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GEOTECHNICAL INVESTIGATION

ACCELERATED SURFACE WATER TRANSMISSION PROGRAM, KELLEY STREET GFS NO. S-0900-55-3; FILE NO. WA 10628 CONTRACT 6B-1 HOUSTON, TEXAS

REPORTED TO

SPARKS & BARLOW, INC.

HOUSTON, TEXAS

by

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HOUSTON, TEXAS

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ACCELERATED SURFACE WATER TRANSMISSION PROGRAM, KELLEY STREET GFS NO. S-0900-55-3; FILE NO. WA 10628, CONTRACT 6B-1 HOUSTON, TEXAS

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EXECUTIVE SUMMARY

Aviles Engineering Corporation (AEC) performed a geotechnical investigation for the proposed 5,105 linear feet of 84-inch waterline construction along Kelly Street, from Maury Street to approximately 500 feet east of Riverton, west of the intersection of North Loop 610 and Highway 59 in Houston, Texas. The waterline will be constructed at invert depths of about 13 to 24 feet using open-cut and tunneling methods. The project will also include pavement reconstruction.

Subsurface conditions were investigated by drilling 7 soil borings (Borings B-1 to B-7) to depths ranging from about 20 to 30 feet below existing grades. Two borings (Borings B-2 and B-6) were converted to piezometers. The natural soils encountered in the borings generally consisted of firm to very stiff sandy clay overlying medium dense silty sand. The sandy clay extended to a depth of about 11 feet in Borings B-1 through B-4, to the termination depths of Borings B-5 and B-6 (30 and 20 feet, respectively), and to a depth of about 13 feet in Boring B-7. Very stiff clay was encountered below the silty sand in Boring B-1 and below the sandy clay in Boring B-7. Cementation was not observed in the silty sands. Fill material, consisting of sandy clay, was encountered in Borings B-1 and B-3 to depths of about 4 and 6 feet, respectively.

Based on the subsurface conditions revealed by the soil borings, the following findings and recommendations are presented:

1. A review of available documented fault locations indicates that a northeast-southwest trending fault, designated as the Pecore fault, is located about ¼ mile south of the project alignment.

2. OSHA soil classifications, trench protection requirements, and bracing design parameters are presented in the text and in a companion report (AEC Report No. G294-00S)

3. Groundwater was encountered at a depth of about 11 to 17 feet during drilling. Groundwater was encountered immediately below the pavement in Boring B-6; we believe it is perched water. Groundwater was at a depth of about 6 feet and 3 feet in the piezometers, approximately 8 weeks after completion. Based on proposed invert depths of about 13 to 24 feet, we anticipate that dewatering will be needed to construct the waterline. A sump and pump arrangement is typically used where cohesive soils occur such as in the vicinity of

Borings B-5 through B-7. Where water-bearing granular soils are encountered (B-1 through B-4), deep wells may be required.

4 We anticipate that the face of the tunnel will be unstable along sections of the alignment where water-bearing granular soils were encountered (Borings B-1 through B-4) and where sandy clays with low undrained shear strength were encountered (Boring B-5 and B-6).

5 Based on the soil borings, we anticipate that soft to very stiff cohesive soils and medium dense, water-bearing silty sand will be encountered during tunneling.

6 Some settlement of the overlying ground is always associated with tunneling in soft ground. Existing nearby structures should be monitored for movements before, during and after project construction. Based on very preliminary analyses, we estimate a surface settlement of about 2 inches for an unsupported tunnel.

7 Existing structures will be closer than the recommended distance between the center of an existing foundation column and the center of the proposed tunnel (25 feet plus 3 times the diameter of the existing pier/pile footing). Existing structures and foundations (located above the base of the tunnel) should be protected before beginning tunneling operations.

8 For waterlines installed by open cut methods, bedding and backfill should be in accordance with Sections 02317 and 02320 (for utility lines) and Section 02716 (for large-diameter water lines, modified for ASWTP) of the 1997 City of Houston Standard Specifications.

9 Recommendations for pavement and subgrade reconstruction are presented in the report.

10 It should be noted that this executive summary does not fully relay our findings and recommendations. The entire report should be carefully reviewed and our findings and recommendations incorporated in the design and construction documents.

AYLES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

ASWTP - KELLEY STREET
GFS NO. S-0900-55-2
FILE NO. WA 10628, CONTRACT 6B-1
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 General

This report presents the results of our geotechnical investigation for the City of Houston Accelerated Surface Water Transmission Program (ASWTP), GFS No. S-0900-55-2; File No. WA 10628, Contract No. 6B-1. This report was prepared in accordance with the ASWTP format for geotechnical reports.

1.2 Location and Description of Project

The waterline will be installed along Kelley Street, from Maury Street to approximately 500 feet east of Riverton, west of the intersection of North Loop 610 and Highway 59 in Houston, Texas. The project alignment is located on Key Maps 453 R and 454 N of the Harris County Atlas; a vicinity map is presented on Plate 1.

According to the information provided to AEC by Sparks & Barlow, Inc., the City of Houston proposes to construct approximately 5,105 feet of 84-in. diameter water main using open-cut excavation and tunneling methods. Access shafts (24 feet in diameter and 25 feet deep) are proposed for the tunnel sections. The waterline will be installed at invert depths of about 13 to 24 feet below existing grade. The project data is summarized in Table 1 below.

TABLE 1. PROJECT DATA SUMMARY

Sta. From	Sta. To	Type of Construction	Location	Approx. Length (ft.)	Approx. Depth below existing grade (ft.)
2+75	5+10	Tunnel	Union Pacific RR	235	19 to 23
5+10	8+98	Open-cut	Kelley Street	388	12 to 15
8+98	14+03	Tunnel	Gold Street	505	21 to 22
14+03	35+93	Open-cut	Kelley Street	2,190	17 to 24
35+93	37+73	Tunnel	Jensen Drive	180	20 to 22
37+73	53+80	Open-cut	Kelley Street	1,607	13 to 23
APPROXIMATE TOTAL LINEAL FOOTAGE OF LINE				5,105	

1.3 Scope of Work

The scope of our investigation included:

- performing field exploration, laboratory testing and engineering analysis, and
- providing geotechnical recommendations to guide the design and construction of the waterline.

2.0 SUBSURFACE INVESTIGATION PROGRAM

As requested, the subsurface investigation program consisted of drilling 7 soil borings (Borings B-1 through B-7) at a spacing of 1,000 feet or less, to depths ranging from 20 to 30 feet, using a truck-mounted drill rig. The approximate boring locations and termination depths are listed below; the locations are shown approximately on the Boring Location Plan on Plate A-1 in Appendix A.

Boring	Approximate Location	Depth (feet)	Comments
B-1	2+70	28	
B-2	8+80	30	Converted to Piezometer
B-3	14+20	28	
B-4	27+00	29	
B-5	38+00	30	
B-6	44+00	20	Converted to Piezometer
B-7	51+00	24	

Undisturbed samples of cohesive soils were obtained from the borings using 3-inch-diameter, thin-wall, seamless steel, Shelby tube samplers in general accordance with ASTM D-1587. The strength of the cohesive soils was estimated in the field using a pocket penetrometer. 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. Undisturbed samples of cohesive soils 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 were then placed in core boxes and transported to our laboratory for testing. Borings B-2 and B-6 were converted to piezometers on completion. Piezometer installation details are presented on Plates B-1 and B-2, in Appendix B.

We did not encounter any odors during our field work or observe unusual staining or evidence of hazardous materials in the soil samples recovered during our field investigation.

3.0 LABORATORY TESTING PROGRAM

The samples were examined and classified in the laboratory by a geotechnical engineering technician. AEC's project engineer reviewed the field data and selected soil samples for laboratory tests to evaluate the engineering properties of the subsurface soils. The test results are summarized on the boring logs on Plates A-2 through A-8 in Appendix A. A key to the symbols used on the boring logs is presented on Plate A-9. ASTM Designation for Soil Laboratory Tests are listed on Plate A-10.

Atterberg limits, moisture content and percent passing the No. 200 sieve tests were performed on representative samples to estimate the index properties and confirm field classification of the subsurface soils. Soil strengths were estimated by unconfined compression and unconsolidated undrained triaxial tests performed on undisturbed samples of cohesive soils.

4.0 SUBSURFACE AND SITE CONDITIONS

4.1 Geology of the Coastal Plains

The Houston area is situated on the Quaternary Coastal Plain of Texas and is located on the north flank of the Gulf Coast Geosyncline. Most of Houston is located on the nearly level, rather featureless depositional plains of the Lissie and Beaumont Formations.

The site is located in the Beaumont formation. The Beaumont formation was deposited on land near sea level in flat river deltas and in fresh water streams and flood plains. During the depositional period, the courses of the major streams and deltaic tributaries changed frequently generating a complex stratification of sand, silt and clay deposits. Stream courses were frequently diverted significant distances from a given point and the water overlying the soil, cut off from a drainage path, would evaporate. Such water, which would be highly alkaline, would precipitate large nodules of calcium nodules (calcareous nodules) throughout the surface of evaporation. With the coming of the Second Wisconsin Ice Age, the nearby sea withdrew, leaving the formation several hundred feet above sea level and permitting the soil to desiccate. The process of desiccation compressed the clays in the

formation such that they became significantly over-consolidated to a large depth. In addition to pre-consolidating the soil, the process of desiccation, together with the later rewetting, produced a network of fissures and slickensides that are now closed but which represent potential planes of weakness in the soils.

4.2 Natural Hazards

We evaluated fault locations by reviewing in-house and available documented fault locations. The closest known fault to the project alignment is the Pecore Fault located approximately one quarter mile to the south. It trends in a southwest-northeast direction.

4.3 Site Stratigraphy and Geotechnical Characterization

Fill soils were encountered to depths of about 4 and 6 feet in Borings B-1 and B-3, respectively. The fill material consisted of sandy clay with shell; the shear strength was variable. The natural soils encountered in the borings generally consisted of a top layer of firm to very stiff sandy clay overlying medium dense silty sand and very stiff clay. The sandy clay extends to a depth of about 11 feet in Borings B-1 through B-4, to the termination depths of Borings B-5 and B-6 and to a depth of about 13 feet in Boring B-7.

The undrained shear strength of the sandy clay was in the range of 250 to 1,000 psf at depths ranging from 15 to 20 feet in Boring B-5 and B-6; within this zone, the material was very sandy and borderline between sandy clay and clayey sand (CL and SC, respectively). The silty sand was medium dense in consistency, was encountered only in Borings B-1 through B-4, and extended below the sandy clay to depths of about 23 to 30 feet below existing grade. The clay was very stiff in consistency and was encountered in Borings B-1 and B-7 at depths of about 23 and 13 feet, respectively.

Details of the soils encountered in the borings are presented on the boring logs. A subsurface profile along the project alignment is presented on Plate A-11.

Existing Pavement Sections The pavement sections encountered at the boring locations during our field investigation are summarized in Table 2 below.

TABLE 2 EXISTING PAVEMENT SECTIONS

Boring No.	Approximate Station	Total Thickness (in.)	AC Thickness (in.)	PCC Thickness (in.)	Base Thickness (in.)
B-1	2+70	6	3	-	3
B-2	8+80	16	4	-	12
B-3	14+20	15	3	-	12
B-4	27+00	11	-	5	6
B-5	38+00	16	-	9	7
B-6	44+00	13	-	9	4
B-7	51+00	18	-	8	10

4.4 Groundwater

Groundwater was encountered during drilling at a depth of about 11 feet in Borings B-1 through B-4 and B-7 and at a depth of about 17 feet in Boring B-5. Perched groundwater was encountered immediately below the pavement in Boring B-6 during drilling. Groundwater was measured at a depth of about 6 feet and 3 feet in the piezometers, about 8 weeks after completion of the piezometers. The measured groundwater depths are summarized in Table 3 below.

TABLE 3. MEASURED GROUNDWATER LEVELS

Boring No.	Groundwater Level During	Piezometric Groundwater
B-1	11	
B-2	11	5.8
B-3	11	
B-4	11	
B-5	17	
B-6	-	3*
B-7	11	

* Groundwater was encountered immediately below the pavement during drilling. Based on the elevated location of the monitoring well, the groundwater appears to be perched water or due to leakage.

4.5 Subsurface Variations

The information in this report summarizes the conditions encountered on the dates the borings were

drilled. Subsurface soil conditions and groundwater depths will vary depending on seasonal rainfall and the time of year when construction is in progress.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

5.1 Trench Excavation Considerations

The contractor should be responsible for designing, constructing, monitoring and maintaining safe excavations. The excavations should not cause any distress to existing structures; the structures should be monitored prior to, during and after construction to check for movements and take necessary action promptly should movements be detected. Excavations may be shored, sheeted and braced, laid back to a stable slope, or other appropriate means used to ensure the safety of workers, public and adjacent structures. Excavation and trenching should be in accordance with OSHA Safety and Health Regulations, 29 CFR, Part 1926. OSHA Soil Types are presented in Appendix C.

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

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

2 If the soil at the surface is dry to the point where tension cracks occur, any water in the 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.

Cut Slopes Plate D-2 in Appendix D presents the maximum (steepest) allowable slopes in Soil Types A, B and C for excavations less than 20 feet. The allowable side slope for temporary excavations in cohesive soils is 1(V):1(H) or flatter, depending on the soil consistency. 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. A logarithmic spiral method is used for calculating earth pressure against bracing for open-cuts. This concept is illustrated on Plate D-4 in Appendix D. For practical application, the pressure is 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) $= K_a\ \gamma\ D$
Where: K_a = Coefficient of Active Earth Pressure
γ = Soil Density (pcf)
D = Depth of Soil (ft)

The design load (L) 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)
$\tan \delta$ = 2/3 tan (angle of soil friction)
example: if ϕ = 41°, δ = 30° (dense to very dense)
example: if ϕ = 30°, δ = 21° (loose to medium dense)

Pressure distribution for the design of struts in open-cuts for clay and sand are illustrated on Plates D-5 through D-8 in Appendix D. If there is water behind the bracing, hydrostatic pressure should be considered in the design. If applicable, sloped soils and traffic loads should also be considered in the bracing design, in addition to the above loads.

Bottom Stability If tight sheeting is terminated at the base of a braced cut in cohesionless soil, the bottom of the excavation can become unstable under a certain condition which is governed by the soil shear strength and the differential hydrostatic head. In granular soils, excavations should be made after de-watering is accomplished to reduce potential bottom stability problems. The sheeting should extend

into stiff cohesive soils located below the granular soil strata. For cuts in cohesive soils, the stability of the bottom can be evaluated according to the procedure outlined on Plate D-9. If the safety factor is less than 1.5, the sheeting should extend a depth of "D_1" below the base of the excavation as shown.

Existing Southern Pacific Railroad Kelley Street crosses the Southern Pacific Railroad via an existing underpass approximately between Sta 50+00 and 51+00. We understand that the underpass has been in existence for several years. Based on "Drawing No. C9-D15, Southern Pacific Lines in Texas and Louisiana, Br. No. 3.61 Highway Underpass Over Loop Highway 137 (Kelly Street)" provided to us, the foundation for the railroad bridge consists of driven pile groups with pile caps. The length of the piles or the foundation bearing stratum are not known. The drawing also shows the side slopes for the underpass are approximately 2.5 horizontal to 1 vertical (2.5:1) under the railroad crossing; the slopes are concrete-lined under the crossing. The existing cut slope away from the railroad crossing grades to about 5:1; these slopes have a grass cover. The slopes appear to be in good condition; we did not observe evidence of movement or distress.

