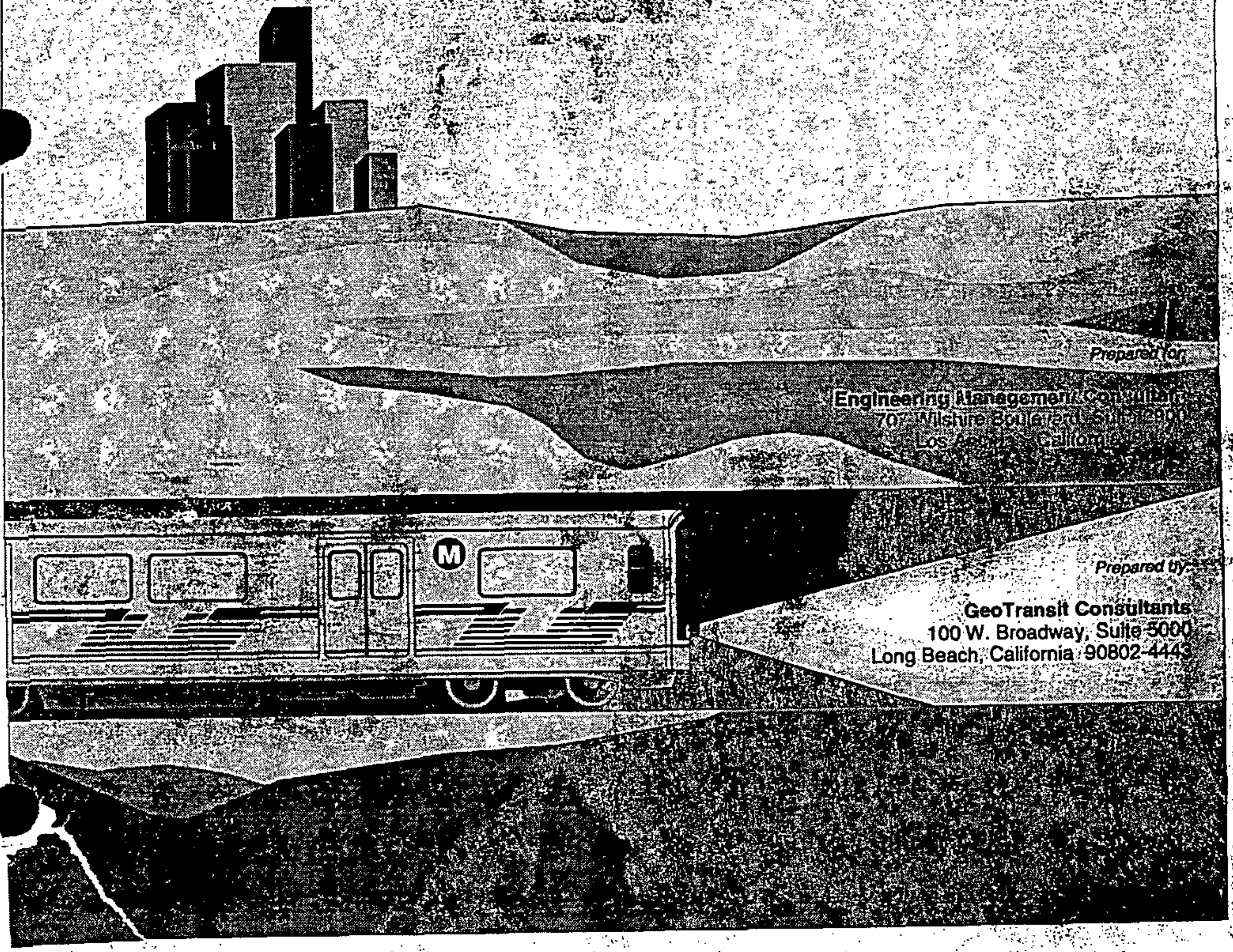
Final Geotechnical Investigation for:

First/Lorena Station & Tail Track Tunnels
Eastside Extension
Metro Red Line Project



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FIRST/LORENA STATION WITH CROSSOVER AND CONNECTING TAIL TRACK TUNNELS
EASTSIDE EXTENSION
METRO RED LINE

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1.0 EXECUTIVE SUMMARY

1.1 GENERAL

This report presents the results of a geotechnical investigation conducted by GeoTransit Consultants for the planned First/Lorena Station with crossover and the connecting Tail Track Tunnels. The station and tunnels are part of the proposed Eastside Extension of the Los Angeles Metro Red Line. The primary purposes of this investigation were to evaluate geologic and geotechnical conditions and to obtain geotechnical data for planning and design of the station and tunnels. The locations, dimensions, and configurations of the proposed station and tunnels covered in this report were based on plans provided by the Engineering Management Consultant (EMC), at the time of this investigation.

1.2 PROJECT DESCRIPTIONS

The proposed First/Lorena Station with crossover consists of a cut-and-cover reinforced concrete structure approximately 60 feet in width and 877 feet in length. The station invert ranges from about 60 to 78 feet below ground surface (BGS). The station is located within the East First Street right-of-way, from about 410 feet west to 467 feet east of the center line of Lorena Street.

The Tail Track Tunnels consist of two single track, 18-foot inside and 22-foot outside diameter openings in a parallel double line configuration. Starting from the eastern terminus of the First/Lorena Station, the tunnel alignments extend eastward along East First Street to approximately the First Street/Indiana Street intersection where the tunnels curve southeasterly to the southeast terminus of the tunnel alignment. The tunnels are about 745 feet long and their invert elevations range from approximately 69 to 85 feet BGS.

1.3 SCOPE

The scope of this investigation consisted of reviewing available literature; conducting a site reconnaissance and preparing a geologic map; performing field explorations including drilling 8 rotary wash borings, installing 2 monitoring wells; monitoring groundwater levels; conducting 12 pressuremeter tests in a single borehole, and two downhole seismic velocity surveys; performing a geotechnical laboratory testing program on selected soil and bedrock samples; conducting an engineering evaluation; and preparing this report.

1.4 GEOLOGIC SETTING

The Eastside Extension alignment is located along the southern flank of the Repetto Hills area of the Los Angeles Basin. In the project area, the station and tunnels will be in alluvial deposits of Holocene and Pleistocene age. The alluvium overlies Fernando/Puente Formation bedrock, and consists of mostly coarse granular deposits with local cobbles and possibly small boulders and fine-grained interbeds of variable thickness. Bedrock consists of siltstone, claystone and occasional sandstone with local, hard, well-cemented zones.

The alignment is located in an area having a high seismic potential and has experienced ground shaking from a number of large earthquakes in historical time. The documented active faults closest to the alignment are the east-west trending Hollywood and Raymond faults about 5 miles northwest and 4 miles northeast of the alignment, respectively. The area is underlain by the Elysian Park seismic zone, generally accepted as the source of the 1987 Whittier Narrows earthquake. The seismic zone is postulated to be a concealed, deep thrust fault that in part expresses itself at the surface as the Elysian Hills and Repetto Hills.

No known active or inactive faults cross or project toward the project area. A linear topographic scarp and possibly displaced drainages suggest the presence of faults ("Coyote Pass escarpment" and unnamed faults associated with it) located approximately 2000 feet north of the project area. This escarpment forms the southern margin of the City Terrace area in the Repetto Hills. The "Coyote Pass escarpment" is considered to be active.

1.5 SUBSURFACE STRATIGRAPHY

The planned tunnel and station excavations will be within alluvium. The alluvium is heterogeneous and consists of predominantly coarse-grained materials ranging from sands to gravel. Local zones of cobbles are present in the station area. Interbedded layers of fine-grained soils of variable thicknesses in the project area consist predominantly of lean clays, sandy silt and fat clay with variable amounts of sand and silt.

Bedrock units of the Fernando/Puente Formation underlie the alluvium. In the station area bedrock is estimated to be at least 28 feet below station invert. The bedrock materials, where encountered in the station borings, consist predominantly of soft to very soft and poorly bedded siltstone and claystone.

1.6 GROUNDWATER LEVELS

The most recent observed groundwater levels are approximately 80 to 100 feet BGS in the project area. The groundwater table dips to the south and southeast at an average gradient of about one percent. Assuming the groundwater levels during construction are the same as the most recently measured data, the station excavation will be at or above groundwater level, and tunnel excavation will be above groundwater level.

The project area is located in a region with known significant fluctuations in historic groundwater levels. Available groundwater-level data from a number of wells in the general region (maintained and recorded by the Los Angeles County Department of Public Works) indicates that groundwater levels in the general region can significantly fluctuate. At some of the well locations, the observed groundwater level fluctuations were as much as 12 feet in one month, 69 feet in one year, and 183 feet in 10 years. Thus, significant fluctuations of groundwater levels should be anticipated during the life of the station and tunnel facilities. Such potential groundwater fluctuation must be considered in the construction and design. It is important that groundwater levels in the project area be monitored prior to and during construction.

1.7 GROUNDWATER AND SOIL CONTAMINATION

Available chemical test data (GeoTransit Consultants, 1994b) on soil samples from two environmental borings in the project area and one water quality sample from a nearby environmental monitoring well detect no significant contamination by hydrocarbons, hydrogen sulfide or a number of commonly encountered constituents. However, because of its proximity to the existing Boyle Heights oil field, there exists a potential for local soils and perched water-related local inflows to be contaminated with oil field-related contaminants (i.e., hydrocarbons, hydrogen sulfide, etc.). These local contaminations if encountered, will require treatment and disposal to approved facilities.

1.8 GASSY CONDITIONS

Although no sulfurous or hydrocarbon odors were detected in the work space from samples above station or tunnel inverts during this investigation, the proximity of the project area to the existing Boyle Heights oil field suggests the potential presence of methane and other oil field related gases in the project area.

1.9 FIRST/LORENA STATION WITH CROSSOVER

The station excavation will be primarily in heterogeneous alluvium. Based on available information from 5 station borings, the alluvium above station invert comprises roughly 75 percent granular and 25 percent fine-grained alluvial soils. It is anticipated that the cut-and-cover excavation can be achieved by conventional excavation methods. However, suitable excavation equipment to handle the local presence of cobbly zones and possibly local zones of small boulders, wil be required. Most of the alluvium in the station area may run or ravel readily. Thus, timely application of ground support is important.

