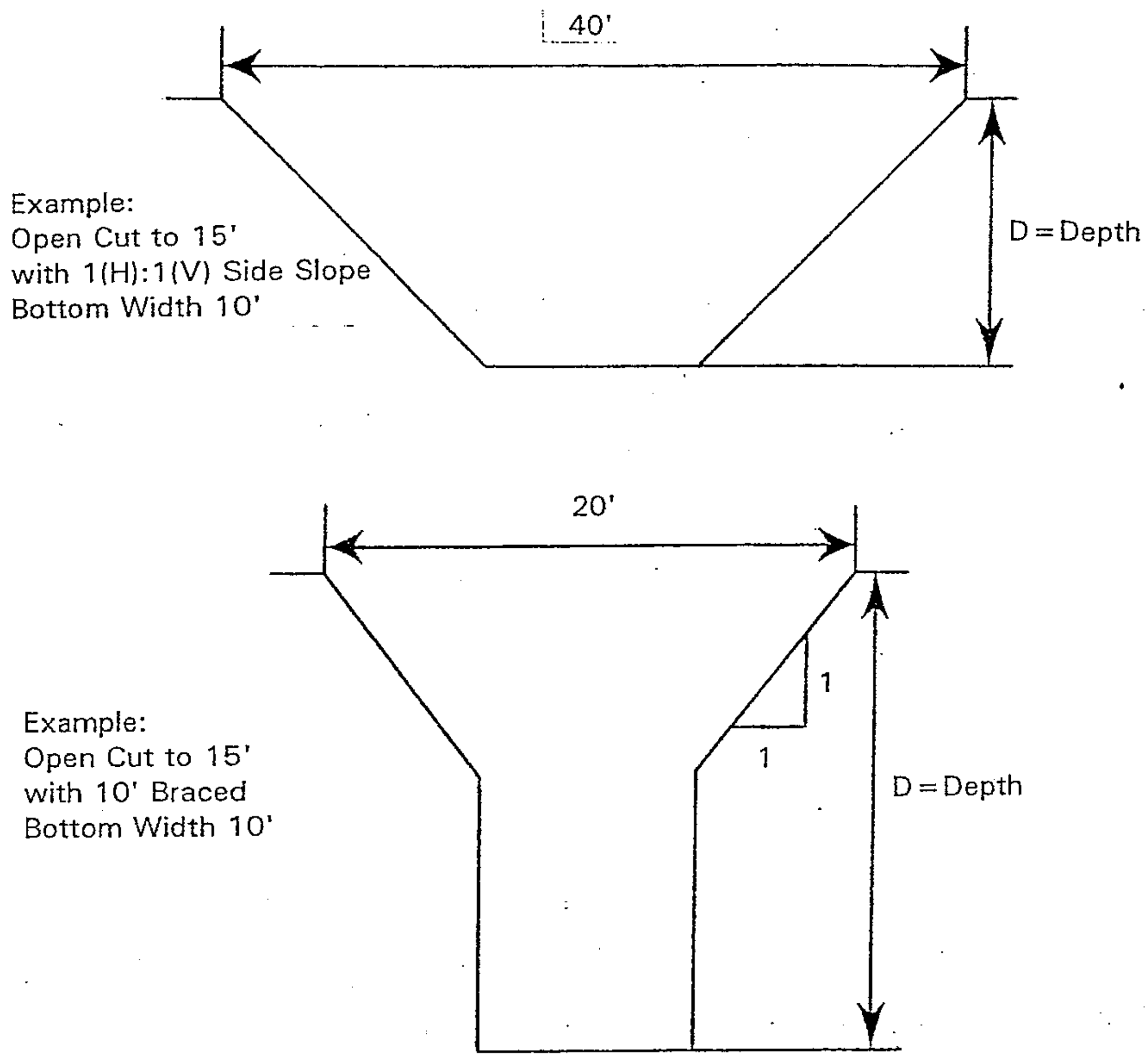
CRITICAL HEIGHTS OF CUT IN STIFF TO VERY STIFF CLAYS