The proposed alignment in this area will be parallel to the above slopes and installed using open-cut excavation. The proposed alignment will be approximately in the center of the east-bound lanes. Our experience with embankment slopes in the Houston area and our observations of the above slopes indicate that the slopes will be stable if they are not undermined, proper excavation and shoring techniques are implemented and the slopes are protected against erosion. We recommend that adequate shoring be installed in the excavation to maintain the integrity of the slope during construction. Guidelines for shoring were presented earlier in this section of this report. Lateral pressure distribution, specific to this location, is presented on Plate D-10.

If the slope is cut during construction, new fill should be placed to reconstruct the slope to its original geometry. The cut section may be backfilled with the excavated soils if they are free of debris, organics and other unsuitable materials. The soils should be placed in horizontal loose lifts not exceeding 8 inches thick and should be compacted to at least 95 percent of the ASTM D-698 maximum dry density within 2 percent of the optimum moisture content. The lifts should be benched into the existing embankment every 3 feet of embankment height; the bench should extend laterally a minimum of 3 feet into the embankment. Shoring should be removed prior to benching.

Existing Slope near Sta 53+00 The proposed alignment will go through an existing slope near Sta.

53+00. We estimate the maximum excavation depth to be about 18 feet (from the bottom of the excavation to the top of the existing slope). We recommend the excavation be shored as outlined earlier in this section. The excavation should be backfilled with select fill (liquid limit less than 50, plasticity index between 8 and 20 and contain no debris, organics and other unsuitable materials). The excavated soils may be used as fill if they meet select fill criteria. Fat clay with liquid limits greater than 50 and PI greater than 20 should not be used since recent studies have indicated that such soils tend to lose their strength when subjected to repeated wetting and drying cycles. Placement and compaction should be in accordance with the preceding paragraph. Adequate ground cover should be re-established with sod or other appropriate erosion protection. The slope should be reconstructed as recommended in the preceding paragraph. The shoring should be removed during backfilling since timber shoring can decay with time and result in compromising the stability of the slope.

Existing Foundations (SPRR Bridge, Hardy Toll Overpass & US 59 Ramp, etc.)

Where the open-cut is near existing foundations for the above structures or near other structures, the excavation should be shored so that the foundations do not move. Guidelines for shoring are presented earlier in this section of the report.

5.2 Excavation Dewatering

The Contractor should be responsible for designing, installing and maintaining dewatering systems for groundwater control. The system should be designed and installed by qualified and experienced professionals who are also capable of monitoring, maintaining and modifying the system, as required. The following discussion provides general guidelines to the Contractor for dewatering.

Dewatering should be in accordance with COH Standard Specifications, Section 01578 - Control of Ground Water and Surface Water. The design of the system should consider the effect of de-watering on adjacent structures. The Contractor should monitor nearby structures along the proposed alignment to ensure that the dewatering does not result in any distress or detrimental effects on these structures.

Excavations in cohesive soils, as encountered in Borings B-5 through B-7, can generally be dewatered by draining the groundwater into sump pits and pumping it out. Excavations in saturated deposits of cohesionless soils (as encountered in Borings B-1 to B-4) are typically controlled by installation of

vacuum well points or deep wells with submersible pumps. The practical maximum depth for the use of vacuum well points is considered to be about 15 feet. Where groundwater control is required below 15 feet, deep wells with submersible pumps have generally proven successful. Multi-stage well points have also been used for deeper excavations up to about 20 feet, depending on site conditions. On some occasions, eductor wells have been used in lieu of wells with submersible pumps.

To enable the Contractor to work on a relatively stable base at the bottom of the excavation, we recommend using a crushed stone mat or other free-draining material at the base of the excavation. Excessive passes with heavy equipment should be avoided on this material to reduce the potential for upward migration of moisture into the excavation. The number of passes considered "excessive" depends on the soil and groundwater conditions, lift thickness relative to the compaction equipment, etc. As a rule of thumb, up to six passes can be considered acceptable. The thickness of the stone mat or free-draining material should be determined in the field. Dewatering should lower the groundwater depth at least 3 feet below the excavation bottom.

5.2.2 Uplift

Underground structures should be designed to resist uplift based on the buoyant weight of the structure and backfilled soil above the structure. The minimum recommended factors of safety against uplift are 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. For uplift design of underground structures, we recommend that the water level be assumed at the ground surface or 100-year flood elevation; whichever is more critical, should be used. If the dead weight of the structure is inadequate to resist uplift force, a toe extension of the base slab may be constructed so that the weight of the soil above the extended slab can be utilized to resist the uplift force. The unit buoyant weight of concrete can be taken as 90 pcf. Design criteria for uplift resistance is shown on Plate D-11.

If a caisson technique is used in constructing access shafts, the buoyant weight of the structure plus the frictional resistance developed between the walls and the surrounding soils can be used to resist hydrostatic uplift. For the cohesive soils encountered along the alignment, an allowable frictional resistance of 300 psf may be used in calculating the frictional resistance; for the granular soils encountered, a coefficient of friction of 0.3 may be used to compute the frictional resistance.

5.3 Vehicular Traffic and Railroad Loads

5.3.1 Pipe Design

Recommended geotechnical parameters for design are presented on Plate 2. Underground conduits may be subjected to loads from overburden and surcharge, and impact loads from traffic. For rigid pipes, overburden and surcharge loads may be computed from Plate 3. Impact loads due to roadway traffic may be obtained from Plate 4. Impact loads due to railroad traffic may be obtained from Plate 5.

A buried flexible pipe deflects under soil load and any surcharge loads; the amount of deflection is a function of the load on the pipe and the combined stiffness (modulus) of the pipe and surrounding soil. The deflection is calculated using the Modified Iowa Formula and the effective stiffness of the surrounding soil. The effective stiff ness of the surrounding soil, E', is a combination of the stiffness of the pipe bedding material, E'_b, and the stiffness of the native soil, E'_n. The long-term deflection is typically used for flexible pipe design. The long-term E'_n values for the native soils encountered along the project alignment to be used for flexible pipe design are presented in Table 4 below.

TABLE 4. LONG-TERM SOIL STIFFNESS VALUES, E'_n

Boring No.	Approx. Invert Depth (ft)	Soil Description	Soil Stiffness (psi)
B-1	18	Medium Dense Silty Sand	600
B-2	20	Medium Dense Silty Sand	600
B-3	14 to 20	Medium Dense Silty Sand	600
B-4	19	Medium Dense Silty Sand	600
B-5	21	Firm Sandy Clay	300
B-6	15	Firm Sandy Clay	300
B-7	19	Stiff Clay	600

5.3.2 Bedding and Backfill

Based on our field investigation, we anticipate that groundwater will be encountered within the bedding zone. Bedding and backfill should be in accordance with Sections 02317 and 02320 (for utility lines) and Section 02716 (for large-diameter water lines, modified for ASWTP) of the 1997 City of Houston

Standard Specifications. The bedding material should consist of cement-stabilized sand extending 6 inches below the bottom of the pipe and to a minimum depth of 12 inches above the top of the pipe. For anticipated wet bottom conditions, a 12-inch layer of crushed stone shall additionally be placed below the bottom of the bedding material. Granular soils other than silt and cohesive soils with a LL less than 45 and PI between 8 to 20 may be used as backfill material above the pipe zone. Cohesive soils with PI greater than 20 may be used as backfill if they are lime- stabilized.

Backfill should be placed in loose lifts not exceeding 8 inches and compacted to a minimum density of 95 percent of the Standard Proctor (ASTM D 698) maximum dry density at a moisture content within 2 percentage points of the optimum moisture content. The lift thickness should be reduced to no more than 4 inches if the backfill is compacted using portable equipment.

Where groundwater is encountered in granular soils, it should be lowered a minimum 3 feet below the invert depth to provide a dry condition before pipe installation. Where cohesive soils with a shear strength of 500 psf or less are encountered, the weak soil should be excavated to a depth of at least 3 feet and replaced with properly compacted select fill.

5.4 Liner Loads

The behavior of a soil-liner system essentially depends on the deformation that develops in the surrounding soil. Following are guidelines for analyzing the loads. The ring load (P_h) is equal to the average radial load exerted by the soil, traffic loads and any other construction loads (see Plate 7) multiplied by the external radius, R, of the lining. The lining should be designed to resist a ring load,

$$P_h = 1.2\ (\gamma z + q + I) \times R$$

where: P_h = ring load, (lb./lin. ft.)
γ = unit weight of soil, (pcf), use 130 pcf
z = depth from the ground surface to the center of tunnel (ft.)
q = vertical stress due to surcharge loads (psf)
I = impact loads from traffic, transportation and jacking operations
R = radius of tunnel (ft.)

Vertical stress due to traffic loads are shown on Plates 4 and 5. Vertical stress due to construction loads can be estimated by the following equation:

$$q = \frac{3Q}{2\pi d^2} \frac{1}{[1+(r/d)^2]^{5/2}}$$

where: q = vertical stress due to surcharge loads (psf)

Q = concentrated load (lb.)

r = horizontal distance from the center of a loaded area to the center line of tunnel (ft.)

d = depth beneath a loaded area to the tunnel crown (ft.)

Higher ring loads may have to be considered if the surrounding soils are known to have high swelling potential, or if the lining is installed very close to the face in soft soils where the radial displacement is reduced. The design ring load may be increased from $1.2(\gamma z+q+I)R$ to $1.3(\gamma z+q+I)R$ to account for the presence of clays with high swelling potential.

5.5 Piping System Thrust Restraint

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

Thrust restraint design procedures based on the American Water Works Association Manual M9 (1979) Concrete Pressure Pipes are discussed herein. Plate D-12 presents the force diagram generated by flow in a bend in a pipe and the equation for computing the thrust force. An example computation of a thrust force for a given surge pressure and a bend angle is presented on Plate D-13.

Frictional Resistance The unbalanced force due to changes in grade and alignment can also be resisted by friction between the pipe and the surrounding soil. The resisting frictional force F_R, along the pipe is computed as:

$$F_R = f\,(2W_e + W_e + W_p)$$

Where, f = Coefficient of friction between pipe and soil

W_e = Weight of soil over pipe (lb/ft)

W_w = Weight of contained water (lb/ft)

W_p = Weight of pipe (lb/ft)

The value of the frictional resistance depends on the material in contact with the backfill and the soil used in the backfill. For a concrete pipe with crushed stone or compacted sand backfill, an allowable coefficient of friction of 0.3 can be used. Soil unit weights of 125 and 62 pcf should be used above and below the groundwater, respectively, to compute the weight of compacted backfill on the pipe.

5.6 Influence of Tunneling on Adjacent Structures

5.6.1 General

The waterline will be installed by tunneling along three segments of the alignment. Based on the soil borings drilled in the vicinity of the tunnels and waterline invert depths, the type of materials that could be encountered within the tunneling zone are shown in Table 5 below. All tunneling operations should be in accordance with Section 02425(LD) of the City of Houston Standard Specifications.

TABLE 5. TYPE OF MATERIALS IN TUNNEL ZONE

Station No.	Alignment	Approximate Invert Depth (ft)	Associated Soil Boring	Type of Material
2+75 to 5+07	Across UPRR	19 to 23	B-1	Medium Dense Silty Sand
7+19 to 12+60	Along Gold St.	21 to 22	B-2, B-3	Medium Dense Silty Sand
36+11 to 37+72	Across Jensen	20 to 22	B-5	Firm to Stiff Sandy Clay

5.6.2 Tunnel Stability During Construction

The Contractor should be responsible for designing, installing, maintaining and monitoring the tunnel system and should retain professionals who are qualified and experienced to perform the above tasks and be capable of modifying the system, as required. Tunneling and associated activities should be in accordance with Section 02425(LD) for large-diameter water main, of the City of Houston Standard Specifications. The following discussion provides general guidelines to the Contractor for tunneling.

The stability of a tunnel face is governed primarily by groundwater and subsurface soil conditions. Based on our investigation, both cohesive and granular soils are likely to be encountered during tunneling. Groundwater will likely be encountered during construction.

We anticipate unstable conditions for tunnels constructed within water-bearing silty sands in the vicinity of Borings B-1 through B-4 and within the tunnel zone in Borings B-5 and B-6 (where very sandy soils were encountered). Granular soils below the groundwater level will tend to flow into the tunnel; granular soils above the groundwater level will generally not stand unsupported but will tend to ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the material in a loose state. Thus granular soils are generally considered unstable in an unsupported tunnel face. Uncontrolled flowing ground can result in large loss of ground; the potential for such ground loss exists at these locations. The Contractor should be made aware of these conditions at the onset and should be responsible to take adequate precautions to prevent distress to existing structures.

Based on our findings, it is our opinion that conventional dewatering techniques may not be effective where the granular soils are located within the tunneling zone. A deep well system may be needed where tunneling is performed in granular soils. Alternatively, tunneling in water-bearing granular soils may require use of an open face Tunnel Boring Machine. Considering the presence of water-bearing granular soils encountered in the tunneling area and based on his local experience, the Contractor should select the appropriate method and means to mitigate distress to existing structures.

A Stability Factor (Overload Factor) N_t may be used to evaluate the stability of a bore face in cohesive soils where $N_t=(P_z - P_a)/C_u$, P_z is the overburden pressure to the tunnel centerline; P_a is the equivalent uniform interior pressure applied to the face; and C_u is the undrained shear strength. Generally, a design value for the Stability Factor (SF) of 4 or less is desirable as it represents a practical limit below which tunneling may be accomplished without significant difficulty. Higher SF values usually lead to large deformations of the soil around the tunnel, with an associated increase in subsidence. It should be noted that the exposure time of the face is most important. If the soils are left under the same conditions, creep of the soil will occur, resulting in a reduction of shear strength. The SF values will therefore increase when construction is slow. We calculated a SF of 5.5, for an unsupported tunnel face for the weak sandy clay encountered within the tunnel zone in Boring B-5 at Jensen Drive.

5.6.3 Ground Subsidence

Construction of tunnels in soft ground often induces some degree of settlement (ground subsidence) of the overlying ground surface. If such settlement is excessive, it may cause damage to existing structures and services located above and/or near the tunnel.

Observations made in a variety of case studies indicate that the surface settlements are more or less symmetrical about the vertical axis of the tunnel system, forming a trough-like depression which has been related to the shape of the probability or error curve (Peck, 1969).

The geometric properties of the normal probability curve, as used to represent a cross section through the settlement trough above a tunnel, are shown on Plate 6 in which:

δ_{max} = the maximum settlement at any cross section, ft.
z = depth from ground surface to the axis of the tunnel, ft.
a = radius of the tunnel, ft.
i = standard deviation of the settlement curve, related to the settlement trough
V = volume of "lost ground", ft^3 (assumed to be equal to the area of the settlement curve)

Predicting the amount of loss of ground (or ground subsidence) due to tunneling is very difficult, primarily because of the numerous factors involved in the analysis and the lack of accurate information available during design. A surface settlement of about 2 inches was estimated in the vicinity of Borings B-1 and B-2 assuming no internal pressure is applied to the face of the tunnel. This is a very preliminary estimate; the actual amount of settlement will vary depending on factors such as those listed below. The estimate should be refined by the Contractor based on his knowledge of the site, construction methods and tunneling experience in the project area.