If the groundwater levels during station construction are similar to the present groundwater levels, only minor dewatering near the bottom of the station excavation will be required during construction. However, groundwater levels should be regularly monitored prior to and during construction to verify this.

The planned station excavation will require vertical cuts and shoring due to proximity of the station to existing buildings, and limited construction space within the public right-of-way. Based on local practice in similar subsurface conditions, soldier piles with internal bracing and/or tiebacks, and timber, shotcrete or pre-cast reinforced concrete lagging are the most likely shoring systems. Design and construction of appropriate excavation support systems to ensure little or no ground loss and to provide a safe work site is the responsibility of the contractor. Recommended lateral earth pressures for design as well as various other design and installation considerations are provided in this report.

Based on the results of this investigation, it is our opinion that the site soils can adequately support the planned main station structure with acceptable total and differential settlements. Various geotechnical design parameters are provided for station design. Where appropriate, ranges of design parameters are provided to account for variability of the subsurface materials.

1.10 TAIL TRACK TUNNELS

Based on current plans and profiles, and current measured groundwater levels, the Tail Track Tunnels will be entirely in alluvium and above groundwater. It is anticipated that soft ground tunneling methods will be generally applicable. In the tunnel zone, the alluvium is predominantly granular and susceptible to raveling and running conditions during tunneling. Raveling and running conditions will slow the excavation progress and result in poor face stability and excessive ground loss problems unless proper face stability controls and settlement minimization measures are implemented.

1.11 SEISMIC DESIGN CRITERIA

No project-specific seismic hazard analyses were conducted for the Eastside Extension. A seismic hazard analysis was not part of the scope of this investigation. For geotechnical analyses and design purposes the results of a 1983 study titled "Seismological Investigation and Design Criteria" prepared by Converse and others for the Metro Rail Project were used. Geotechnical recommendations presented herein should be reviewed and revised as appropriate, should seismic criteria be revised in the future.

1.12 LIQUEFACTION POTENTIAL

Liquefaction potential of subsurface soils in the tunnel segment and station area was evaluated. The results indicate that the potential for liquefaction of the soils in the project area under the maximum design earthquake (based on the 1983 "Seismological Investigation and Design Criteria" report for the Metro Rail Project) is very low and is not a consideration for design.

2.0 INTRODUCTION

2.1 GENERAL

This report presents the results of a geotechnical investigation for the proposed First/Lorena Station with crossover and the connecting Tail Track Tunnels. The station and tunnel segment are part of the proposed Eastside Extension of the Los Angeles Metro Red Line. The investigation was performed to support the engineering efforts being undertaken by Engineering Management Consultant (EMC) for the Los Angeles County Metropolitan Transit Authority (MTA).

This geotechnical investigation is part of an overall geotechnical investigation for the design of the first portion of the proposed Metro Red Line Eastside Extension which begins at the southeastern terminus of the Union Station in Los Angeles, and ends at the First/Lorena Station and Tail Track Tunnels in East Los Angeles. This first portion of the Eastside Extension is approximately 3.7 miles in length and consists of twin tunnels and four cut-and-cover stations, including the First/Lorena Station and Tail Track Tunnels addressed in this report. Plate 1 presents the layout plan showing the locations of the station and tunnel segment with respect to the Eastside Extension alignment.

2.2 PROJECT DESCRIPTION

Locations, dimensions and configurations of the proposed station with crossover and the connecting Tail Track Tunnels in this report were based on available plan and profile drawings provided by EMC at the time of this investigation.

The plan and ground surface and station/tunnel profiles are shown in a plan and profile drawing presented in Section 4.0 (Plate 2). The following section describes the dimensions and configurations of the tunnel alignment and station. All elevations used in this report are with respect to the City of Los Angeles datum.

2.2.1 First/Lorena Station with Crossover

The proposed First/Lorena Station with crossover consists of a cut-and-cover reinforced concrete station structure and auxiliary facilities. The station with crossover starts at Station CR 188+00 (western terminus) and ends at Station CR 196+77 (eastern terminus). The station with crossover is about 877 feet in length with an inside width of about 60 feet. The crossover tracks start at the station's western terminus and ends at Station CR 191+71.71 where the platform structure begins. The platform structure ends at Station CR 196+21.71. The First/Lorena Station is located within the East First Street right-of-way between about 410 feet west and 467 feet east of the center line of Lorena Street.

The ground surface at the station site varies from approximate elevation 310 feet at the western terminus to approximate elevation 291.5 feet in the vicinity of First Street/Lorena Street intersection. The ground surface then slopes up to approximate Elevation 304 feet at the eastern terminus. The top-of-rail elevation within the station area is approximately 239 feet. The station invert is approximately at Elevation 232 feet. Thus, the depth of excavation for station construction varies from approximately 60 to 78 feet.

2.2.2 Tail Track Tunnels

The Tail Track Tunnels consist of two single track, 18-foot inside and 22-foot outside diameter openings in a double line configuration. Starting from the eastern terminus of the First/Lorena Station (Station CR 196+77), the tunnel alignment runs eastward along East First Street to the vicinity of the First Street/Indiana Street intersection where the tunnel alignment curves southeasterly to the southeastern terminus of the Tail Track Tunnels at Station CR 204+21.71. The east-west trending portion of the tunnel segment is within the First Street/Indiana Street right-of-way while the remaining southwesterly trending portion is under existing commercial and residential buildings.

The existing ground surface varies from approximate Elevation 304 feet at the western end of the tunnel segment (approximate Station CR 196+77) to about Elevation 319 at the southeastern corner of the First Street/Indiana Street intersection beyond which the ground elevation is at about Elevation 320 feet. The planned tunnel inverts are at elevation 235 feet. Therefore, the tunnel inverts are approximately 69 to 85 feet BGS.

2.3 OBJECTIVES AND SCOPE

The objectives of the geotechnical investigation were to evaluate subsurface soil and groundwater conditions and to obtain geotechnical data for planning and design of the proposed tunnels and station.

The scope of this investigation consisted of the following:

- Review of available literature and reports regarding the geologic, geotechnical, groundwater and seismic conditions at the station site and along the alignment.

- Planning and coordination of field work, including:

    - Development of field procedures and manuals
    - Planning of the field investigation program
    - Obtaining necessary permits and licenses
    - Coordination with government agencies and utility companies during the course of this investigation
    - Development and implementation of a project-specific Health and Safety Plan.

- Performance of a field exploration program, including:

    - Drilling and sampling seven rotary wash test borings
    - Performing 12 pressuremeter tests in one of the borings along the tunnel alignment
    - Installing two monitoring wells
    - Monitoring groundwater levels at all available monitoring well locations, including the two monitoring wells installed in this investigation and one monitoring well installed in the preliminary engineering program
    - Conducting two downhole seismic refraction geophysical surveys to evaluate shear wave velocity and dynamic modulus characteristics of the subsurface materials

- Performance of a laboratory testing program on selected representative soil samples to assess the index and engineering properties of subsurface materials

- Preparation of this report documenting the results of the geotechnical investigation and providing recommendations for project design.

2.4 PREVIOUS INVESTIGATIONS AND AVAILABLE DATA

A number of project-specific geologic, geotechnical, and environmental investigations were previously performed by GeoTransit Consultants in the project area. Results of these previous investigations were reviewed to plan and supplement the results of this investigation.

Previous project-specific investigations for the Eastside Extension performed by GeoTransit Consultants include the following:

- Preliminary geotechnical investigation (GeoTransit Consultants, 1994a) which includes drilling and sampling of one geotechnical boring (PE-13) in the station area, and installation of a monitoring well in the boring. Log of Boring PE-13 has been incorporated in Plate 2 (plan and profile) and is included in Appendix A of this report.

- Stage II Environmental Site Assessment, Eastside Extension, Metro Red Line Project (GeoTransit Consultants, 1994b) which includes environmental sampling in two environmental borings (EB-10 and EB-11) in the project area.

In addition to the above investigations performed by GeoTransit Consultants, Engineering-Science, Inc. (ESI) conducted a subsurface gas investigation to support preliminary design of the Eastside Extension (Engineering-Science, Inc., 1994). This gas investigation included installation, monitoring, and gas sampling and testing of two gas probes (deep and shallow) in one boring location (MW-6) in the First/Lorena Station area.

3.0 FIELD EXPLORATION, FIELD TESTING AND LABORATORY TESTING

3.1 GENERAL

This section provides a description of the subsurface exploration, field testing and laboratory testing performed for this program. The field exploration and field testing program are part of a larger overall geotechnical investigation program being performed for the entire Eastside Extension alignment. Applicable results from the overall geotechnical investigation program, and previous investigations in the project area (Section 2.4) were also used in developing findings and conclusions presented in this report.

3.2 GEOTECHNICAL BORINGS

Seven rotary-wash borings were drilled in the project area. Three of the borings (identified by the prefix DD-) were drilled within the tail track tunnel segment. Four of the borings (identified by the prefix SD-) were drilled within the First/Lorena Station area. Locations and penetration depths of these borings are summarized in Table 3-1 and Plate 2. For completeness, the geotechnical Boring PE-13 completed during the preliminary investigations in this area (GeoTransit Consultants, 1994a,) is also included in Table 3-1 and Plate 2.

The borings were logged in the field by a geologist or engineer under the direct supervision of a Registered Geotechnical Engineer (RGE) or a Certified Engineering Geologist (CEG). The materials were classified in general accordance with American Society of Testing and Materials (ASTM) Standards and the Unified Soil Classification System. The field logs were refined after laboratory examination and testing of selected soil samples. Boring logs are presented in Appendix A.

Rotary-wash borings for the geotechnical subsurface exploration program were drilled using either a Mayhew 1,000 or a Midway 13 mud-rotary drill rig with 4-7/8-inch diameter tricone drill bits producing nominal 5- to 6-inch diameter boreholes.

Borings were drilled to a depth of about 30 to 60 feet below the station invert and 20 feet or more below the tunnel invert. In this report, Borings DD-27 and DD-27S (Table 3-1) are considered as one boring.