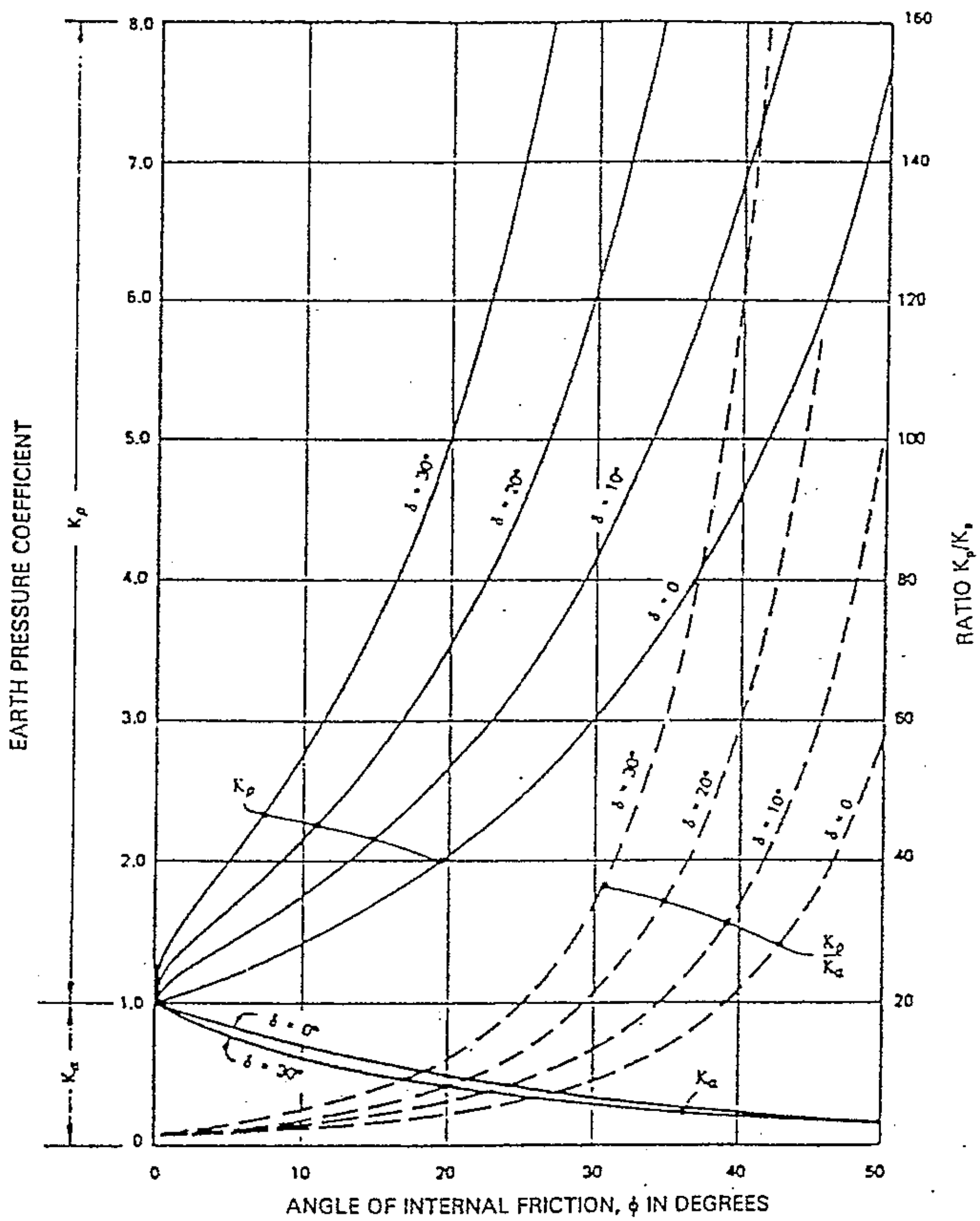
MAXIMUM ALLOWABLE SLOPES

A short term maximum allowable slope of ½ (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth. Short term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3/4 (H) : 1 (V)