The above estimate assumes that: proper tunneling techniques will be implemented (for example, excessive face loss, pitch and yawing of the shield, and over-cutting will not occur, no flowing sands); the backfilling will be properly accomplished and in a timely manner; the subsurface soils do not run or flow; the clays do not squeeze; and no obstructions are caught in the rotating cutter. Other factors such as other existing tunnels, excavations and dynamic or vibratory loads in the vicinity could also affect the above values significantly, and should be taken into consideration.

The influence zone of a tunnel is assumed to extend a distance of about 2.5i from the center of the tunnel where i is defined above and presented graphically on Plate 6. We estimated a value of i to be about 10 feet for the proposed tunnel having a primary liner diameter of 114 inches; this resulted in a tunnel influence zone extending approximately 25 feet from the centerline of the tunnel. The preceding analysis assumes no raveling of the subsurface soils will occur; if raveling occurs, the above

distance will be greater than estimated.

We emphasize that the size of the influence zone of a tunnel is difficult to determine because several factors influence the response of the soil to tunneling operations including type of soil, groundwater level, type of tunneling equipment , method of tunneling, experience of operator and other construction in the vicinity. The values of tunnel influence zone presented herein are therefore rough estimates.

As stated above, existing structures in the vicinity of the proposed alignment are subject to possible distress due to induced settlement. These structures should be closely monitored prior to, during and for a period after tunneling operations. Monitoring should be as recommended in Section 5.6.6 of this report. The Contractor should be aware of, and make provisions for, other on-going or proposed construction in the project vicinity such as excavation, tunneling, installation of new structures, etc. that could have an impact on the design and construction of the proposed waterline and vice versa.

5.6.4 Influence of Tunneling Under Existing Union Pacific Railroad Facilities

The top of the railroad was at approximately El. 59 and the top of Boring B-1 at El. 54, suggesting that the waterline will be constructed at an invert depth of about 24 to 28 feet below the top of the railroad embankment. Based on the results of Boring B-1, we anticipate that silty sand and sandy clay will be encountered during tunneling below the Union Pacific Railroad line. The Contractor should be aware that the silty sand is water-bearing. Adequate precautions should be taken to prevent or minimize movements of the existing railroad tracks and other structures during construction. We also recommend that the Contractor closely monitor excavated volume versus estimated quantities so that any discrepancies (indicating possible subsidence/sloughing of the subsurface soils) can be detected and corrected in a timely manner.

5.6.5 Influence of Tunneling near the Existing Bridge Columns

Tunneling will be performed in the vicinity of the existing bridge columns for the overpass bridge for Loop 610. The impact of tunneling operations on existing bridge columns will depend on the horizontal and vertical location of the tunnel relative to the columns, and other factors listed in Section 5.6.3 of this report.

As in Boring B-1, Borings B-2 and B-3 (which are located in the general area of the bridge columns) encountered silty sand in the tunneling zone. Precautions recommended for tunneling near Boring B-1 should also be implemented here.

Details for the existing foundations (foundation bearing strata, lengths and capacities of the piers/piles, etc.) are not known. Based on our experience, we assume that the foundation system consists of either drilled piers or driven piles. In the Houston area, drilled piers or driven piles obtain their load carrying capacity from side friction and end bearing. For a given pier or pile to develop 100 percent efficiency in side friction capacity, the minimum center-to-center spacing between the pier and any adjacent pier is 3 times the diameter of the larger pier. For the existing structures, we anticipate that the piers/piles are below the elevation of the base of the proposed tunnel.

To reduce the potential for the tunnel influencing the existing foundations or structures, we recommend a minimum horizontal distance of 25 feet plus 3 times the diameter of the nearest column footing, between the centerline of the tunnel and the centerline of the column footings. We understand however, that some of the existing structures will be closer than the above recommended distance.

If the bridge foundations are located too close to the proposed tunnel, subsidence or settlement of the soils adjacent to these foundations can result in downdrag or additional loads on the existing piers/piles and could adversely affect their performance. Also, lateral movements of the soils away from the piers/piles (as a result of collapse/cave-in or subsidence of the soils above the tunnel) can reduce the friction capacity of the existing foundations due to loss of contact between the soil and the piers/piles. Existing structures and foundations (located above the base of the tunnel) should be adequately protected before beginning tunneling operations. Methods to prevent movement and/or distress to existing structures will require the services of a specialty contractor.

5.6.6 Monitoring Existing Structures

Although we have recommended measures and provided guidance for reducing movements of existing structures during construction of the proposed pipeline, several factors (including construction methods, weather conditions, other construction in the vicinity and 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 alignment.

Prior to construction, surface settlement markers and geotechnical instrumentation (inclinometers, etc.) should be installed in accordance with the COH Standard Specification 02425(LD)-26, "Item 3.09 - Monitoring". This should be performed during and after completion of construction.

5.7 Lateral Earth Pressure

Lateral earth pressures for clays and sands may be computed as illustrated on Plates D-5 through D-8.

5.8 Pavement Recommendations

5.8.1 General

We understand that flexible pavement reconstruction will only be required near the intersection of Gold and Kelley Streets and at Sta. 17+00 where a 10-foot wide pavement will be saw-cut to a depth of 2 inches to reinstall a finger joint. We also understand that the traffic volume is light at the above intersection. The other pavements will have concrete surface and be in accordance with ASWT Program requirements for concrete pavements.

The replacement pavement section at Sta. 17+00 should match the existing pavement section. The pavement design recommendations for Gold Street presented herein are based on the "AASHTO Guide for Design of Pavement Structures," 1993 edition and review of the existing pavement sections.

For the light traffic that occurs along Gold Street, we assumed a daily, equivalent single axle load (ESAL) repetitions of 5 over 25 year design life; this resulted in an ESAL of 45,625. All assumptions and input parameters should be verified prior to final design.

5.8.2 Flexible Pavement Assumptions

The design procedure includes the determination of the structural number of the pavement structure, as well as the thickness of individual components of the surface and subbase courses, and subgrade. The basic equation developed by the AASHTO Road Test is:

$$SN = a_1(D_1) + a_2(D_2) + a_3(D_3) \quad \text{.....(1)}$$

where SN = Structural Number for the total pavement structure

a_1, a_2, a_3 = layer coefficients for surface, base and subbase course, respectively.

D_1, D_2, D_3 = thickness of surface, base and subbase courses, respectively, in inches.

Following are the other parameters used in the design.

1) Resilient Modulus, M_R = 2,250 pci
2) Drainage Coefficient C_d = 1.2
3) Overall Standard Deviation, S_o = 0.35
4) Reliability, R = 95%
5) Initial Serviceability, P_0 = 4.0
5) Terminal Serviceability, P_t = 2.0

Using the above parameters, we computed the structural number (SN) to be 3.25. Pavement sections should have the summation of the product of the layer thickness and corresponding layer coefficient equal to or greater than the above SN values. To determine the thickness of the surface and base courses, the general Equation (1) presented above should be used. The input values should be confirmed prior to final design.

5.8.3 Recommended Flexible Pavement Section

We recommend the following pavement section:

SECTION	ASPHALT PAVEMENT
Surface Course	2-inch AC*
Base Course	8-inch crushed limestone
Subgrade	6-inch lime-stabilized subgrade

*AC = Hot-mix hot-laid asphalt concrete

It should be noted that various thicknesses of AC, base and subbase can be used to achieve the same SN. The top 6 inches of subgrade should be stabilized with at least 6% hydrated lime by dry weight of soil; the amount of lime required should be confirmed by laboratory testing prior to construction. Lime-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 between optimum and 3 percent wet of

optimum. Lime-stabilization should be in accordance with City of Houston Specification 02336.

The limestone base material should be in accordance with TxDOT Item 247 Type B, Grade 2 (or equivalent). The limestone base should be compacted to a minimum 95 percent of the ASTM D-1557 maximum dry unit weight.

The surface of the compacted limestone base should be primed with 0.20 gallons per square yard of MC-30 cut back asphalt. Hot mix AC should be in accordance with TxDOT Item 340, Type D. The AC should be compacted between 94 and 97 percent of the ASTM D-1559 maximum unit weight.

5.8.4 Subgrade Preparation

We recommend that the top six inches of the subgrade below the pavement be stabilized with lime. For preliminary planning purposes, 6 percent hydrated lime may be used; the actual amount of lime should be determined by laboratory testing prior to start of construction. Lime-stabilization should be in accordance with City of Houston Standard Specification 02336.

We recommend that the subgrade be prepared as follows:

- Clear the existing pavement sections and strip the surface soils to suitable depths.
- Proof-roll surfaces exposed after stripping in accordance with TxDOT Specification Item 216 or equivalent City of Houston Standard Specification. On-site soils may be used as backfill if backfill is required. The backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 698.
- Scarify the top 6 inches of the exposed surface, mix with stabilizing agent as discussed earlier and compact it to 95 percent of the ASTM D 698 maximum dry density in accordance with the City of Houston Standard Specifications.

6.0 LIMITATIONS

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 at this site or similar structures located at other sites, without additional evaluation and/or investigation.

7.0 AUTHORIZATION

This investigation was authorized by Mr. Edgar C. Barlow, P.E., with Sparks & Barlow, Inc. upon approval of AEC Revised Proposal No. G99-08-05R, dated August 18, 2000.

8.0 REFERENCES

A list of the literature used as reference in preparing this report is attached at the end of this report.

9.0 GENERAL

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

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10.0 CLOSING REMARKS

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

AVILES ENGINEERING CORPORATION

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February 13, 2001

ACO/AAJ:cr

Copies Submitted: 3 Sparks & Barlow, Inc.
1 File

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REFERENCES

1. Deere, D. U., Peck, R. B., Monsees, J. E., and Schmidt, B., (1969), "Design of Tunnel Liners and Support Systems, Washington, D.C." National Technical Institute Service, Publication No. Report No. PB-183799

2. Peck, R. B. (1969) "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

3. Schmidt, B. (1979). Settlements and Ground Movements Associated with Tunneling in Soil. Ph.D. Thesis, University of Illinois.

4. Thomson, J. (1993) Pipejacking and Microtunneling, published by Blackie Academic & Professional, London, England.

5. Bickel, J.O., Kuesel, T.R., King, E.H. (1996) Tunnel Engineering Handbook, published by Chapman & Hall, New York, N.Y., USA.

6. American Water Works Association Manual M9 (1979), Concrete Pressure Pipes.

7. Amster K. Howard, Modulus of Soil Reaction Values for Buried Flexible Pipe. Journal of Geotechnical Engineering Division, January 1977.

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ATTACHMENTS

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PLATES

PLATES

Plate 1	Vicinity Map
Plate 2	Geotechnical Design Parameters
Plate 3	Earth Pressure on Rigid Pipe
Plate 4	Live Loads on Pipe Crossing Under Highway
Plate 5	Live Loads on Pipe Crossing Under Railroad Tracks
Plate 6	Settlement Trough Above a Tunnel
Plate 7	Loading on Liner

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TABLE 2 - GEOTECHNICAL DESIGN PARAMETERS

Boring	Depth (ft.)	Soil Description	Wet Unit Weight (pcf)	Submerged Unit Weight (pcf)	Undrained Conditions						Drained Condition					
					c (psf)	ϕ (deg)	E (psi)	k_a	k_o	M	C' (psf)	ϕ'	E' (psf)	k_a	k_o	M
B-1	0-5	Fill	130	67	1000	0	100,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	5-11	Sandy Clay	130	67	1500	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	11-23	Silty Sand	125	62	0	30	280,000	0.33	0.5	0.3	0	35	280,000	0.27	0.45	0.3
	23-28	Clay	130	67	2000	0	250,000	0.8	1.0	0.45	100	20	280,000	0.49	1.0	0.3
B-2	0-11	Sandy Clay	130	67	1500	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	11-30	Silty Sand	125	62	0	30	280,000	0.33	0.5	0.3	0	35	280,000	0.27	0.45	0.3
B-3	0-6	Fill	130	67	1000	0	100,000	0.8	1.0	0.45	100	20	200,000	0.49	0.7	0.3
	6-11	Sandy Clay	130	67	1500	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	0.7	0.3
	11-30	Silty Sand	125	62	0	30	280,000	0.33	0.5	0.3	0	35	280,000	0.27	0.45	0.3
B-4	0-11	Sandy Clay	130	67	1500	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	11-30	Silty Sand	125	62	0	30	280,000	0.33	0.5	0.3	0	35	280,000	0.27	0.45	0.3
B-5	0-10	Sandy Clay	125	62	1300	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	10-25	Sandy Clay	120	57	300	0	50,000	0.8	1.0	0.45	100	20	70,000	0.49	1.0	0.3
	25-30	Sandy Clay	130	67	1200	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
B-6	0-15	Sandy Clay	120	57	300	0	50,000	0.8	1.0	0.45	100	20	70,000	0.49	1.0	0.3
	15-20	Sandy Clay	130	67	2000	0	250,000	0.8	1.0	0.45	100	20	280,000	0.49	1.0	0.3
B-7	0-13	Sandy Clay	130	67	1200	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	13-20	Clay	125	62	1200	0	140,000	0.8	1.0	0.45	100	20	200,000	0.49	1.0	0.3
	20-28	Clay	125	62	2000	0	250,000	0.8	1.0	0.45	100	20	280,000	0.49	1.0	0.3

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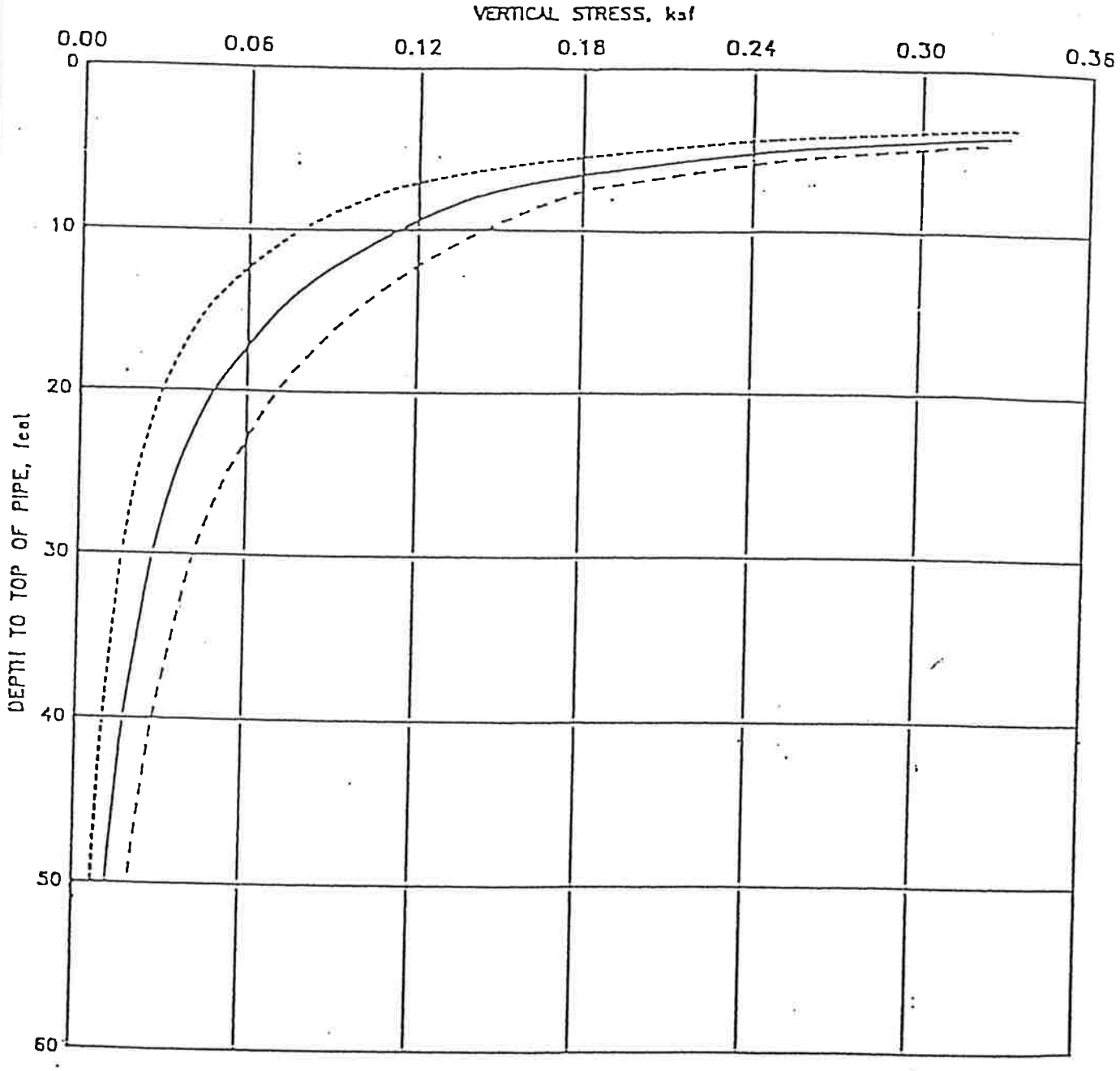

Legend:

——— One passing truck
——— Two passing trucks
– – – Four passing trucks

Notes: 1. The vertical stress was estimated using AASHTO H20 or HS20 truck axle loadings on paved surfaces.
2. Impact factor was included in the vertical stress.