TABLE 3-1 SUMMARY OF FIELD EXPLORATION PROGRAM

(First/Lorena Station & Tail Track Tunnels)

Boring #*	Type of Boring	Approximate Station Along Centerline of CR Track	Approximate Offset From Centerline of CR (ft)	Approximate Offset From Centerline of CL (ft)	Location	Approximate Ground Surface Elevation (ft)	Approximate Tunnel/ Station Invert (Depth/ Elevation, ft)	Total Penetration Depth (ft)	Monitoring Well Installed
SD-15	Rotary Wash	187+50	5 left	-	First Street, 105' E. of Concord	312	76/236	111	Yes
SD-16	Rotary Wash	190+40	-	28 right	First Street/ 127' W. of Lorena	298	66/232	130	No
SD-17	Rotary Wash	193+56	5 left	-	First Street, 130' E. of Lorena	294	62/232	106	No
SD-18	Rotary Wash	195+80	-	28 right	First Street/Lorena	302	70/232	116	No
DD-27**	Rotary Wash	201+53	-	2 left	First Street, 40' E. of Indiana Street	318	83/235	62.5	No
DD-27S**	Rotary Wash	201+50	-	2 left	First St./ Indiana St. (3' W. of DD-27)	319	84/235	125	Yes
DD-28	Rotary Wash	205+70	10 left	-	S. Alma, 328' S of E. First Street	320	85/235	131	No
DD-62	Rotary Wash	198+90	37 right	-	Velasco, 37.4' S. of First Street	312	77/235	100.5	No
PE-13	Rotary Wash	192+90	3 left	-	First Street, 63' E. of S. Lorena	292	60/232	90.5	Yes

NOTE:

* Borings along tunnel alignment are identified by the prefix DD-; borings at station locations are identified by the prefix SD-; one boring drilled during the preliminary investigation is identified by the prefix PE-.

** DD-27 was terminated prior to reaching the planned penetration depth (125 feet) due to loss of steel ring from pressuremeter probe in the borehole. DD-27 was then replaced by DD-27S and drilled to planned penetration depth.

Boring DD-27 was terminated at a depth of 62½ feet BGS due to loss of a component (steel ring) of the pressure-meter test equipment downhole. Boring DD-27S was drilled adjacent to DD-27 as a replacement hole to the scheduled penetration depth of about 125 feet BGS. Soil samples were obtained by alternately using a split-spoon sampler (Standard Penetration Test Method) and a California drive sampler lined with 2.4-inch diameter by 1-inch high brass rings. In the station area soil samples were obtained at approximately 5-foot depth intervals and at changes in stratigraphy, whichever occurred first. Along the tunnel alignments, a similar sampling schedule was used outside of the tunnel zone while an approximate 3-foot sampling interval was employed within the tunnel envelope. The tunnel zone is defined as the depth interval between 20 feet above the planned tunnel crown and 20 feet below the planned tunnel invert.

Pressuremeter tests were performed at various depth intervals in Boring DD-27/DD-27S. Nearly continuous soil samples were obtained within depth intervals between pressuremeter tests in the borings. After performing a pressuremeter test at a depth interval between 56 and 60 feet BGS in Boring DD-27, a steel ring (part of the pressuremeter test equipment) was inadvertently dislodged in the borehole. Boring DD-27 was abandoned after repeated attempts failed to retrieve the steel ring. A new borehole (DD-27S) located about 3 feet west of Boring DD-27 was then drilled to continue the scheduled soil sampling and pressuremeter testing to a depth of about 125 feet BGS. With the exception of one pitcher tube sample taken at a depth of 37 feet, there was no sampling within the top 60 feet of Boring DD-27S.

3.3 MONITORING WELL INSTALLATION AND GROUNDWATER LEVEL MONITORING

Two monitoring wells were installed in Borings SD-15 and DD-27S. Well installation diagrams are presented in Appendix A. Groundwater levels of these wells and the existing well (PE-13) installed during the preliminary engineering program (GeoTransit Consultants, 1994a) were periodically monitored using an electronic water-level indicator. Table 3-2 presents a summary of these readings.

TABLE 3-2 SUMMARY OF PROJECT-SPECIFIC GROUNDWATER LEVEL DATA

(First/Lorena Station & Tail Track Tunnels)

MONITORING WELL NO.	APPROXIMATE STATION ALONG CENTER LINE OF CR TRACK	APPOXIMATE GROUND SURFACE ELEVATION (FT)	APPROXIMATE DEPTH OF TUNNEL/ STATION INVERT (FT)	MEASURED GROUNDWATER LEVEL								
				DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)	DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)	DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)
SD-15	187+50	312	76	02/14/96	78.8	233.2	8/26/95	80.0	232.0	-	-	-
DD-27S	201+53	319	84	-	-	-	9/15/95	97.3	221.7	-	-	-
PE-13	192+90	292	60	02/14/96	Dry*	-	11/21/95	Dry*	-	11/17/93	Dry*	-

NOTE:

* Because the bottom of well screen of monitoring well PE-13 is about 65 feet BGS, "dry" indicates that no groundwater encountered at this depth.

3.4 FIELD TESTS

3.4.1 Pressuremeter Testing

A series of 12 pressuremeter tests were performed in Boring DD-27/27S as stated in the previous section, to obtain soil modulus characteristics in the proposed tail track tunnel area. Pressuremeter test intervals are noted in logs of Boring DD-27/27S in Appendix A. Table 3-3 shows a summary of the pressuremeter tests and the results.

The pressuremeter tests were conducted by Hughes In Situ Engineering Inc. in accordance with the American Society for Testing and Materials (ASTM) Standard Procedure D4719. The pressuremeter tests were performed using a 2.92-inch diameter Cambridge-type pressuremeter. Pressure within the membrane was measured using a pressure transducer, and radial displacements were measured using four strain sensors located within the probe. These sensors are placed inside the membrane. In each test, the boring was advanced to within about 2.25 feet of the test location using a 4-7/8-inch diameter bit. An approximately 4 feet deep pilot hole was then drilled using a 2-15/16-inch diameter tricone bit or a 2-3/4-inch diameter drag bit. The pressuremeter probe was lowered into the pilot hole and was expanded by manually pumping hydraulic oil from a hand operated hydraulic pump. The pressure in the pressuremeter was increased in discrete steps to the desired pressure, and held constant for about 60 seconds while the corresponding radial displacements were recorded. The volumetric strain at any given pressure was computed as twice the radial strain at that pressure. In order to account for the stiffness of the pressuremeter membranes, membrane corrections were applied to the measured strains and pressure.

The tests were terminated when (a) one of the four displacement sensors reached 18 percent strain or (b) there was no further increase in strain as oil continued to be pumped into the probe. Further, in each test, at least one unload/reload cycle was conducted at the end of a 60-second loading period.

Pressure-radial strain curves for all the tests including the unload/reload cycles are presented in Appendix A-1. Also included in these figures is the plot of the creep strains which occurred while the pressure was held constant for 60 seconds.

TABLE 3-3 RESULTS OF PRESSUREMETER TESTS AT BORINGS DD-27 / 27S
(First/Lorena Station & Tail Track Tunnels)

Boring #	Depth Range (ft)	Geological Stratigraphy +	Measured Pressuremeter Shear Modulus (10^3 ksf)	Calculated Menard Modulus* (10^3 ksf)	Measured Pressuremeter Cyclic (unload/reload) Shear Modulus (10^3 ksf)	Calculated Cyclic (unload/reload) Young's Modulus** (10^3 ksf)
DD-27	23.0-28.0	Sand/Silty Sand	***	***	***	***
	34.5-40.5	Lean Clay	***	***	>3.74	>10.47
	43.0-47.0	Silty Sand	1.20	3.11	10.4	27.04
	49.5-53.5	Silty Sand	0.75	1.94	7.92	20.59
	56.0-60.0	Silty Sand	0.83?	2.15?	17.64?	45.86?
DD-27S	62.5-66.5	Sandy Silt	0.55	1.54	4.72	13.22
	69.0-73.0	Sandy Silt	0.61	1.71	6.05	16.94
	75.5-79.5	Silty Sand	0.92	2.39	7.01	18.23
	82.0-86.0	Silty Sand	0.52	1.36	4.18	10.87
	88.5-92.5	Silty Sand	0.32	0.82	4.54	11.80
	95.5-99.5	Silty Sand	0.51	1.33	4.54	11.80
	108.5-112.5	Sandy Lean Clay	0.69	1.94	4.70	13.16

NOTES:

+ Geological Stratigraphy description is defined in Appendix A, Figure A-1.

* Calculated from pressuremeter tests by using a Poisson's ratio of 0.3 for coarse-grained alluvium and 0.4 for fine-grained alluvium.

** Calculated from cyclic unload/reload pressuremeter shear modulus by using a Poisson's ratio of 0.3 for coarse-grained alluvium and 0.4 for fine-grained alluvium.

*** No useful data obtained.

? Questionable data due to dislodging of steel ring from pressuremeter probe during test.

The pressure-radial strain curves have been analyzed to provide information on the pressuremeter modulus ("Menard" modulus) and corresponding shear modulus, the cyclic (unload/reload) pressuremeter shear modulus, the limit pressure (P_ℓ) and creep pressure (P_c). Definitions of these terms are as follows:

Pressuremeter Shear Modulus Pressuremeter shear modulus is defined as one-half of the slope of the linear portion of the pressure-radial strain curve. Pressuremeter shear modulus multiplied by the factor $2 (1 + \mu)$ is the pressuremeter modulus E_m (Menard modulus - akin to Young's modulus), where μ is the Poisson's ratio. The value of μ for granular alluvium and fine-grained alluvium were assumed to be 0.3 and 0.4, respectively.

Cyclic (Unload/Reload) Pressuremeter Shear Modulus Cyclic (unload-reload) pressuremeter shear modulus is defined as one half of the slope of the initial portion of the unload/reload cycle of the pressure-radial strain curves. Unload/reload shear modulus multiplied by a factor $2 (1 + \mu)$ is unload/reload pressuremeter modulus E_u. Since there is more than one such cycle for each test, an average value of E_u is reported in Table 3-3.

Limit Pressure (P_ℓ) Limit pressure is defined as the pressure at which the probe has expanded to twice its original volume. The value of P_ℓ is determined from pressure-radial strain curves in a semi-log plot.

Creep Pressure (P_c) Creep pressure is estimated from the pressure-creep strain curve, and is defined as the pressure at which the creep strains increase substantially.