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



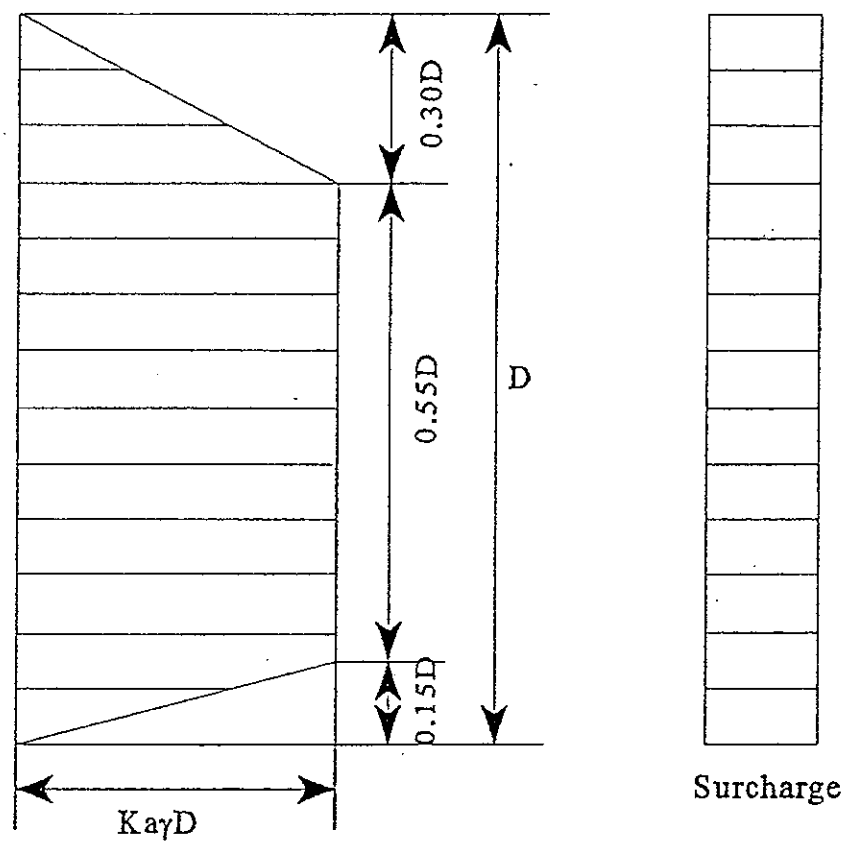
A COMBINATION OF BRACING AND OPEN CUT



δ = ANGLE OF WALL FRICTION

COEFFICIENT OF ACTIVE AND PASSIVE EARTH PRESSURES

DISTRIBUTION OF LATERAL EARTH PRESSURE
FOR DESIGN OF STRUT IN OPEN CUT IN CLAY

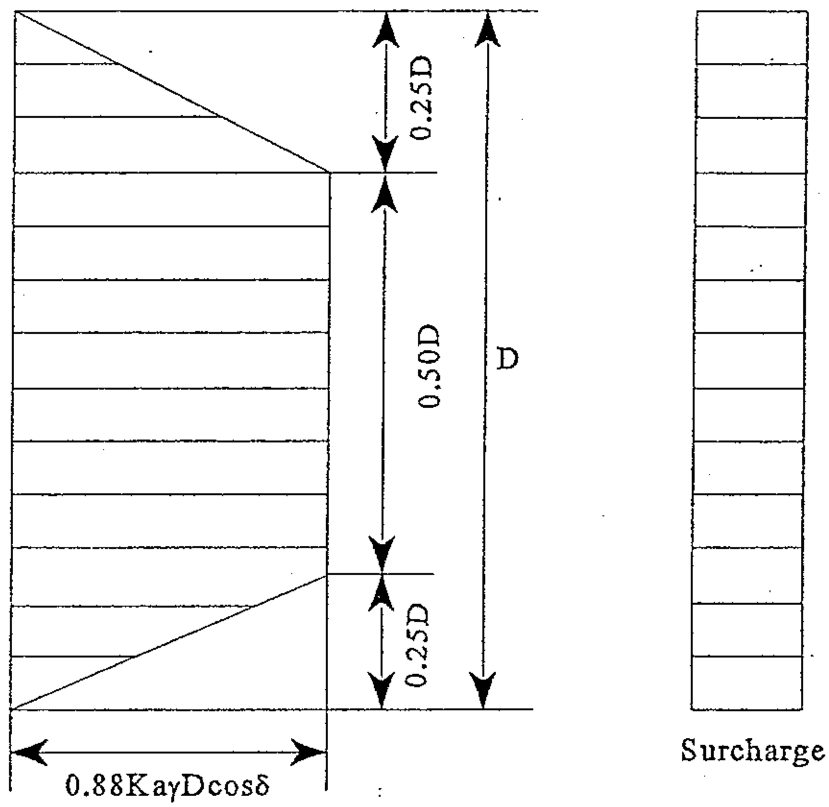


Design Load = KaγD + Surcharge

Where: Ka = 0.33, Coefficient of Active Earth Pressure
γ = 130 pcf, Unit Weight
Surcharge = 400 psf
D = Depth of Strutted Excavation, Feet

AVILES ENGINEERING CORP.

DISTRIBUTION OF LATERAL EARTH PRESSURE
FOR DESIGN OF STRUT IN OPEN CUT IN SAND



Design Load = 0.88KaγDcosδ + Surcharge

Where: Ka = 0.33, Coefficient of Active Earth Pressure
γ = 130 pcf, Unit Weight
δ = 2/3ϕ
ϕ = 30°, Angle of Internal Friction
Surcharge = 400 psf
D = Depth of Strutted Excavation, Feet