LIVE LOADS ON PIPE CROSSING UNDER HIGHWAY

PLATE 4

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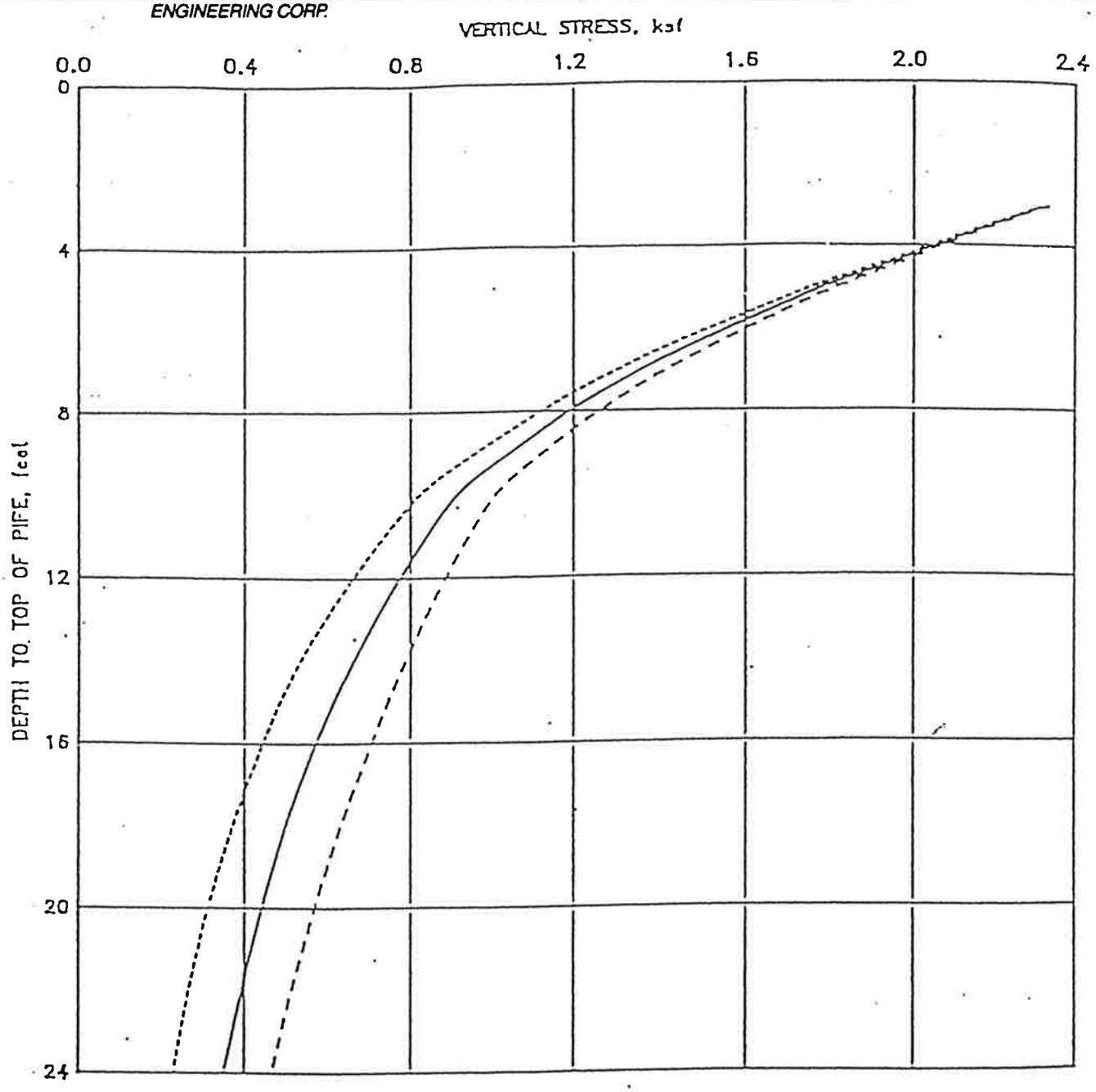

Legend:

——— Single passing track
——— Two passing tracks
– – – Three passing tracks

Notes: 1. The vertical stress was estimated using a Cooper E–80 loading.
2. Impact factor was included in the vertical stress.

LIVE LOADS ON PIPE CROSSING UNDER RAILROAD TRACKS

PLATE 5

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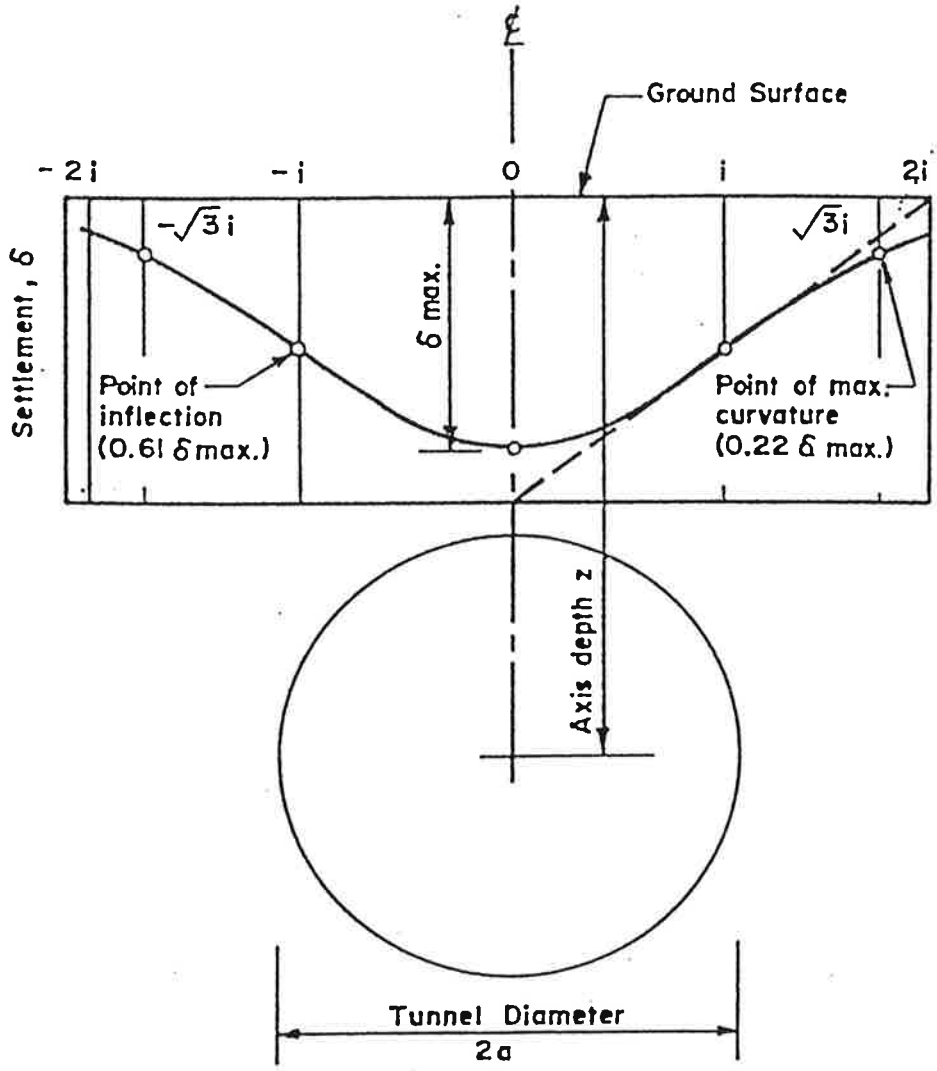

Ratio $\frac{i}{a}$ is function of $\frac{z}{2a}$ and soil conditions

Volume of trough $\simeq 2.5 i \delta max.$

SETTLEMENT TROUGH ABOVE A TUNNEL

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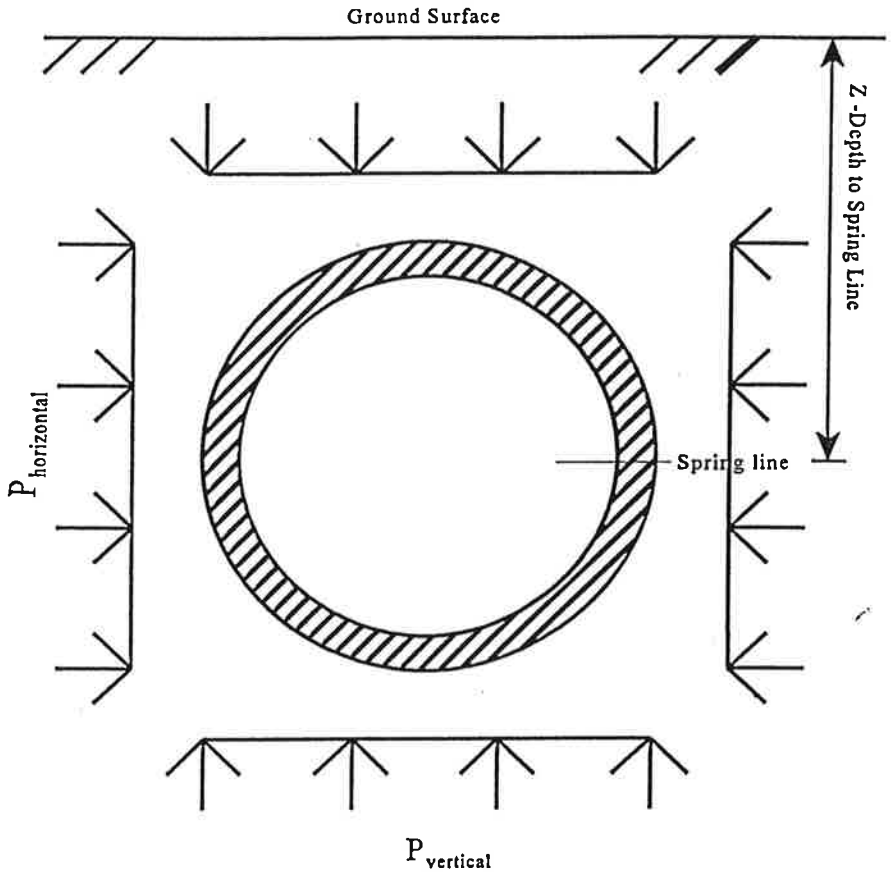
LOADINGS ON LINER/PIPE/CASING



P_v = 130.Z + q + I (psf)

P_h = 1.2 P_v (psf)

Where:

P_v	=	Maximum/permanent vertical load
P_h	=	Maximum/permanent lateral load
q	=	Surcharge load (psf)
I	=	Impact load from traffic, transportation and jacking operations (psf)

APPENDICES

APPENDIX A

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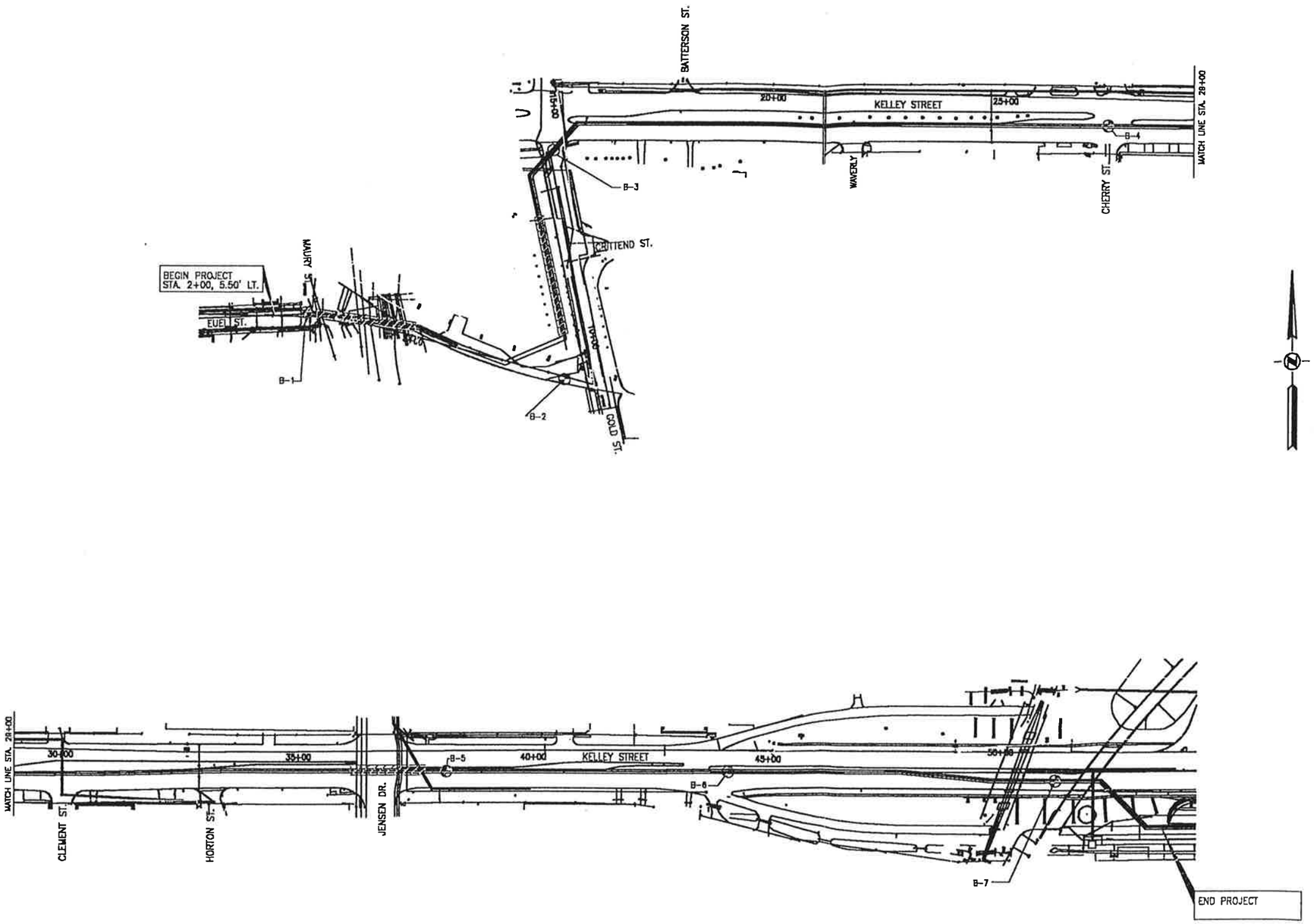
APPENDIX A

Plate A-1	Boring Location Plan
Plate A-2 thru 8	Boring Logs
Plate A-9	Key to Symbols
Plate A-10	ASTM Designation for Soil Laboratory Tests
Plate A-11	Generalized Soil Profiles



Aviles Engineering Corp.