3.4.2 Downhole Seismic Velocity Survey

Two downhole seismic velocity surveys were performed in PVC-cased Borings SD-15 and DD-27S. Procedures used for seismic velocity measurements were in general accordance with those described by U.S. Army Corps of Engineers (1979) and Mooney (1984). In general, downhole compressional (P) and shear (S) wave velocities were measured at approximately 5-foot intervals, using three-mutually perpendicular geophones (one vertical and two horizontal) mounted in a 1.75 inch steel cylinder in the borehole. P and S wave sources were generated by hitting a vertical hammer against a metal plate and a horizontal hammer on a wooden beam, respectively. Both metal plate and wooden beam were located on the ground surface at about 10 to 12 feet away from the borehole. The results of the seismic velocity surveys are summarized in Table 3-4.

3.5 LABORATORY TESTING PROGRAM

All drive, split spoon, Pitcher barrel, and bulk samples obtained during the subsurface exploration were brought to EARTH TECH's soil mechanics laboratory where they were visually examined to verify field classification. Samples of the various material types encountered were selected for laboratory testing. The laboratory test program was designed to classify the predominant soil types encountered in the borings and to evaluate the in situ moisture and density, gradation, shear strength, uniaxial compressive strength, permeability and consolidation characteristics, and corrosion potential. The tests were performed in accordance with applicable standard test methods specified by ASTM, the Environmental Protection Agency (EPA), and/or the California Department of Transportation (Caltrans).

The test program and applicable test standards are summarized in Table 3-5. Laboratory test results are summarized in Table 3-6 and included in Appendix B. In situ density and moisture content are also shown on the boring logs included in Appendix A. A discussion of the engineering properties of subsurface materials is presented in Section 4.4.

3.6 FIELD OBSERVATIONS

Various field observations were noted in this investigation during drilling, sampling and development of monitoring wells. These observations include the following:

- Locations and approximate sizes of cobbles
- Headspace organic vapor analyzer (OVA) readings of soil samples and the corresponding background readings.
- Sulfurous and/or hydrocarbon odors.

TABLE 3-4 RESULTS OF DOWNHOLE SEISMIC VELOCITY SURVEYS AT TWO BORING LOCATIONS
(First/Lorena Station & Tail Track Tunnels)

Boring No.	Depth Range (ft)	Geological Straigraphy*	Measured Compressional Wave Velocity (ft/sec)	Measured Shear Wave Velocity (ft/sec)	Calculated Dynamic Modulus and Poisson's Ratio			
					Shear Modulus 10^3ksf	Young's Modulus 10^3ksf	Bulk Modulus 10^3ksf	Poisson's Ratio
SD-15	0-17	Silty Sand	1750	940	3.6	9.3	7.6	0.30
	17-40	Sandy Lean Clay/Sandy Silt/ Silty Sand/Gravel	3050	1560	9.8	26.0	24.5	0.32
	40-110	Sand/Gravel/Silty Sand	5150	1990	16.0	45.2	85.8	0.41
DD-27S	0-19	Sandy Lean Clay/ Silty Sand with Gravel	2250	1040	4.4	11.9	14.6	0.36
	19-72	Silty Sand/Sandy Lean Clay/Sandy Silt	2750	1480	8.8	22.9	18.7	0.30
	72-97	Sandy Silt/ Silty Sand with Gravel/Silty Sand	3600	1920	14.9	38.7	32.5	0.30

Note: * Geological Stratigraphy description is defined in Appendix A, Figure A-1.

TABLE 3-5 GEOTECHNICAL LABORATORY TEST PROGRAM

(First/Lorena Station & Tail Track Tunnels)

TEST TYPE	NUMBER OF TESTS	TEST PROCEDURE
Visual Soil Classification	Every Sample	ASTM D2487 / D2488
Moisture Content	95	ASTM D 2216
Dry Density	94	ASTM D 2937
Grain Size Distribution	56	ASTM D 422
Percent Passing #200 Sieve	41	ASTM D 1140
Atterberg Limits	17	ASTM D 4318
Specific Gravity	2	ASTM D 854
Direct Shear (3 Points)	9	ASTM D 3080
Unconfined Compression	1	ASTM D 2166
Triaxial Compression	-	ASTM D 4767
One Dimensional Consolidation / Collapse Potential	7	ASTM D 2435 / ASTM D 5333
Triaxial Permeability	2	ASTM D 5084
Slake Durability Test	-	ASTM D 4464
pH	6	EPA Method 9045
Chloride Content	6	CALTRANS Test 422
Sulphate Content	6	CALTRANS Test 417-B
Electrical Resistivity	6	CALTRANS Test 532

TABLE 3-6 SUMMARY OF LABORATORY TEST RESULTS
(First/Lorena Station & Tail Track Tunnels)

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength)		Triaxial Compression ASTM D4767 (Peak Strength)		Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D5333					Triaxial Permeability ASTM D5084		Soil pH USEPA Method 9045	Sulphate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	Insitu <Minimum> Electrical Resistivity @ Insitu <Adjusted> Moisture Content DOT CA Test 632 (ohm-cm)
													Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Cc'	Cr'	Cα'	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Effective Confining Stress (psf)	Permeability (cm/sec)				
A) THIS INVESTIGATION																											
SD-15	S-1	5	SM	Qa	22			0:53:47																			
	D-2	10	SM		(10)	23.4	95.9	-		45																	
	S-3	15	SM		15			0:65:35																			
	D-4	20	CL		(14)	19.9	106.8	-	48,22,26				1400	27													
	S-5	25	ML		26			1:46:53																			
	D-6	30	SM		(31)	13.4	118.1																				
	S-7	35	SM		>100			18:69:13																			
	D-8	40	SP/GP		(61)	11.0	133.1																				
	S-9	45	(SP-SM)		>100					12																	
	D-10	50	SP-SM		(27)	12.2	101.2			7			400	37													
	S-11	55	(SM)		>100			25:62:13																			
	D-12	60	SP		>(100)	11.5																					
	S-13	65	(SM)		>100	10.4				13														8.50	95	110	862
	D-14	70	SP		(92)	13.0	109.8															30	4.0E-04				
	S-15	75	(SW-SM)		>100			20:68:12																			
	D-16	80	SP		(76)	18.8	90.0																				
	S-17	85	CL		51				32,19,13																		
	D-18	90	SM		(26)	19.5	109.3			37							0.063	0.006	0.0011	1	-0.1						
	S-19	95	SM		>100			1:76:23																			
	D-20	100	CH		(36)	19.2	108.6																				
	S-21	105	CH		44				63,23,40																		
	D-22	110	CL		(30)	18.2	111.8			59																	
SD-16	D-1	10	CL	Qa	(19)	14.8	112.9			52																	
	S-2	15	SC		50			2:60:38																			
	D-3	19	(SP-SM)		(35)	13.8	118.0			10																	
	S-4	25	(SM)		81			16:68:16																			
	D-5	30	SP/GP		(67)	13.8	119.8																				
	S-6	35	SC		43			3:51:46																			
	D-7	39.0	SM		(40)	16.2	107.8			19			0	45													
	S-8	45	SW-SM		>100			18:71:11																			
	D-9	50	SW-SM		>(100)	8.7	128.5																				
	S-10	55	SW-SM		>100			31:57:12																			
	D-11	60	SW-SM		>(100)	13.8	109.6			10																	
	S-12	65	(SM)		>100			1:81:18																			
	D-13	68.5	CH		(26)	23.6	102.3		53,21,32	82							0.082	0.02	0.0006	8	+0.3			7.44	121	107	1531
	D-14	72	CH		(22)																						
	S-15	75	SM		>100			1:76:23																			
	D-16	80	SP-SM		(63)	22.5	105.2			8																	
	S-17	85	CL		64					85																	
	D-18	89	CL		(26)	20.0	108.2		40,15,25								0.075	0.020	0.0006	10	+0.26						
	S-19	94	CL		47					60																	
	D-20	100	SC		(23)	18.9	110.1																				
	S-21	105	SC		92			0:57:43																			
	D-22	110	SC		(32)	17.4	113.4																				
	D-23	113	CL		(31)	15.8	118.2		28,14,14																		

TABLE 3-6 SUMMARY OF LABORATORY TEST RESULTS
(First/Lorena Station & Tail Track Tunnels)

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength)		Triaxial Compression ASTM D4767 (Peak Strength)		Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D5333					Triaxial Permeability ASTM D5084		Soil pH USEPA Method 9045	Sulphate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	Insitu <Minimum> Electrical Resistivity @ Insitu <Adjusted> Moisture Content DOT CA Test 532 (ohm-cm)
													Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Cc	Cr	Cv	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Effective Confining Stress (psf)	Permeability (cm/sec)				
SD-16	T-24	115	CL	Qa	-																						
	S-25	120	SM		64																						
	D-26	123	CL	Tf/Tp	(31)	17.8	110.6						3000	25													
	T-27	124	CL			17.7	113.5		34,15,19			12.224															
SD-17	D-1	5	GM	Qa	(4)	13.1	117.0	-																			
	S-2	10	GM		9			52:35:13																			
	D-3	15	CL		(3)	18.9	106.9																				
	S-4	20	CL		4			9:39:52																			
	D-5	25	SP-SM		(21)	8.7	125.0			9																	
	S-6	30	SM		85			11:71:18																			
	D-7	35	SM		(58)	12.7	113.3																				
	S-8	40	ML		59	28.9		0:35:65																7.85	76	57	2329
	D-9	45	SM/ML		(17)	16.8	103.1			50			600	31													
	S-10	50	SM/ML		35																						
	D-11	55	SP-SM		(67)	10.4	113.1			7																	
	S-12	60	ML		52			5:38:57																			
	D-13	65	SP/GP		(76)	11.4	131.1																				
	S-14	70	SW-SM		>100			17:71:12																			
	D-15	75	SW-SM		(43)	19.0	106.2				2.69																
	S-16	80	ML		>100			0:45:55																			
	D-17	85	CL		(26)	18.8	110.3		44,13,31				2000	25			0.08	0.018	0.001	1	+0.31						
	S-18	90	CL		82																						
	D-19	95	SM		(46)	22.6	103.1			34																	
	S-20	100	CL	Tf/Tp	50																						
	D-21	105	CL		(33)	20.3	107.4		43,18,25								0.05	0.016	0.0004	10	+0.88						
SD-18	D-1	8	CL	Qa	(6)	17.5	114.4																				
	S-2	10	CL		7																						
	D-3	15	CL		(14)	19.1	110.5		47,24,23																		
	S-4	20	SM		21			1:57:42																			
	D-5	25	SM		(43)	15.0	113.3																				
	S-6	30	SM		73					15																	
	D-7	35	SM		(34)	9.4	111.4																				
	S-8	40	SM		>100			27:52:21																			
	D-9	45	GP-GM		(52)	9.8	129.5			6																	
	S-10	50	(ML)		65			4:41:55																6.76	138	907	<2941>
	D-11	55	SM		(63)	10.8	117.5			21																	
	S-12	60	SM		>100			14:63:23																			
	D-13	62	SM		>(100)	11.3	124.8																				
	S-14	65	SM		100					26																	
	D-15	70	SM		(40)	22.2	100.1																				
	S-16	75	ML		50			0:29:71																			
	D-17	80	CL		(28)	17.9	114.4		31,13,18	51			920	29													
	S-18	82	CL		>100																						
	D-19	85	SP/SM		(64)	17.9	113.7																				
	S-20	90	SM		>100			8:77:15																			
	D-21	94	CL		(35)	18.5	112.8		33,18,15								0.076	0.012	0.0007	1	+0.52						

TABLE 3-6 SUMMARY OF LABORATORY TEST RESULTS
(First/Lorena Station & Tail Track Tunnels)

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength)		Triaxial Compression ASTM D4767 (Peak Strength)		Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D5333					Triaxial Permeability ASTM D5084		Soil pH USEPA Method 9045	Sulphate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	Insitu <Minimum> Electrical Resistivity @ Insitu <Adjusted> Moisture Content DOT CA Test 532 (ohm-cm)
													Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Cc'	Cr'	Cα'	Vertical Stress (ks)	Swell(+)/ Collapse(-) (%)	Effective Confining Stress (psf)	Permeability (cm/sec)				
SD-18	S-22	99	CL	Qa	57																						
	D-23	104	CL		(28)	23.6	105.3																				
	S-24	110	SP/SM		51																						
	D-25	114	ML/SM		(20)																						
	D-26	115	ML/SM		(21)																						
DD-27	D-1	10	SM	Qa	(17)	10.1	108.6			20																	
	S-2	15	SM		33																						
	D-3	20	SP/SM		(43)	8.9	114.4																				
	S-4	21	SM		50			14:73:13																			
	D-5	22.5	SP/SM		(31)																						
	D-6	28	SP/SM		(15)	18.7	108.5																				
	S-7	29	SP/SM		31																						
	D-8	34	CL		(17)	12.1	113.6																				
	S-9	35	CL		27			1:19:80																			
	S-10	40.5	CL		21																						
	D-11	42	CL		(19)	20.2	107.3		40,19,21	66																	
	S-12	47	CL		45			1:47:52																			
	D-13	48.5	CL		(25)	14.3	113.9																				
	S-14	53.5	SM		77			9:50:41																			
	D-15	55	SM		(23)	11.6	112.6																				
	S-16	60	SM/ML		88																						
DD-27S	T-16	37	CL	Qa																							
	S-17	60	ML		92			0:44:56																			
	D-18	61.5	ML		(37)	19.5	97.0																				
	S-19	66.5	SM		>100			1:70:29																			
	D-20	68	ML		(56)	12.3	109.8																				
	S-21	73	ML		99			0:35:65																			
	D-22	74.5	SM		(41)	11.0	115.7				2.63													6.97	46	90	<2128>
	S-23	80	SM		>100					14																	
	D-24	81.3	SM		(50)	11.7	116.1			27			1200	32													
	S-25	86	SM		>100			16:62:22																			
	D-26	87	SM		(37)	9.5	116.9																				
	S-27	92.5	SM		>100			0:79:21																			
	D-28	94	SM		(45)	11.0	113.5																				
	D-29	99.5	SM		(27)	24.7	99.7																				
	S-30	100.5	ML		65			0:49:51																			
	D-31	102	ML		(30)	21.5	105.0																				
	D-32	107.5	CL		(27)	21.9	104.6																				
	S-33	112.5	(SC)		>100			5:61:34																			
	D-34	113.5	SP/SM		34	21.6	104.0																				
	S-35	119	CL		75																						
	D-36	124	CL		(30)	15.5	117.4	0:35:65																			
DD-28	D-1	10	SC	Qa	(18)*	13.1	117.9																				
	S-2	14.5	SM		48																						
	D-3	20	CL		(15)	16.4	102.6			60																	
	S-4	25	CL		25																						

TABLE 3-6 SUMMARY OF LABORATORY TEST RESULTS
(First/Lorena Station & Tail Track Tunnels)

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength)		Triaxial Compression ASTM D4767 (Peak Strength)		Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D3333					Triaxial Permeability ASTM D5084		Soil pH USEPA Method 9045	Sulphate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	Insitu <Minimum> Electrical Resistivity @ Insitu <Adjusted> Moisture Content DOT CA Test 532 (ohm-cm)
													Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Cc'	Cr'	Cα'	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Effective Confining Stress (psi)	Permeability (cm/sec)				
DD-28	D-5	30	SM	Qa	(16)	14.2	100.9			40																	
	S-6	35	SP/SM		38																						
	D-7	40	CL		(22)	15.6	114.6			56																	
	S-8	44	CL		32			0:35:65																			
	D-9	50	CL		(17)	26.1	96.6			96																	
	S-10	53.5	SM		94			5:80:15																			
	D-11	60	(SP-SM)		(43)	8.0	125.3			9																	
	S-12	63.5	CL		52			1:36:63																			
	D-13	67	SC		(35)	19.6	106.5																				
	S-14	70	(CL)		76			0:49:51																			
	D-15	73	ML		(26)	32.8	86.8		38,30,8													60	5.0E-06				
	S-16	76	ML		62					67																	
	D-17	79	ML		(28)	28.5	89.1																				
	S-18	82	SM		>100			0:59:41																			
	D-19	85	SP		(49)	7.6	102.7			3			300	41													
	S-20	88	ML		76			0:49:51																7.42	74	60	<2776>
	D-21	90.5	SM		(48)	12.7	104.9			29																	
	S-22	94	ML		67			0:27:73																			
	D-23	97	ML		(43)	19.2	107.9			63																	
	S-24	100	ML		74			0:32:68																			
	D-25	105	ML		(33)	23.6	103.8			54																	
	S-26	107	ML		79			0:39:61																			
	D-27	110	SM		(45)	22.1	101.9																				
	S-28	115	SM		>100			0:76:24																			
	D-29	120	CL		(37)	13.8	120.9																				
	S-30	125	CL		>100																						
	D-31	130	SC/SM		(55)	22.0	107.9																				
DD-62	D-2	10	SC	Qa	(6)	23.5	96.8		30,20,10																		
	S-3	15	SC		31																						
	D-4	20	SP-SM		(17)	15.1	106.5													2.5	-0.17						
	S-5	25	CL		23																						
	D-6	30	CL		(21)	18.3	109.8																				
	S-7	35	CL		26																						
	D-8	38	SC-SM		(22)	13.1	117.3																				
	S-9	41	SC-SM		45				21,14,7	44-47																	
	D-10	44	CL-ML		(23)	19.2	108.2		25,20,5																		
	S-11	47	SM		>100			6:68:26																			
	D-12	50	SP		(40)	13.3	111.7																				
	S-13	53	(SP-SM)		>100			27:62:11																			
	D-14	56	(SP-SM)		>(100)	12.7	119.1																				
	S-15	59	(SP-SM)		>100			45:47:8																			
	D-16	62	(SP-SM)		>(100)	14.9	122.5																				
	S-17	65	(SM)		>100			20:61:19																			
	D-18	68	(SP-SM/GM)		>(100)																						
	S-19	71	ML		100			0:20:80																			
	D-20	74	SM		(34)	18.6	102.5																				

TABLE 3-6 SUMMARY OF LABORATORY TEST RESULTS
(First/Lorena Station & Tail Track Tunnels)

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength)		Triaxial Compression ASTM D4767 (Peak Strength)		Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D 333					Triaxial Permeability ASTM D5084		Soil pH USEPA Method 9045	Sulphate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	Insitu <Minimum> Electrical Resistivity @ Insitu <Adjusted> Moisture Content DOT CA Test 532 (ohm-cm)
													Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Cc'	Cs'	Cx'	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Effective Confining Stress (psf)	Permeability (cm/sec)				
OD-62	S-21	77	ML	Qa	>100			0:38:62																			
	D-22	80	SM		(49)	11.1	106.7			15																	
	S-23	83	SM		>100					33																	
	D-24	86	SP-SM		>(100)	12.7	115.3																				
	S-25	89	SP-SM		>100																						
	D-26	92	SP-SM		>(100)																						
	S-27	95	SP-SM		>100																						
	D-28	100	SP-SM		>(100)																						
B) PREVIOUS PRELIMINARY INVESTIGATION																											
PE-13	S-1	5	CL	Qa	6																			7.80	54	95	1923
	D-2	10	CL		(8)	26.6																					
	S-3	15	CL		5			0:33:67	24,14,10																		
	D-4	20	SC		(9)	15.5			25,14,11	38																	
	S-5	25	SM/ML		24																						
	D-6	30	SM		(55)	16.4	114.3			13																	
	S-7	35	SM		95			3:83:14																			
	D-8	40.2	SM		(46)	19.2	94.4																				
	S-9	46	(SM)		100			0:72:28																			
	D-10	55.5	(SP-SM)		>(100)	12.6	119.0	24:69:7																8.68	71	92	3378
	S-11	65	GP-GM/SP-SM		>100																						
	D-12	75	(SP-SM)		(98)	21.2																					
	S-13	80	CL		52																						
	D-14	85	ML/CL		60	21.8	105.1																				

NOTES:

1. USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).
2. Some of the material classifications (identified in parentheses) shown in column 4 of this table are not consistent with the general classification of that interval in the boring logs. This generally occurs in gravelly and cobbly layers where due to presence of gravels and cobbles larger than sampler size, laboratory gradations and classifications only reflect the finer matrix materials in alluvium.
3. For the same reasons presented above, results of gradation, fines content, in situ moisture content and in situ dry density tests in layers identified as gravels, silty/clayey gravel, gravel with sand and sand with gravel, are only representative of the finer matrix materials in the alluvium.
4. Geologic units Af, Qa and Tf/Tp denote artificial fill, alluvium and Fernando/Puente Formation, respectively.
5. For California Drive Samples, Equivalent SPT values were obtained by applying the appropriate corrections for different hammer weights, hammer drop and sampler dimensions (see Appendix A, Figure A-2), Equivalent SPT values corrected from drive sampler blowcounts are shown in parentheses.
6. Compression Index (Cc') and Swelling Index (Cs') are based on vertical strain-log stress plots, Rate of Secondary Compression (Cx') is based on vertical strain-log time plot.
7. Swell/Collapse indicates measured strains in sample when saturated from in situ moisture conditions at approximately the in situ overburden pressure.
8. Explanation of symbols:
   GR:SA:FI = Gravel: Sand : Fines (percent passing #200 sieve)
   LL,PL,PI = Liquid Limit, Plastic Limit, Plasticity Index

During field investigation activities (drilling, sampling, pressuremeter testing and well development), no noticeable sulfurous and/or hydrocarbon odors were present. The field activities did not encounter adverse drilling conditions such as caving or loss of circulation of drill mud. Within all borings drilled and sampled in the project area, only two samples (one from Boring SD-15 at a depth of 95 feet, and one from Boring SD-17 at a depth of 105 feet) were noted to have volatile organic vapor (OVA) concentrations of 10 ppm or more above the corresponding background readings. Borings SD-15, SD-16, SD-17 and PE-13 in the station area encountered drill chatter/slow drilling at a number of depth intervals (refer to boring logs in Appendix A). Taking into account the size and type of drill bit employed on this project 4⅞ inch diameter tricone bit which is capable of drilling through cobbles and boulders, and based on drilling experience with the particular drill rigs and drill bits used in this investigation, the occurrence of rig chatter is considered to be indicative of the presence of dense cobbly soils with clast sizes on the order of three inches or more. Thus, localized cobbly zones may be encountered during station excavation.

4.0 GEOLOGIC AND GROUNDWATER CONDITIONS

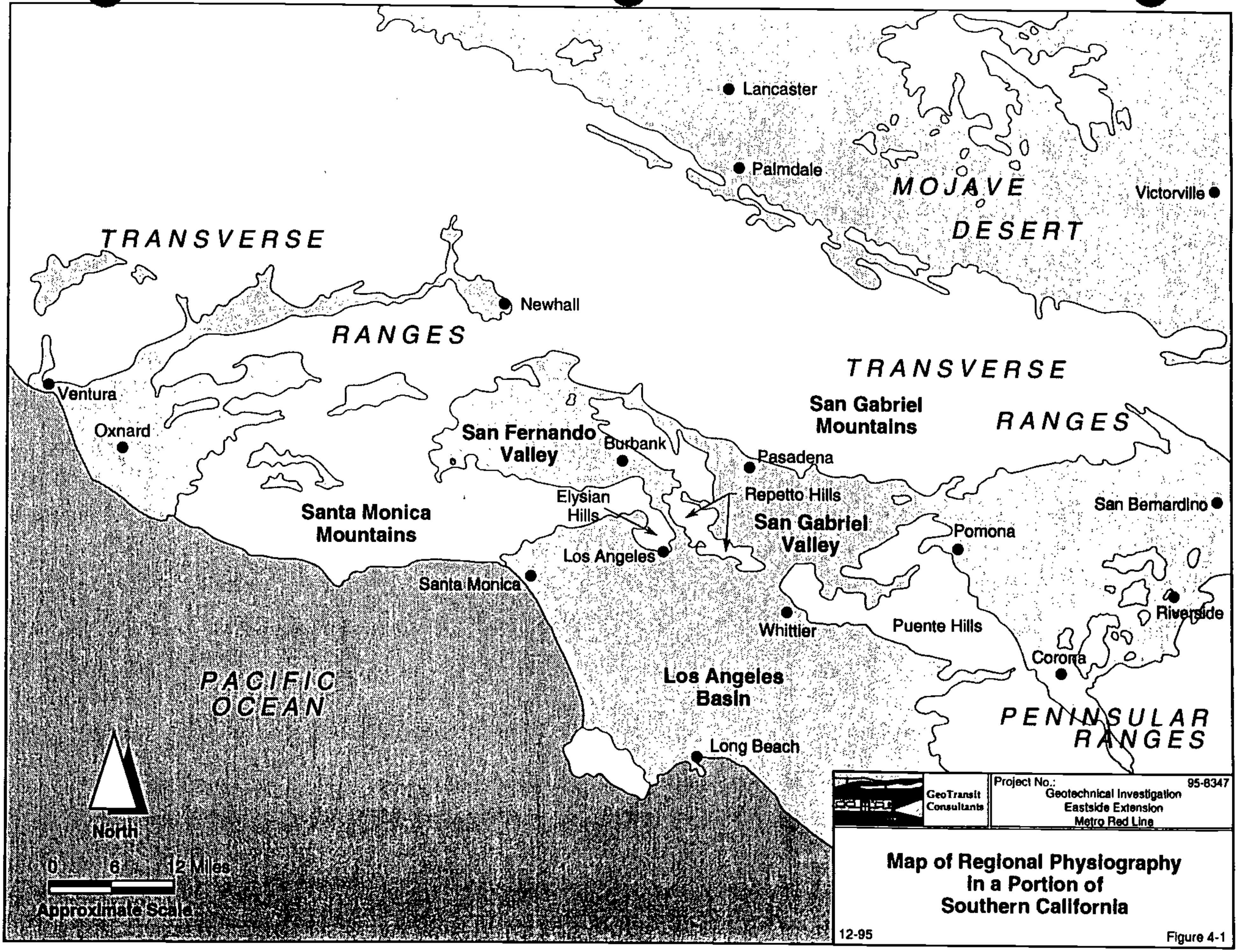
4.1 REGIONAL SETTING

4.1.1 Regional Geology

The proposed Metro Rail Eastside Extension alignment is on the northern edge of the Los Angeles coastal plain and the underlying structural basin, at the junction between the Transverse Ranges and Peninsular Ranges geomorphic provinces in Southern California (Figure 4-1). The Elysian and Repetto Hills in central and eastern Los Angeles are a northwest extension of the Peninsular Ranges trending northwest from Baja California. The east-west oriented San Gabriel, Verdugo, and Santa Monica Mountains to the north of the hills are in the western part of the Transverse Ranges, which extend across Southern California from the Colorado Desert to Point Arguello. The Peninsular Ranges are largely defined by right-lateral strike-slip faulting and associated folding parallel to their trend, and the western Transverse Ranges are uplifted by northward-dipping thrust faults along their southern margin. The hilly terrain of the Eastside Extension area appears to result from folding and faulting in a zone of convergence between these major sets of structures.

Bedrock units of the mountainous areas consist of a wide variety of Precambrian to Mesozoic igneous and metamorphic basement rocks, and a partial cover of Mesozoic to early Tertiary sedimentary and volcanic strata. Tertiary marine sediments and lesser volcanic rocks that were deposited in the developing Los Angeles basin during Miocene and Pliocene time compose much of the folded and faulted, northwest-trending hills of the present coastal plain. The oldest strata exposed in the southern and western Repetto Hills near the proposed alignment are those of the Puente Formation, which consist primarily of siltstone, claystone, and sandstone. Puente Formation strata are conformably overlain by deposits of the Pliocene-age Fernando Formation, which generally grade upward from siltstone near the base to conglomerate near the top. This unit apparently records the final episode of marine deposition in the Los Angeles Basin, before the coastal plain was elevated above sea level.

Deformation of Miocene and Pliocene marine deposits in the Repetto Hills has been accompanied during Pleistocene time by deposition of alluvium from the Transverse Ranges to the north. Cycles of alluvial



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Map of Regional Physiography in a Portion of Southern California

12-95

Figure 4-1

deposition, continued deformation, and partial erosion have left a fringe of uplifted and dissected alluvial fans and terraces on the flanks of the hills.

There is current debate among geologists about the geologic structure and ongoing tectonic activity in the Repetto and Elysian Hills. Speculation in the wake of the 1987 Whittier Narrows earthquake suggests that a northeast- to north-dipping extension of faulting beneath the northwest trend of the Whittier fault has produced thrust-fault offsets of well-consolidated bedrock at depth that are expressed in the weaker near-surface materials by folding, faulting, or a combination of the two comprising the Repetto and Elysian Hills.