BORING LOCATION PLAN
WATER TRANSMISSION LINE
KELLEY STREET
HOUSTON, TEXAS

AEC Project No.: G294-00	Source Dwg By: Sparks & Barlow, Inc.	
SCALE:1" = Approx. 220'	Date: 11/13/00	Dwg By: WESTBROOK

File No.LD3\drawings\G294-00

Plate A-1

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-1

DATE SEPTEMBER 19, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION Elevation: N/A	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF ○ POCKET PENETROMETER △ CONFINED COMPRESSION ● UNCONFINED COMPRESSION □ TORVANE 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 3" Asphalt and 3" Base (stabilized silty sand)		12						
			Fill: very stiff gray & tan Sandy Clay (CL) with ferrous nodules and shells		17	112		61	38	19	19
5			Stiff tan and gray Sandy Clay (CL), with sand seams		13						
					15	118			36	17	19
10					17				32	16	16
			Medium dense light gray Silty Sand (SM) ▽	13	24			17			
15											
				13	26						
20											
			Very stiff red & light gray Clay (CH) with calcareous and ferrous nodules		26				50	23	27
25											
					20						
30			Bottom of Boring at 28'								
			Groundwater was encountered at a depth of 11 feet, and 1/2 hour after completion of drilling the hole was caved at a depth of 17.3 feet below existing grade.								
35											

BORING DRILLED TO 28 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 11 FEET WHILE DRILLING ▽

WATER LEVEL AT - FEET AFTER - HOURS ▽

DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-2

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-2

DATE SEPTEMBER 19, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION (Elevation: N/A)	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 4" Asphalt and 12" Base (silty sand)							
			Stiff tan Sandy Clay (CL) with ferrous nodules		12	115				
5					17	113		32	16	16
			Firm with sand seams 6'- 8'		16	112				
			with calcareous nodules 8'- 10'		15			35	18	17
10										
			Medium dense light gray Silty Sand (SM) ▽	21	21					
15										
			Sand with silt (SP-SM) 18'- 20'	23	23		10			
20										
				20	25					
25										
				25	23		18			
30			Bottom of Boring at 30'							
			Groundwater was encountered at a depth of 11 feet, and 1/2 hour after completion of drilling the hole was caved at a depth of 17 feet below existing grade.							
35										

SHEAR STRENGTH, TSF

○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE

0.5 1 1.5 2

BORING DRILLED TO 30 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 11 FEET WHILE DRILLING ▽
WATER LEVEL AT - FEET AFTER - HOURS ▽
DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-3

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-3

DATE SEPTEMBER 19, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION Elevation: N/A	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF ○ POCKET PENETROMETER △ CONFINED COMPRESSION ● UNCONFINED COMPRESSION □ TORVANE 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 3" Asphalt and 12" Base								
			Fill: stiff gray Sandy Clay (CL) with ferrous nodules and sand seams		14				28	15	13
5			-with shells 4'- 6' Tan 4'- 10'		15	113					
			Very stiff, tan and gray Sandy Clay (CL), with sand seams 6'- 8'		14	118					
			with calcareous nodules 8'- 10'		15				35	18	17
10			Medium dense tan Silty Sand (SM)								
				12	24			47			
15											
			Reddish 18'- 28'	16	26			19			
20											
				18	22						
25											
				18	21						
			Bottom of Boring at 28'								
30											
			Groundwater was encountered at a depth of 11 feet, and 1/2 hour after completion of drilling the hole was caved at a depth of 16.5 feet below existing grade.								
35											

BORING DRILLED TO 28 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 11 FEET WHILE DRILLING ▽

WATER LEVEL AT - FEET AFTER - HOURS ▽

DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-4

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-4

DATE SEPTEMBER 22, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF (○ POCKET PENETROMETER, △ CONFINED COMPRESSION, ● UNCONFINED COMPRESSION, □ TORVANE; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 5" Concrete and 6" Base (stabilized silty sand)		16			65	33	16	17
			Stiff gray Sandy Clay (CL) with ferrous nodules -tan & light gray 2'- 10'		18	111					
5					16						
			-with calcareous nodules 6'- 8'		16	110			39	18	21
10					18						
			Medium dense tan Silty Sand (SM)	14	23						
15											
			Light gray 18'- 29'	13	38			25			
20											
				15	28						
25											
				15	21						
30			Bottom of Boring at 29'								
			Groundwater was encountered at a depth of 11 feet, and 1/2 hour after completion of drilling the hole was caved at a depth of 18.5 feet below existing grade.								
35											

BORING DRILLED TO 29 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 11 FEET WHILE DRILLING ▽

WATER LEVEL AT - FEET AFTER - HOURS ▼

DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-5

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-5

DATE SEPTEMBER 22, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION Elevation: N/A	BLOWS/FOOT	M.C. %	U.D.W. PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 9" Concrete and 7" Base (stabilized silty sand)							
			Very stiff dark gray Sandy Clay (CL) with ferrous nodules		13			23	14	9
			Stiff 2'- 6'		19	107				
5			Light gray & tan 4'- 20'		16	112	52	39	19	20
			with calcareous nodules 6'- 8'		25	103				
					24					
10										
			-very sandy @ 13'- 20'		22	105		29	16	13
15										
			-light gray 18'- 29'		21	84		28	14	14
20										
					17	114	80			
25										
					15	117				
30			Bottom of Boring at 30'							
			Groundwater was encountered at a depth of 17 feet, and 1/2 hour after completion of drilling it was measured at a depth of 19.5 feet below existing grade.							
35										

SHEAR STRENGTH, TSF
○ POCKET PENETROMETER
△ CONFINED COMPRESSION
● UNCONFINED COMPRESSION
□ TORVANE
0.5 1 1.5 2

BORING DRILLED TO 30 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 17 FEET WHILE DRILLING ▽
WATER LEVEL AT 19.5 FEET AFTER 1/2 HOURS ▼
DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-6

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-6

DATE SEPTEMBER 19, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 9" Concrete and 4" Base				○ POCKET PENETROMETER △ CONFINED COMPRESSION ● UNCONFINED COMPRESSION □ TORVANE 0.5 1 1.5 2				
			Soft gray Sandy Clay (CL) with ferrous nodules Very stiff 2'- 4'		22	100		67	25	14	11
					17						
			Very soft 4'- 6'								
5					24	100					
			Very stiff tan & light gray 6'- 20'					78	46	21	25
					19						
					21	102					
10											
			with sand seams 13'- 15'					50	30	16	14
					20	104					
15							3.20				
					15	113					
20			Bottom of Boring at 20'								
			Water was encountered below the pavement.								
25											
30											
35											

BORING DRILLED TO 20 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT - FEET WHILE DRILLING ▽
WATER LEVEL AT 1.00 FEET AFTER - HOURS ▽
DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-7

AVLES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: KELLEY STREET ASWTP BORING B-7

DATE SEPTEMBER 11, 2000 TYPE 3" CORE LOCATION SEE PLATE 1

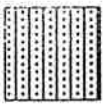
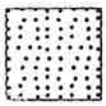
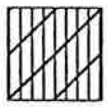
DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS/FOOT	M.C. %	U.D.W. PCF	SHEAR STRENGTH, TSF (○ POCKET PENETROMETER, △ CONFINED COMPRESSION, ● UNCONFINED COMPRESSION, □ TORVANE; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0			Pavement: 8" Concrete and 10" Base (stabilized silty sand)								
			Stiff gray Sandy Clay (CL) with calcareous and ferrous nodules Very stiff 2'- 8' Gray & tan 2'- 10'		13	116			31	17	14
5					14						
					16	114					
					14			68	35	18	17
10					19	111					
			Very stiff red & light gray Clay (CH) with calcareous and ferrous nodules		20	107		85			
15											
			Stiff 18'- 20'		19	110					
20											
25			with silt stone 26'- 28'		30	99					
			Bottom of Boring at 28'								
30			Groundwater was encountered at a depth of 11 feet, and 1/2 hour after completion of drilling it was at a depth of 20 feet below existing grade.								
35											

BORING DRILLED TO 28 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 11 FEET WHILE DRILLING ▽
WATER LEVEL AT 20 FEET AFTER 1/2 HOURS ▽
DRILLED BY J.H. CHECKED BY A.Z. LOGGED BY J.H.

PROJECT NO. G294-00

PLATE A-8

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Silty sand
	Low plasticity clay
	Poorly graded sand
	Silt
	High plasticity clay
	Silty low plasticity clay
Misc. Symbols	
▽	Water table during drilling
▼	Water table at boring completion
□	Torvane
●	Unconfined Compression
△	Confined Compression
○	Hand Penetrometer

Symbol	Description
Soil Samplers	
	Standard penetration test
	Shelby tube sampler

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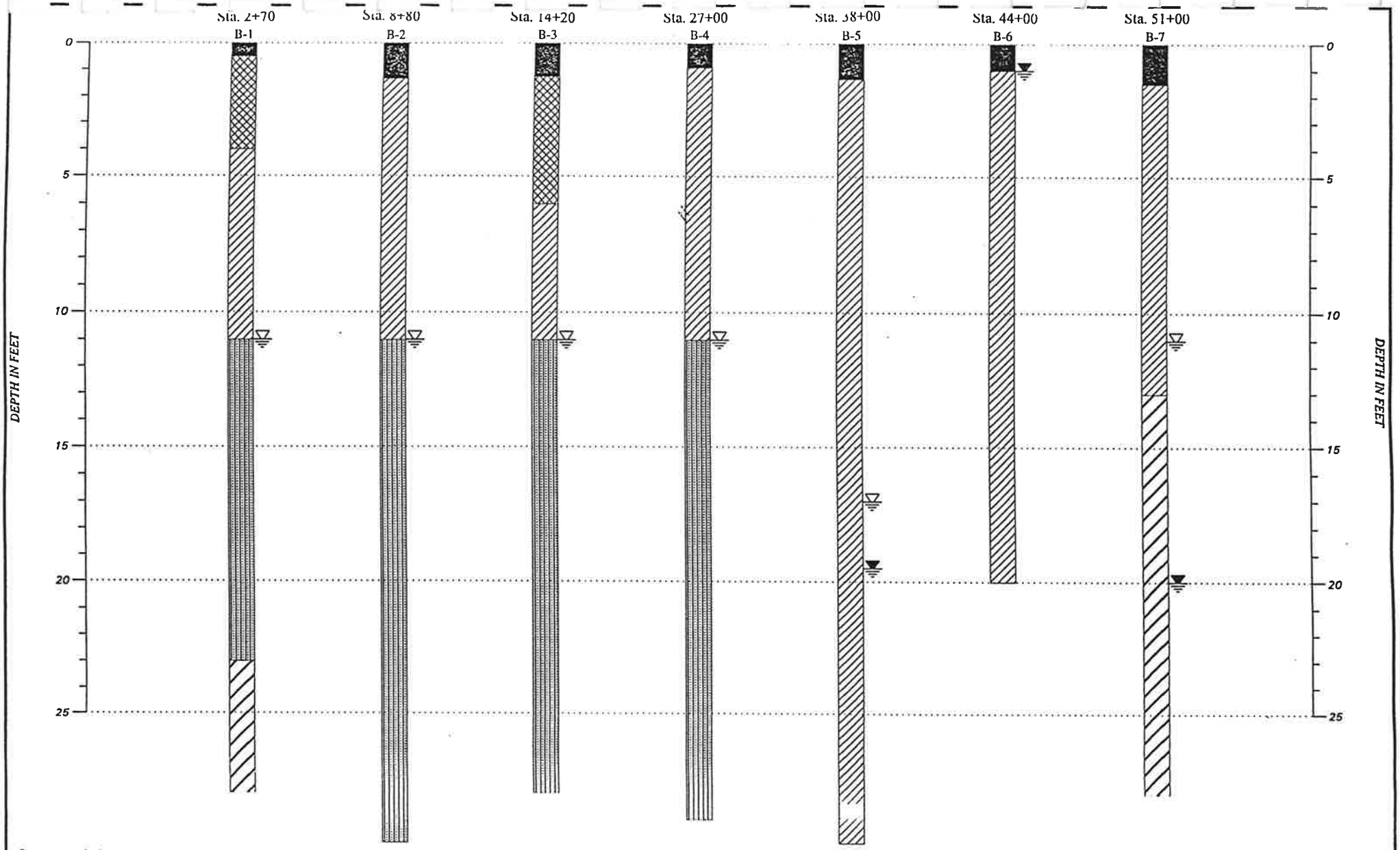
ASTM DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION
Moisture Content	D 2216
Specific Gravity	D 854
Sieve Analysis	D 421 D 422
Hydrometer Analysis	D 422
Minus No. 200 Sieve	D 1140
Liquid Limit	D 4318
Plastic Limit	D 4318
Shrinkage Limit	D 427
Standard Proctor Compaction	D 698
Modified Proctor Compaction	D 1557
Permeability (constant head)	D 2434
Consolidation	D 2435
Direct Shear	D 3080
Unconfined Compression	D 2166
Unconsolidated-Undrained Triaxial	D 2850
Consolidated-Undrained Triaxial	D 4767
California Bearing Ratio	D 1883
Unified Soil Classification System	D 2487