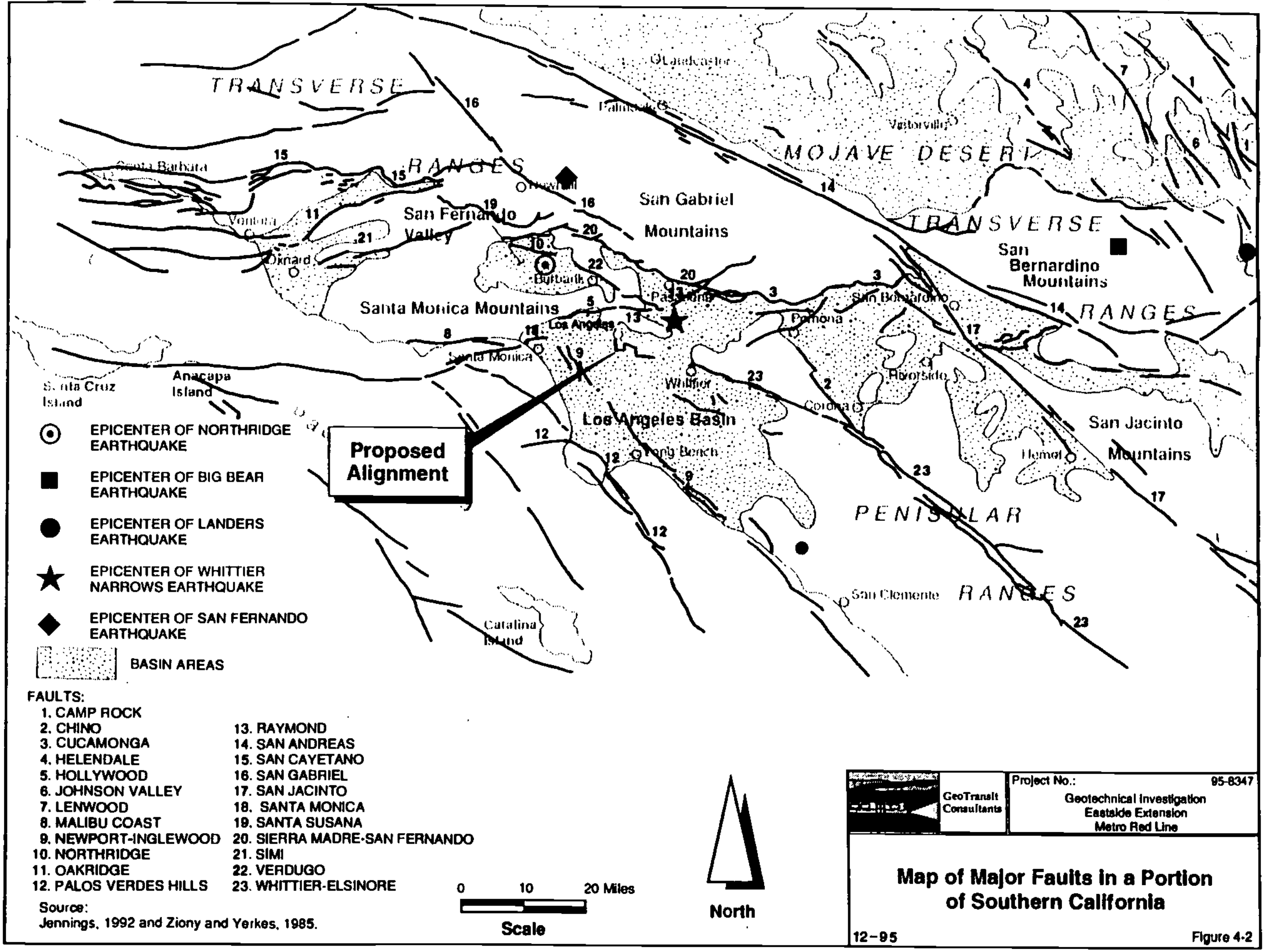
4.1.2 Regional Faulting and Seismicity

4.1.2.1 Regional Faulting

The proposed station is located in a region of high seismic-potential that has experienced ground shaking from numerous large earthquakes in historical time. The earthquakes are being generated by periodic slip along the northwesterly-trending strike-slip San Andreas and Peninsular Ranges fault systems, and on the generally east-west trending thrust faults of the Transverse Ranges.

Figure 4-2 shows the known major active and potentially active faults in the greater Los Angeles area. According to the California Division of Mines and Geology (CDMG), the term "active" applies to any fault that has moved within Holocene time (i.e., the past 11,000 years). Such activity is recognized by displacement of Holocene-age sediments or by direct association with seismic activity. The term "potentially active" applies to a fault that has been active during Pleistocene time (i.e., 2 to 3 million years preceding the Holocene). Such faults may have remained active during Holocene time, but direct geologic evidence for continued activity is not available. The CDMG does not specifically define an inactive fault, although they do indicate that a fault may be presumed to be inactive based on "direct geologic evidence" of inactivity during the past 11,000 years or longer (Hart, 1990).

The documented active faults closest to the alignment are the east-west trending Hollywood, Santa Monica, and Raymond faults. The Hollywood-Santa Monica fault is located at the southern base of the Santa Monica Mountains about 7 miles northwest of the station. The Raymond fault passes through the



Source:
Jennings, 1992 and Ziony and Yerkes, 1985.

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Map of Major Faults in a Portion of Southern California

12-95

Figure 4-2

northern part of the Repetto Hills into the south Pasadena-San Marino area to the east, and is about 5 miles north of the station at its closest point. A fault that is postulated to be the extension of the Whittier fault to the northeast of the Montebello and Monterey Park Hills area is located approximately 5 miles northeast of the station (Treiman, 1991; Bullard and Lettis, 1993). Other active and potentially active faults that are within 30 miles of the station are listed in Table 4-1 together with the San Andreas fault, which has been included in the table for comparative purposes.

In addition to the fault traces that are shown in Figure 4-2, topographic features having tectonic origins have been identified in the vicinity of the station (Plate 1). An east-west-trending linear escarpment in alluvium that crosses the Eastside Extension alignment at three locations (approximate Stations CR 35 + 50, CR 108+00 and CR 154+00) west of the station, probably coincides with the "Coyote Pass fault" as mapped by the California Department of Water Resources (1961). Several investigators have recently interpreted the escarpment to be a tectonic feature related to surface faulting associated with a postulated buried thrust fault system within this part of the Los Angeles basin (Bullard and Lettis, 1993; Sieh, 1993; Dolan and Sieh 1992a and 1992b). Our subsurface evaluation of the escarpment and its geologic significance for the Eastside Extension tunnel alignment are discussed in a separate report by GeoTransit Consultants (1996a).

4.1.2.2 Regional Seismicity

Moderate to large earthquakes can be expected to occur in the site region during the life of the project. In the event that a nearby fault were to slip and produce a major earthquake, very strong ground motions would affect the alignment.

An earthquake computer search (Blake, 1992) was performed to locate historical earthquake epicenters with respect to the station. A search radius of 150 miles from the approximate mid-point of the alignment of the Eastside Extension was selected in order to include the larger magnitude earthquakes that have occurred in Southern California. Catalogued earthquakes since the year 1800 with magnitudes ranging from 4 to 7.9 are shown in Figures 4-3 and 4-4. The largest historical event was the 1857 Fort Tejon earthquake (estimated M 7.9) on the San Andreas fault, about 125 miles northwest of the proposed station. The epicenter of the closest moderate-sized historical earthquake was that of the 1987 Whittier

TABLE 4-1. ESTIMATED SEISMIC CHARACTERISTICS OF PRINCIPAL FAULTS

Fault	Approximate Distance from Station[1] (miles)	Magnitude of Maximum Credible Earthquake[2]	Age of Most Recent Displacement[3]
Chino	27	7 1/2	Late Quaternary
Cucamonga	29	7	Holocene
Hollywood	7	7 1/2	Holocene
Malibu Coast	24	7 1/2	Holocene
Newport-Inglewood	9	7	Historic (1933)
Northridge	23	7 1/2	Late Quaternary; Holocene
Palos Verdes Hills	18	7	Late Quaternary; Holocene
Raymond	5	7 1/2	Holocene
San Andreas	33	8	Historic (1857)
San Gabriel	16	7 1/2	Late Quaternary; Holocene
Santa Monica	12	7 1/2	Late Quaternary; Holocene[4]
San Fernando	18	7 1/2	Historic (1971)
Sierra Madre	12	7 1/2	Late Quaternary; Holocene
Verdugo	10	6 3/4	Late Quaternary; Holocene
Whittier	5	7 1/2	Late Quaternary; Holocene

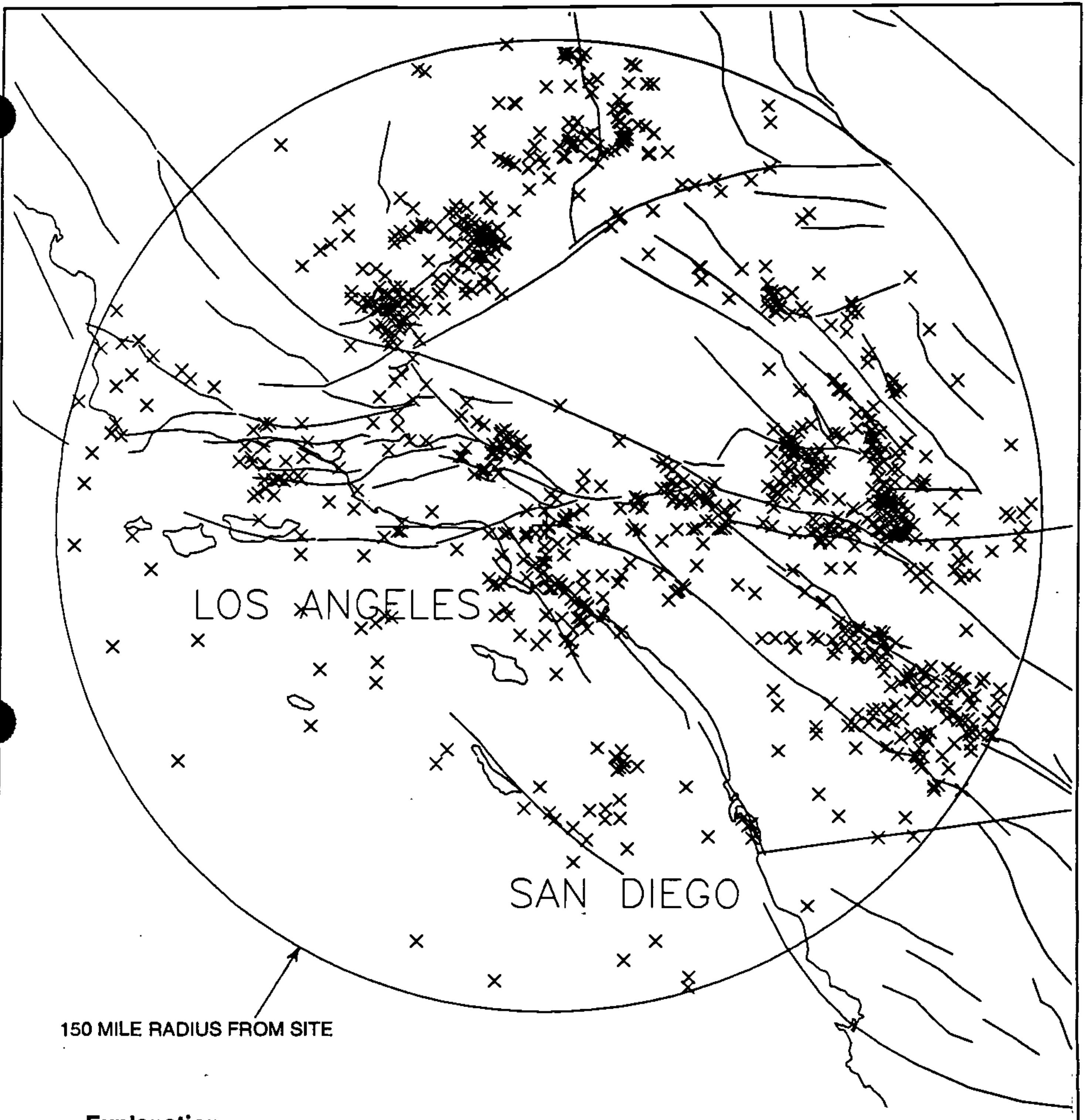
Notes:

(1) Distance measurements are based on fault traces shown in Jennings (1992) and Treiman (1991).