Strata symbols

- Paving
- Fill
- Low plasticity clay
- Silty sand
- High plasticity clay

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GENERALIZED SOIL PROFILE

HORIZONTAL SCALE: 1" = (proportional)'	DRAWN BY/APPROVED BY	DATE DRAWN
VERTICAL SCALE: 1" = 5'		1/5/2001

ASWTP - KELLY STREET

PROJECT NO. G294-00	FIGURE NUMBER A-11

APPENDIX B

Plates B-1 and B-2 Piezometer Installation Details

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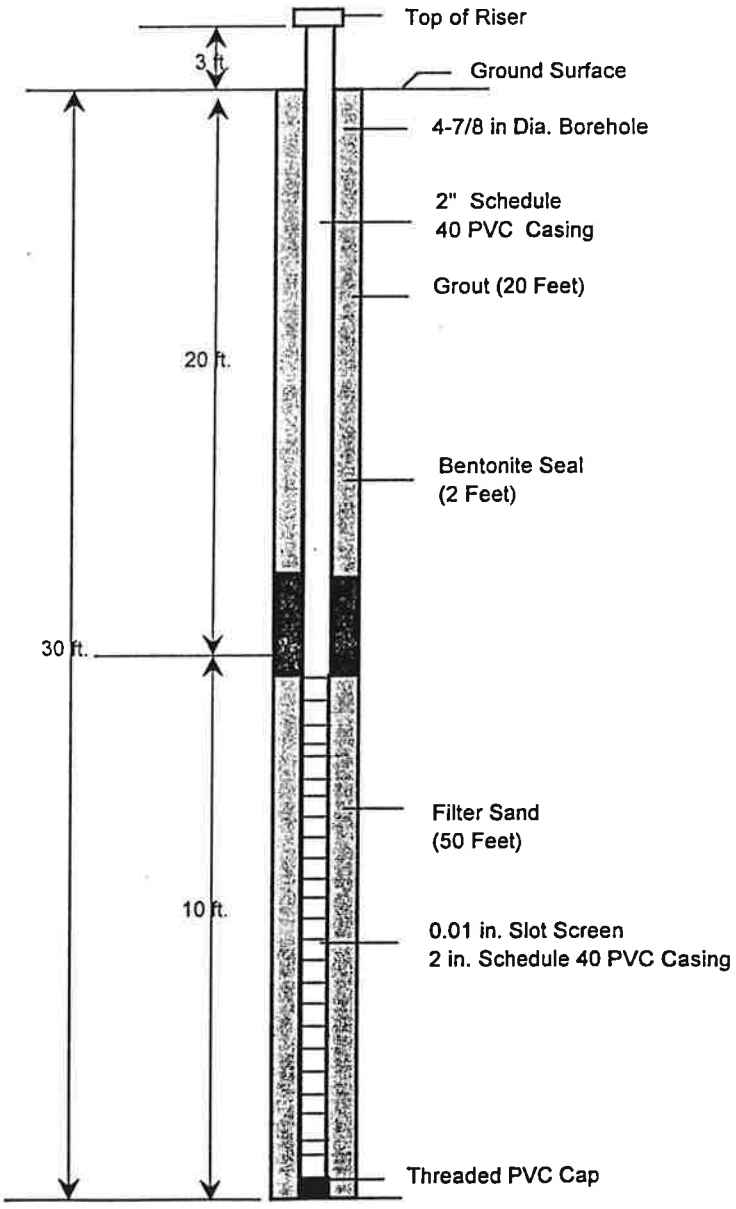
Piezometer No. : B -2 AEC Project No.: G294-00

Project Name: Accelerated Surface Water Transmission Program - Kelly Street

Ground Surface Elevation: N/A Location: See Plan of Borings



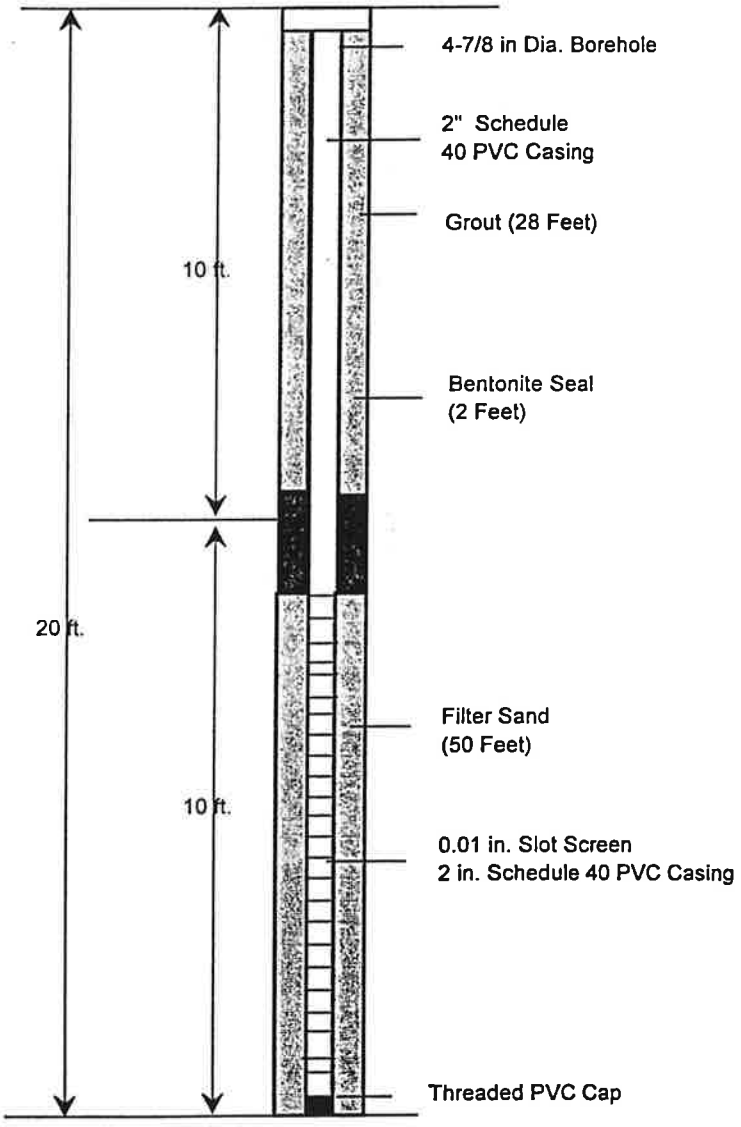
PIEZOMETER INSTALLATION DETAILS
(Not to Scale)

AVLES
ENGINEERING CORP.

Piezometer No. : B - 6 AEC Project No.: G294-00

Project Name: Accelerated Surface Water Transmission Program - Kelly Street

Ground Surface Elevation: N/A Location: See Plan of Borings



PIEZOMETER INSTALLATION DETAILS
(Not to Scale)

APPENDIX C

Plate C-1 OSHA Soil Classification for Excavation Trenching and Shoring

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OSHA SOIL CLASSIFICATION FOR EXCAVATING, TRENCHING AND SHORING

BORING	0-5'	5'-10'	10'-15'	15'-20'	20'-25'	25'-30'
B-1	C	B	C	C	C	C
B-2	C	B	C	C	C	C
B-3	C	C	C	C	C	C
B-4	B	B	C	C	C	C
B-5	B	B	C	C	C	C
B-6	C	C	C	C	-	-
B-7	B	B	B	B	B	B

Soil Type Criteria

A: q_u = 1.5 tsf or greater
B: q_u = 0.5 tsf or greater
C: Less than 0.5 tsf or granular soils

Note: q_u = Unconfined Compressive Strength in tsf

APPENDIX D

Plate D-1	Critical Heights
Plate D-2	Maximum Allowable Slopes
Plate D-3	A Combination of Bracing and Open-Cut
Plate D-4	Logarithmic Spiral Method For Calculating Earth Pressure Against Bracing
Plate D-5	Lateral Pressure Diagrams - For Open-Cuts in Cohesive Soils - Long Term Conditions
Plate D-6	Lateral Pressure Diagrams - For Open-Cuts in Cohesive Soils - Short Term Conditions
Plate D-7	Lateral Pressure Diagrams - For Open-Cuts in Sand - Long Term Conditions
Plate D-8	Lateral Pressure Diagrams - For Open-Cuts in Sand - Short Term Conditions
Plate D-9	Bottom Stability for Braced Excavation
Plate D-10	Lateral Pressure Diagram - Open Cuts at S.P. Railroad - Long Term Condition
Plate D-11	Uplift Pressure and Resistance
Plate D-12	Thrust Force On A Bend
Plate D-13	Thrust Force Example Calculation

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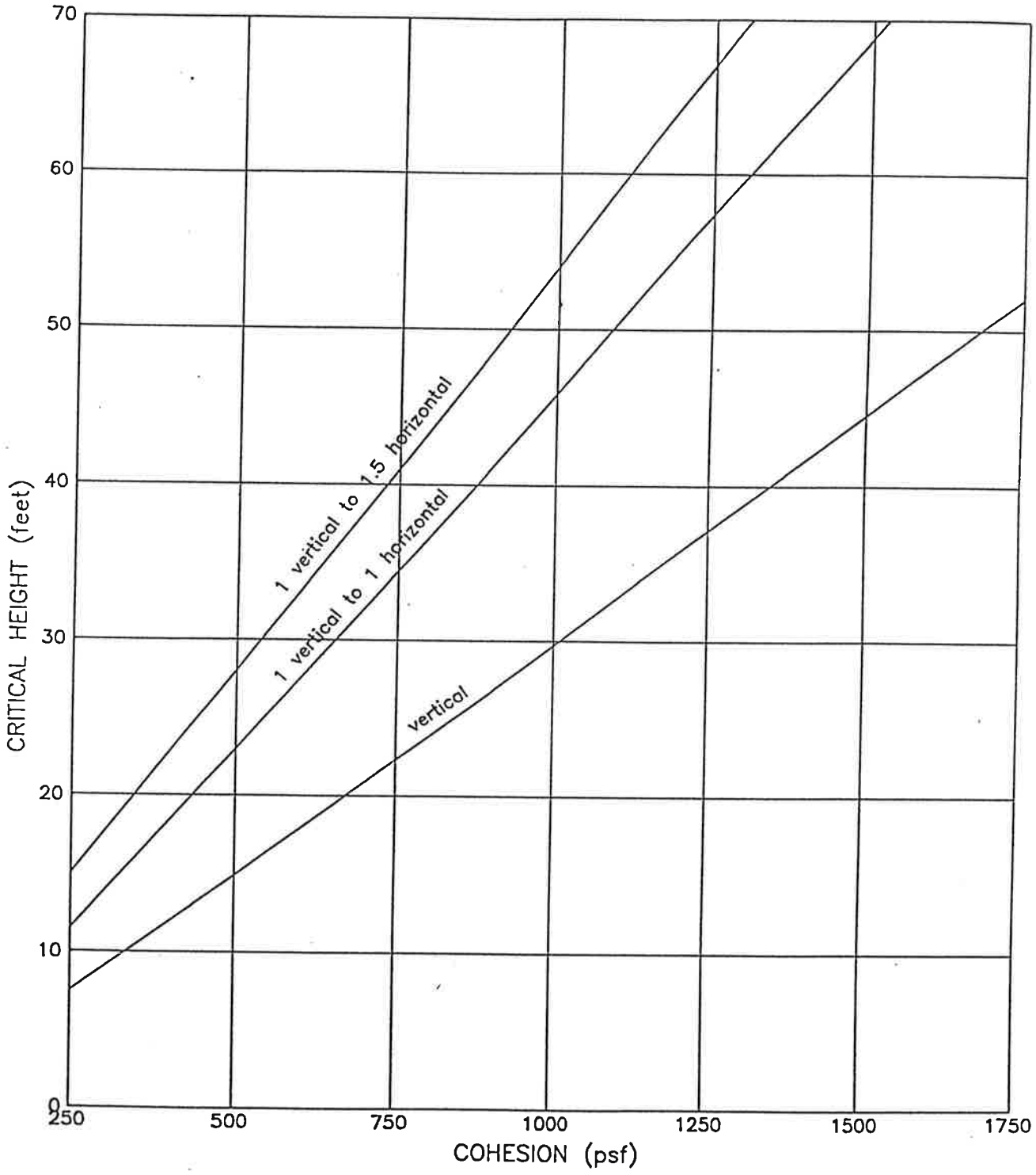

Critical Heights of Cut Slopes in Stiff to Very Stiff Clays

PLATE D-1

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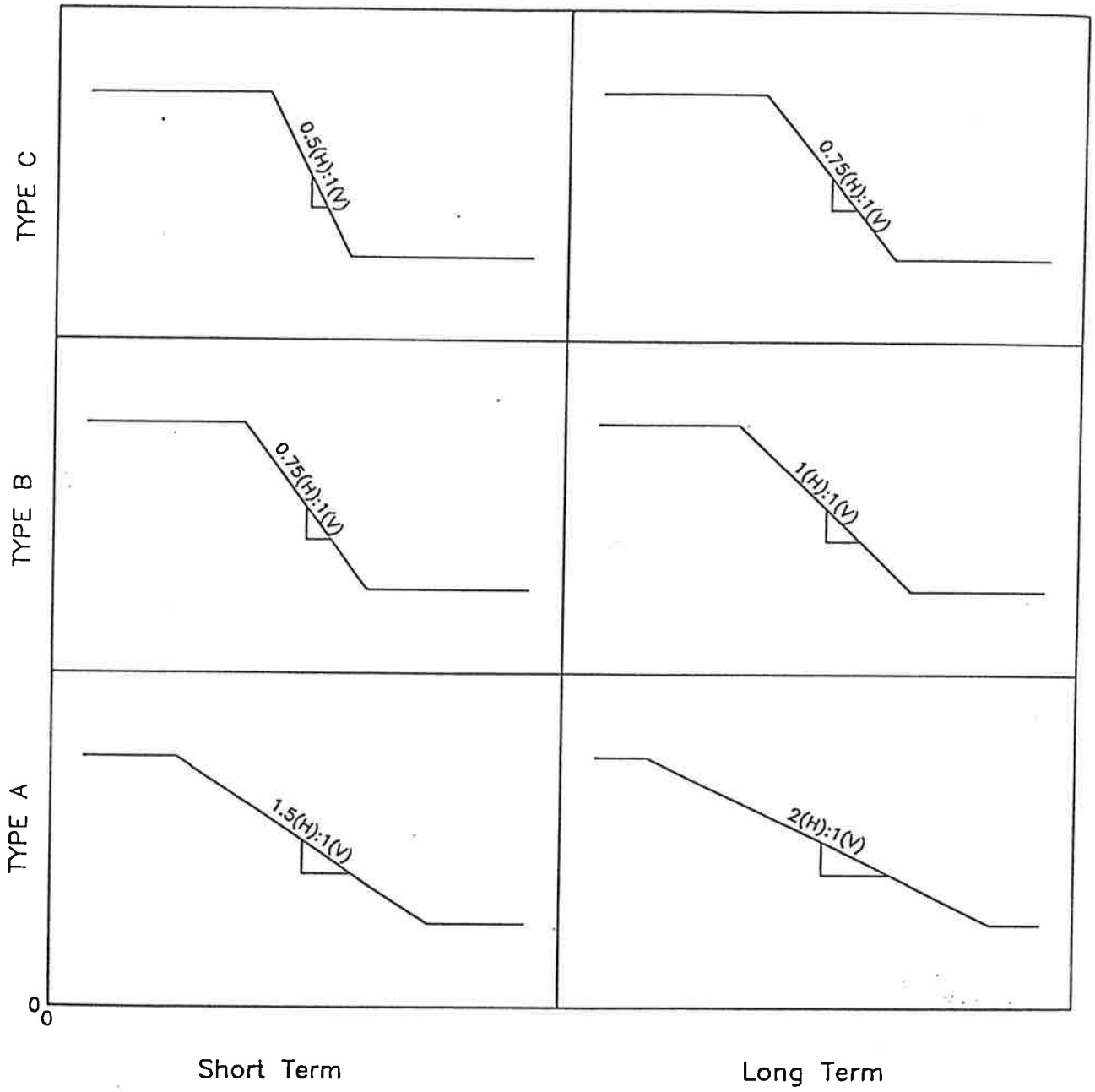

MAXIMUM ALLOWABLE SLOPES

A short term maximum allowable slope of 0.5(H):1(V) is allowed in excavations that are twelve feet or less in depth. Short term maximum allowable slopes for excavations greater than twelve feet in depth shall be 0.75(H):1(V)

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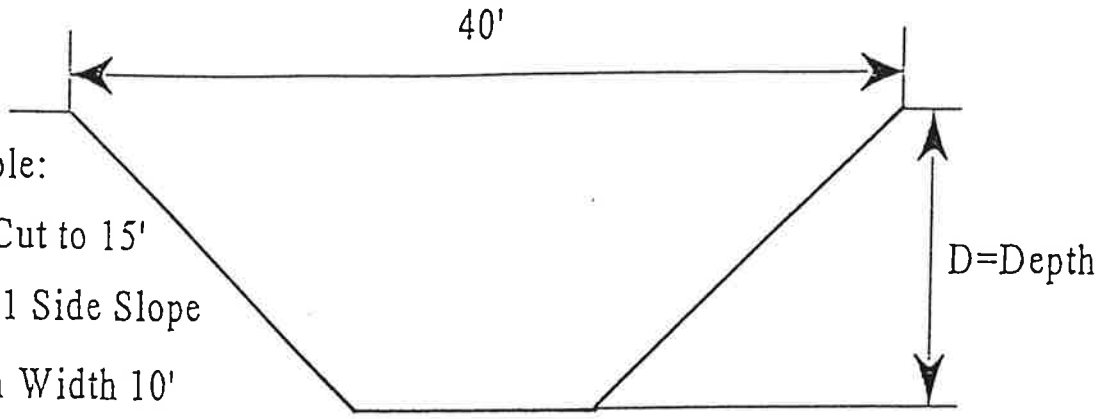
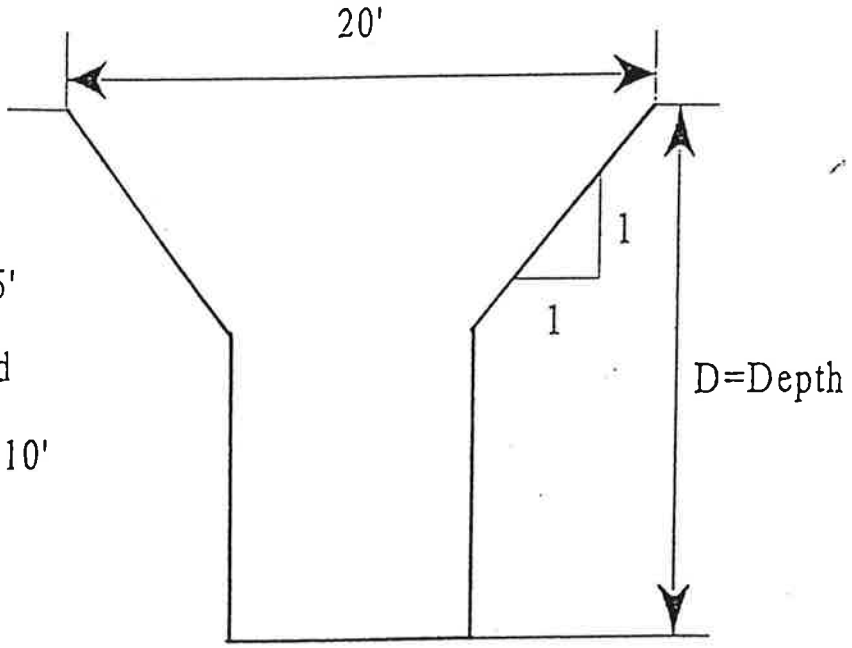
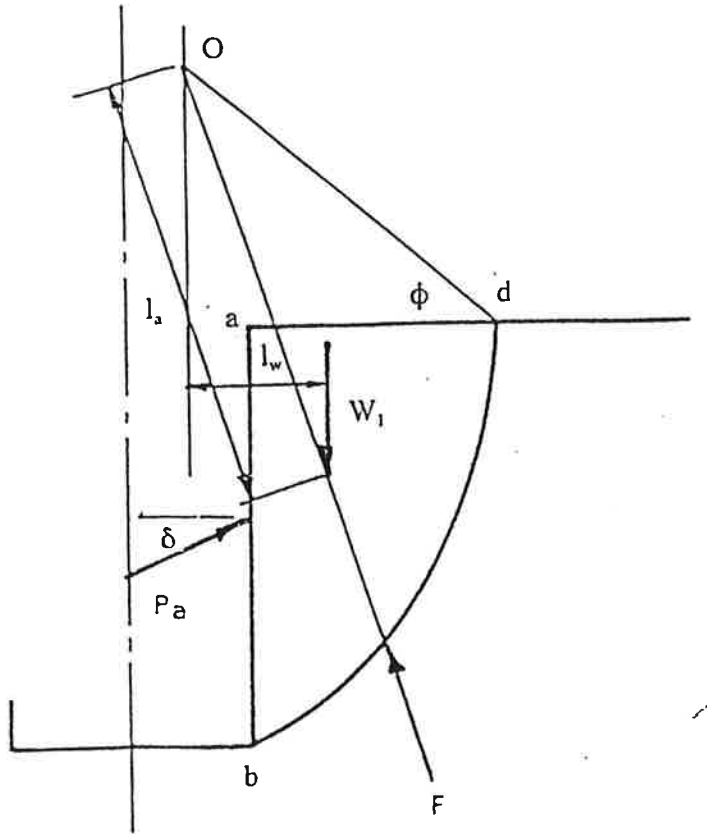


Example:
Open Cut to 15'
with 1:1 Side Slope
Bottom Width 10'



Example:
Open Cut to 15'
with 10' Braced
Bottom Width 10'

A COMBINATION OF BRACING AND OPEN CUT



Where,

O = Center of spiral wedge abd,
W_1 = Weight of wedge abd,
P_a = Maximum pressure,
F = Reaction of surface of sliding passing through center O,
l_a = Moment arm for P_a,
l_w = Moment arm for W_1.
δ = Angle between resultant stress on plane and nomal to plane,
ϕ = Angle of internal friction of soil.

LOGRITHMIC SPIRAL METHOD FOR CALCULATING EARTH PRESSURE AGAINST BRACING

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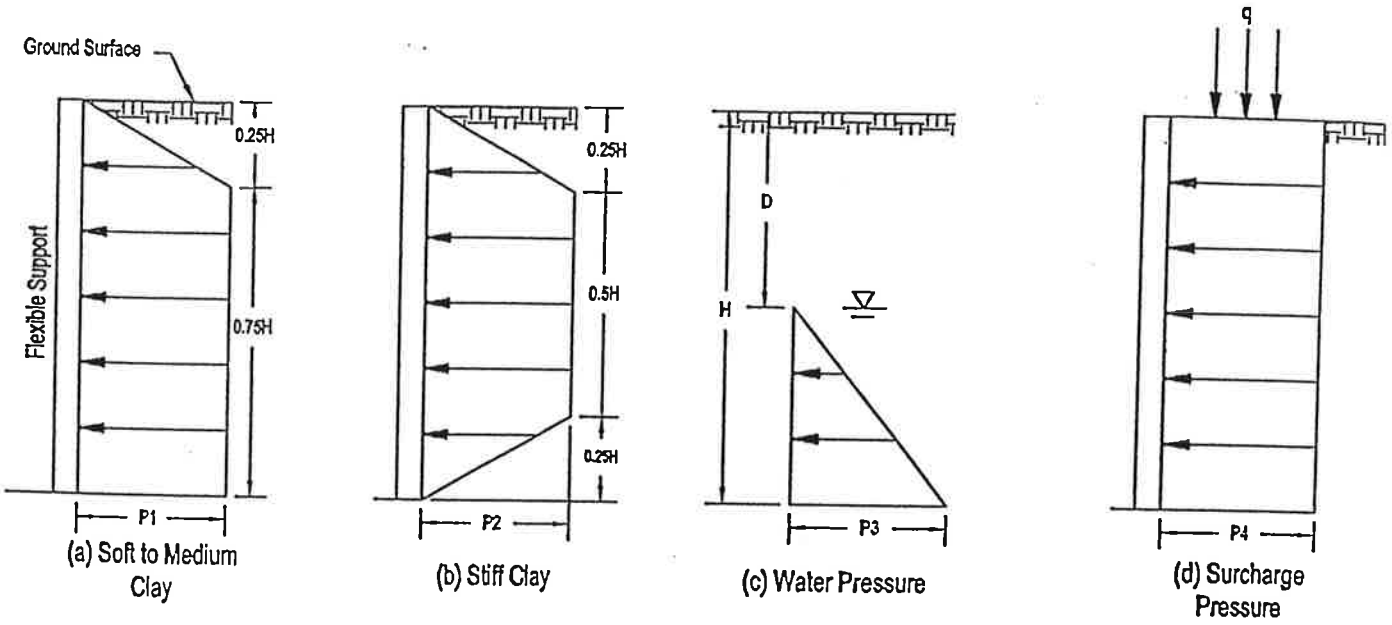
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOILS - LONG TERM CONDITIONS



Empirical Pressure Distributions

where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H - 4C$, psf
P2 = Lateral earth pressure = $0.4\gamma H$, psf
P3 = Water pressure = $\gamma_w(H-D)$, 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 pressure are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If $\gamma H/C \leq 4$, use section (b),
   If $4 < \gamma H/C < 6$, use larger of sections (a) or (b),
   If $\gamma H/C > 4$, use section (a).

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

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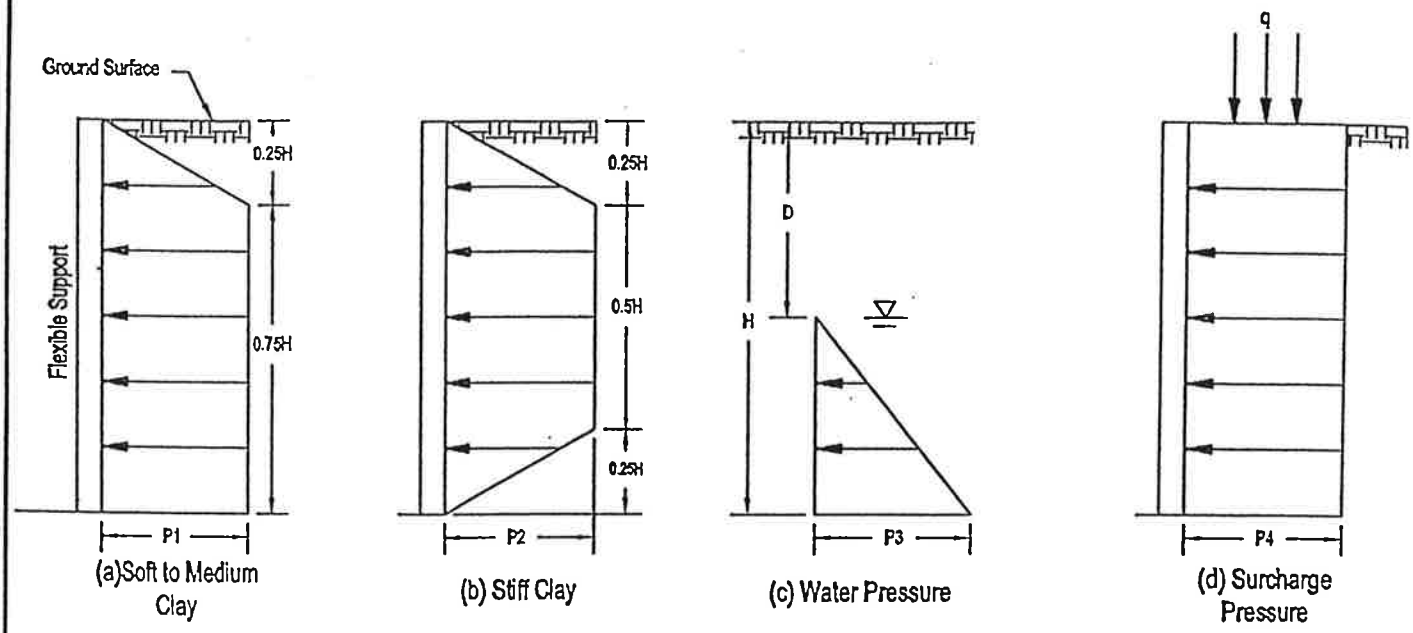
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOILS - SHORT TERM CONDITIONS



Empirical Pressure Distributions

where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H - 4S_u$, psf
P2 = Lateral earth pressure = $0.2\gamma H$, psf
P3 = Water pressure = $\gamma_w(H-D)$, 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 pressure are additive.
2. No safety factors are included.
3. For use only during short term construction.
4. If $\gamma H/S_u \leq 4$, use section (b),
   If $4 < \gamma H/S_u < 6$, use larger of sections (a) or (b),
   If $\gamma H/S_u > 4$, use section (a).

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

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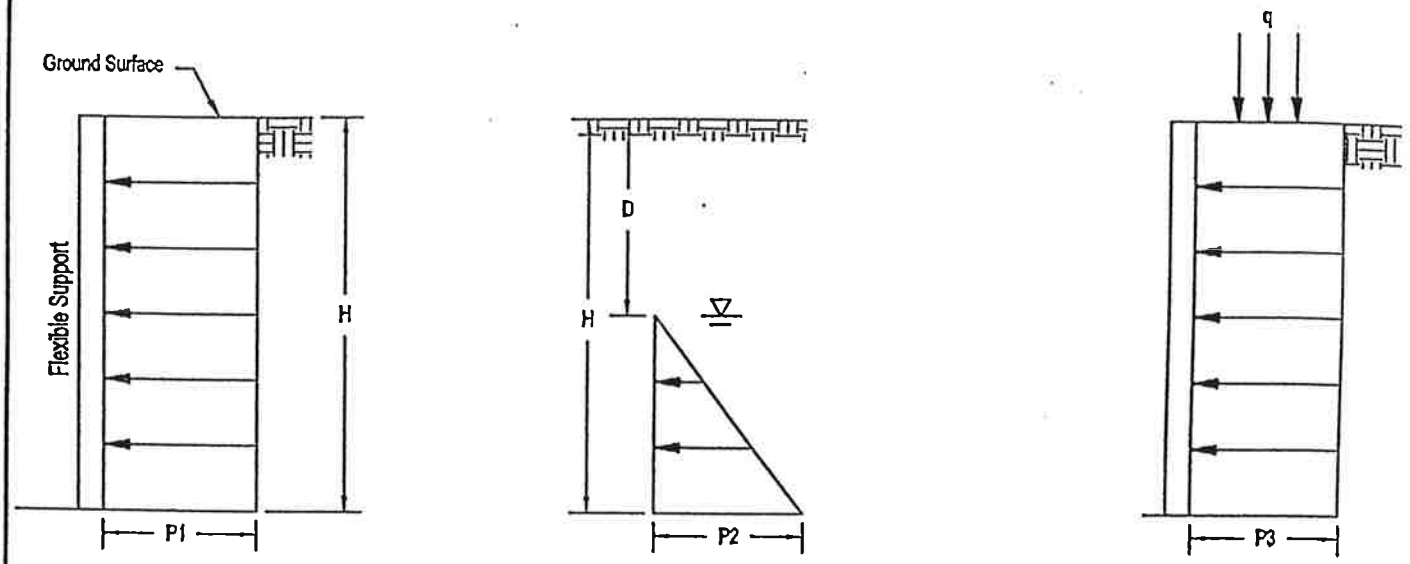
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND - LONG TERM CONDITIONS



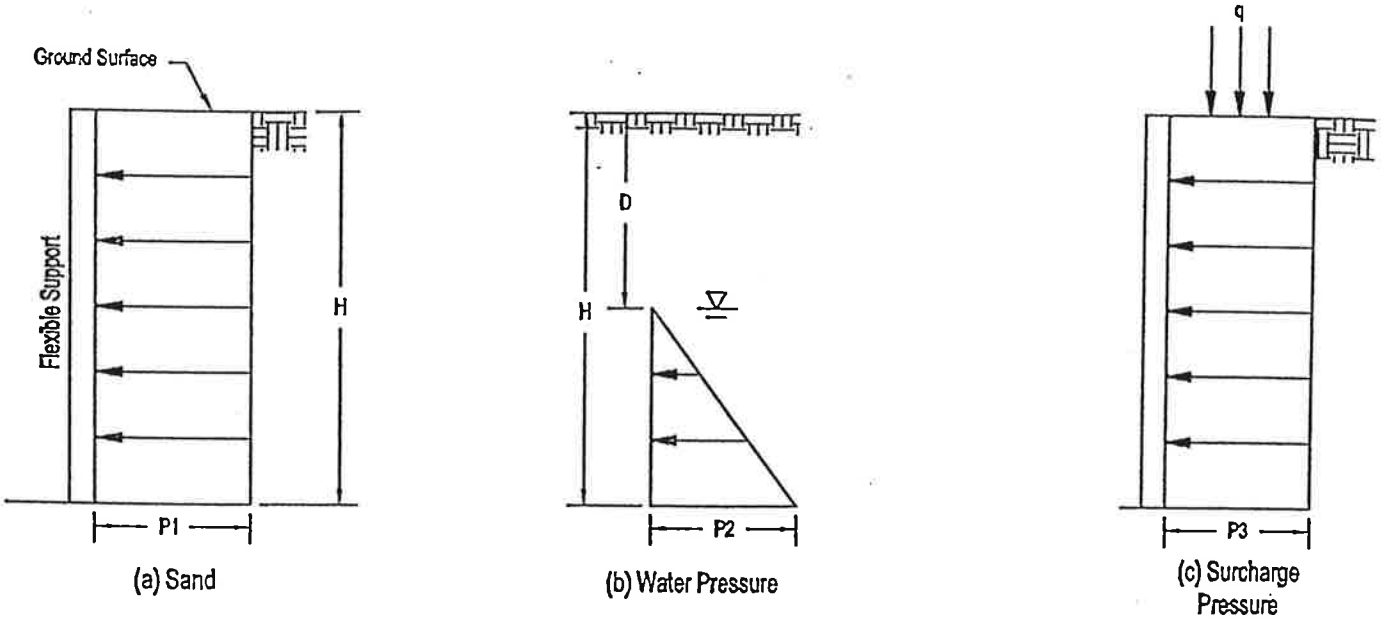
Empirical Pressure Distributions

where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma HK_a$, psf
P2 = Water pressure = $\gamma_w(H-D)$, 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 pressure are additive.
2. No safety factors are included.
3. For use only during long term construction.

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

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND - SHORT TERM CONDITIONS



Empirical Pressure Distributions

where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma HK_a$, psf
P2 = Water pressure = $\gamma_w(H-D)$, 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 = Undrained friction angle

Notes:
1. All pressure are additive.
2. No safety factors are included.
3. For use only during short term construction.

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

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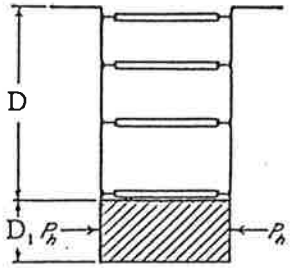
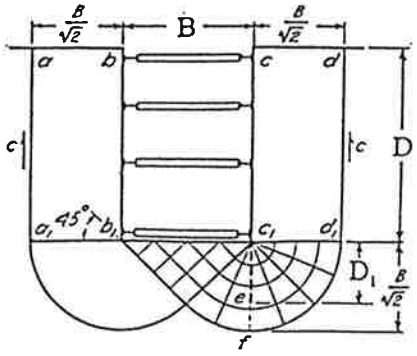




Factor of Safety against bottom heave,

$$F.S = \frac{Nc.C}{(\gamma.D + P)}$$

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,
P = Surface surcharge.

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

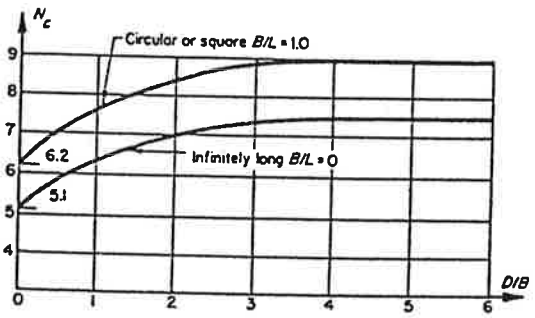
$$Depth of Buried Length, (D_1) = \frac{1.5(\gamma.D + P) - Nc.C}{(C/B) - 0.5\gamma}; D_1 \geq 5ft.$$

Pressure on buried length, P_h:

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

For $D_1 > 0.47B$; $P_h = 0.7(\gamma.D.B - 1.4\ C.D - 3.14C.B)$

where, B = width of excavation



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

BOTTOM STABILITY FOR BRACED EXCAVATION

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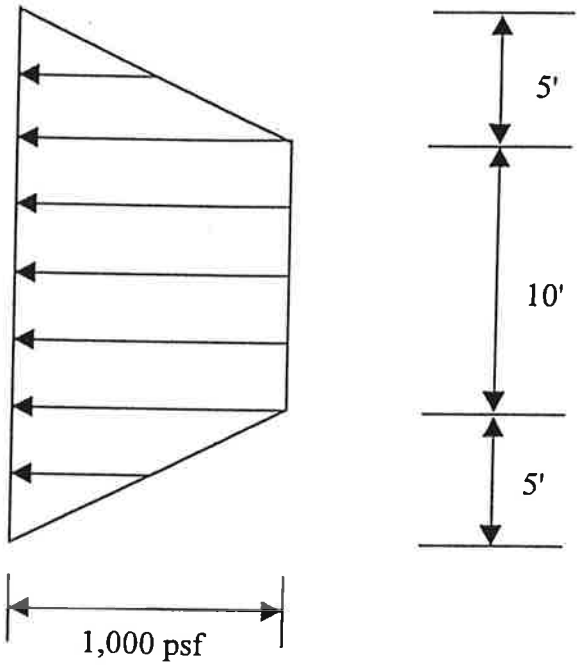
LATERAL PRESSURE DIAGRAM
FOR OPEN CUTS IN COHESIVE SOILS
LONG-TERM CONDITIONS
UNION PACIFIC RAILROAD (Ref: Boring B-7)



The lateral pressures shown do not include hydrostatic pressure or pressure due to surcharge.

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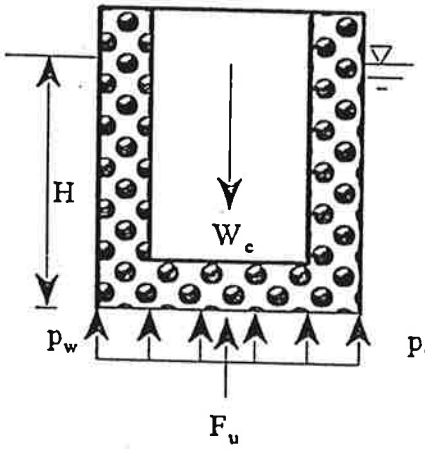
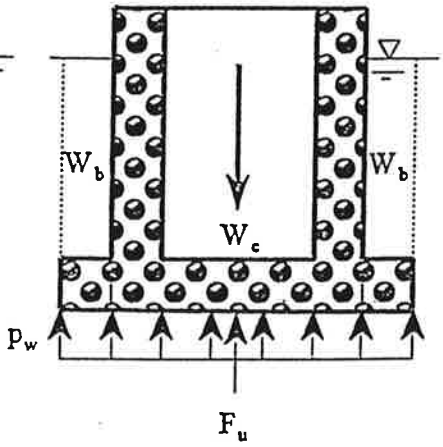
(a) Dead Weight of Structure



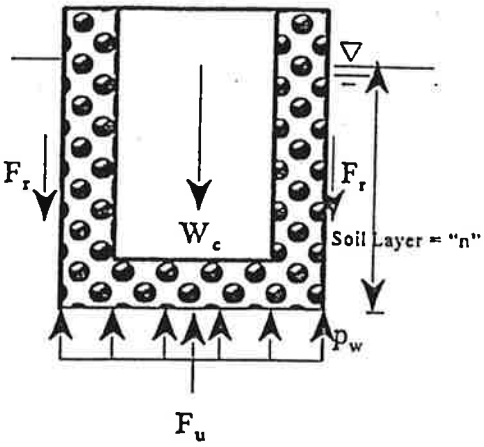
$p_w = H\gamma_w$
$F_u = A_b p_w$
$W_c = S_f F_u$

(b) Weight of Backfill Above Base Extension Plus Dead Weight of Structure



$p_w = H\gamma_w$
$F_u = A_b p_w$
$W_c + W_b = S_f F_u$

(c) Soil -Wall Friction Plus Dead Weight of Structure



$p_w = H\gamma_w$
$F_u = A_b p_w$
$W_c + F_r = S_f F_u$

For Cohesive Soils, $F_r = \alpha c_n A_n$
For Cohesionless Soils, $F_r = p_n K \tan\delta_n A_n$

Where:

A_b	=	area of base, sq.ft.
A_m	=	cylindrical surface area of layer "n", sq.ft.
c_m	=	undrained cohesion of layer "n", psf
F_u	=	hydrostatic uplift, lbs.
F_r	=	friction resistance, lbs
H	=	height of buried structure, ft
K	=	coefficient of lateral pressure = 0.5
p_m	=	average overburden pressure for layer "n", psf
p_w	=	hydrostatic uplift pressure, psf
S_f	=	factor of safety
W_b	=	Weight of backfill above base extension, lbs
W_c	=	dead weight of concrete structure, lbs
α	=	adhesion factor = 0.5
δ_n	=	friction angle between soil layer "n" and concrete wall, degrees = $0.75\phi_n$
γ_w	=	unit weight of water = 62.4 pcf

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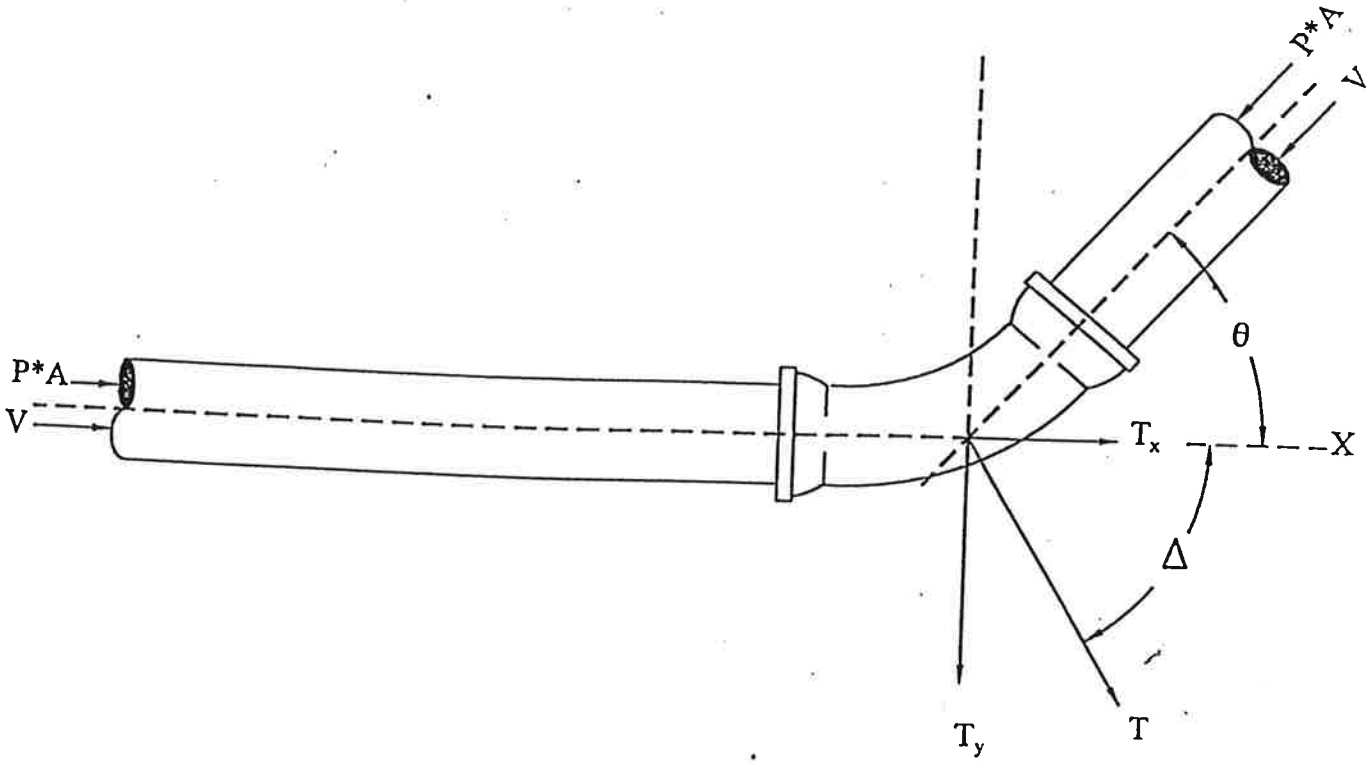
Thrust Force on a Bend



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

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

$T_y = P*A*SIN\theta$

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

Where:

T = resultant thrust force
T_x = thrust force component along the X axis
T_y = thrust force component along the Y axis
P = maximum sustained pressure
A = cross-sectional area of pipe = $(\Pi/4)*(D)^2$
D = inside diameter of conduit
θ = angle of bend
Δ = angle between X axis and T
V = fluid velocity

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Thrust Force Example Calculation

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

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

$T_y = P*A*SIN\theta$

Where:

T = resultant thrust force
T_x = thrust force component along the X axis
T_y = thrust force component along the Y axis
P = maximum sustained pressure
A = cross-sectional area of pipe = $(\Pi/4)*(D)^2$
D = inside diameter of conduit
θ = angle of bend

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

Find: T, T_x and T_y

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

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

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

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