(2) Maximum Credible Earthquake Magnitudes from Mualchin and Jones (1992).

(3) Age of Most Recent Displacement from Jennings (1992) except where noted; multiple ages apply to separate fault segments; "Late Quaternary" is the past 700,000 years; Holocene is the past 11,000 years.

(4) Dolan and Sieh (1992a).



Explanation:

× M = 4.0–4.9

Site Location(+):

Latitude - 34.0340 N
Longitude - 118.1920 W

Source:
Epicenters from Blake, 1992.

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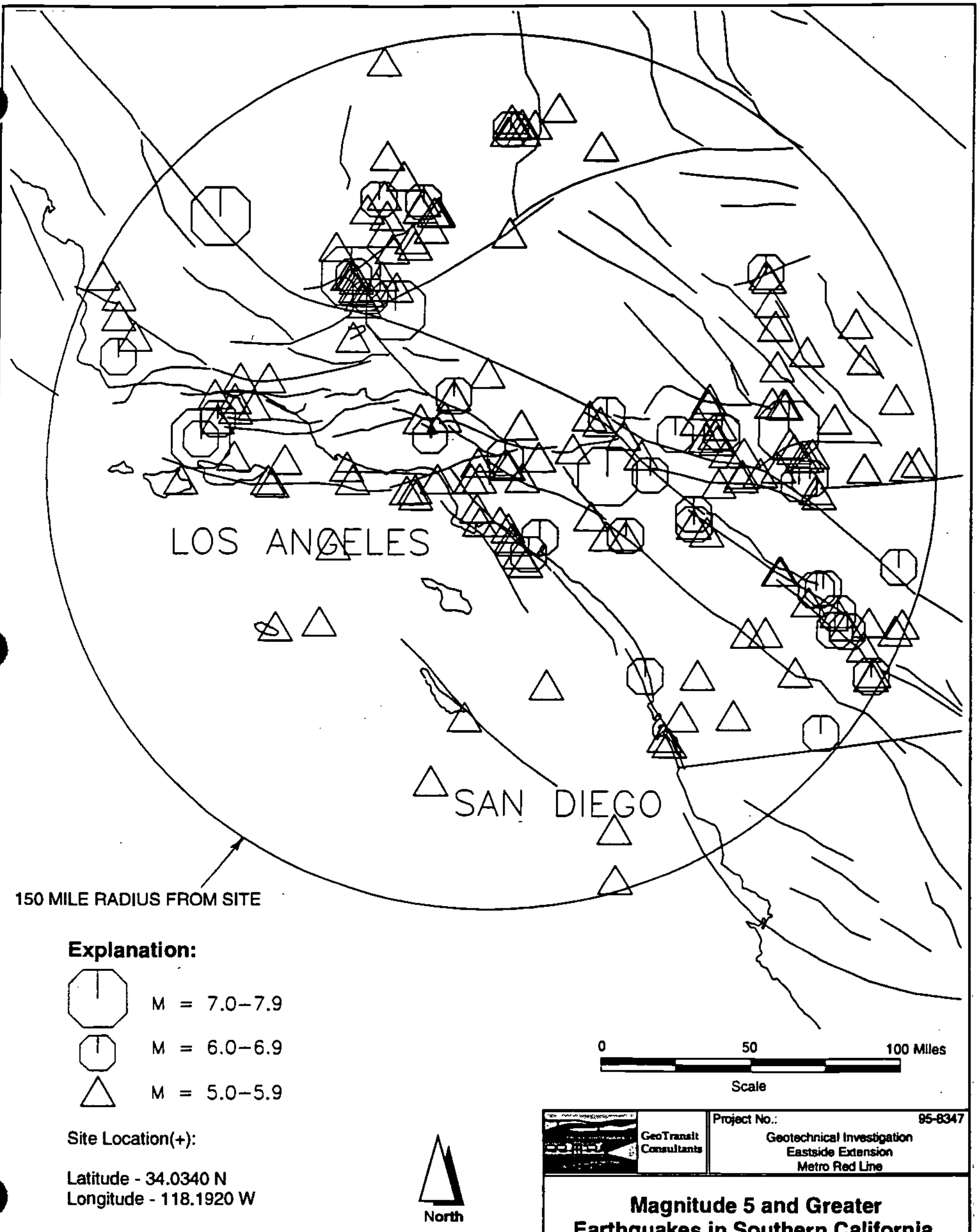
Magnitude 4.0 - 4.9 Earthquakes in Southern California, 1800-1993

12–95

Figure 4-3



Source:
Epicenters from Blake, 1992.

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Magnitude 5 and Greater Earthquakes in Southern California, 1800-1993

12–95

Figure 4-4

Narrows earthquake (M 5.9), with an epicenter about 10 miles east-northeast of the proposed First/Lorena Station. This earthquake occurred on a previously unknown northeast-dipping buried thrust fault that has since been named the Elysian Park seismic zone (Mualchin and Jones, 1992).

More recently, the M 6.6 Northridge earthquake occurred on January 17, 1994 on a previously unknown buried thrust fault dipping south beneath the alluvium of the San Fernando Valley. The epicenter of this earthquake was about 24 miles northwest of the station. Records of ground accelerations released by the California Division of Mines and Geology for a strong ground motion instrument located at City Terrace (approximately 2 miles northeast of the project area) indicates maximum free field accelerations of 0.32g horizontal and 0.13g vertical for the January 17, 1994 earthquake.

4.1.3 Regional Hydrogeology

The hydrogeology of the greater Los Angeles area includes two general types of groundwater regimes: bedrock uplands and alluvial lowland basins. The bedrock uplands surrounding most of the basins are generally considered to be non-water bearing, while adjacent alluvial basins have supplied groundwater that has been extensively used for domestic, commercial, and agricultural purposes.

The California Department of Water Resources (CDWR, 1961) divides the Los Angeles coastal plain into the Santa Monica, Hollywood, Central, and West Coast groundwater basins. The Central Basin of the coastal plain is further subdivided into the Los Angeles Forebay, Montebello Forebay, Whittier and Central Basin Pressure Areas. The subject segment of the Eastside Extension alignment lies entirely within the Los Angeles Forebay Area.

The Los Angeles Forebay Area extends southward from the narrows of the Los Angeles River and has been characterized by the CDWR as an area of unrestricted infiltration of surface water. Because of the presence of low permeability sediments in the shallow bedrock of the Repetto and Elysian Hills, however, the actual area of effective surface water infiltration to underlying aquifers is largely restricted to the younger and older alluvial deposits in the vicinity of the narrows.

Groundwater in the Los Angeles Forebay Area occurs in young alluvium and in older permeable Pleistocene sediments. Some limited groundwater also may be present in Pliocene and Miocene bedrock underlying these deposits. According to the CDWR (1961), the water-bearing sediments extend to depths on the order of 1,600 feet below the ground surface, particularly in the southern portions of the Forebay Area.

Aquifers underlying the Forebay Area in the vicinity of the subject segment of the alignment include the Semiperched, Gaspur, Exposition, Gage, and Gardena aquifers at increasing depths in the Holocene and Pleistocene sediments (CDWR, 1961). Because bedrock occurs at relatively shallow depths along the subject segment, only the upper Semiperched and Gaspur aquifers appear to be present in the project area (CDWR, 1961). The Semiperched aquifer consists of older Pleistocene deposits overlying bedrock near the Repetto and Elysian Hills. The Gaspur aquifer largely comprises the coarse-grained Holocene deposits overlying bedrock in the Los Angeles River Narrows. The aquifers are generally separated from each other by aquicludes, but the aquiclude materials may be locally absent in the northern part of the Forebay Area, allowing hydraulic continuity between aquifers. Local groundwater conditions are discussed in Section 4.3.2 below.

4.2 LOCAL SETTING

4.2.1 Local Geologic Conditions

Unconsolidated to weakly consolidated Pleistocene and Holocene alluvial sediments will be encountered during construction of the tunnels and station in the project area. In this investigation bedrock of the Miocene and Pliocene age Fernando/Puente Formation was encountered at Borings SD-16 and SD-17 at approximate depths of 123 feet and 99 feet BGS (approximately 58 and 36 feet below the planned station invert), respectively. Bedrock was also encountered at Boring PE-13 drilled during the preliminary engineering program at a depth of about 99 feet BGS (approximately 28 feet below the station invert). Plate 2 illustrates the subsurface conditions along the alignment based on the available information.

4.2.1.1 Surficial Deposits

A variety of surficial alluvial deposits are present in the project area. These deposits are differentiated on Plate 1 into three units: Older Alluvium and Older Gravel (map symbols Qoa and Qp, respectively) of

Pleistocene age, and overlying Young Alluvium (Qya) of Holocene age. Young Alluvium occurs primarily in the First/Lorena Station area within drainage courses eroded into the Older Alluvium.

Several fluvial terrace surfaces that range in age from middle Pleistocene to Holocene have been identified on the alluvial deposits (Bullard and Lettis, 1993). For the Older Alluvium, these are designated on the geologic map (Plate 1) by a numerical subscript on the map symbol (e.g. Qoa_1). The configuration of these surfaces indicates that the alluvial deposits south of the Repetto Hills area are being actively tectonically deformed and uplifted (Bullard and Lettis, 1993).

The Older Alluvium and Young Alluvium in the project area are extremely heterogeneous and non-uniform. No age-dating tests were performed on the alluvium samples to differentiate Young Alluvium and Older Alluvium. Thus, the actual thickness and areal extent of Young Alluvium overlying the Older Alluvium is unknown. In this investigation, no differentiation of young and old alluvial units was made in the borings. They are collectively referred to as "alluvium" and designated as "Qa" in the logs of borings in Appendix A of this report.

Alluvium in the project area consist of both granular and fine-grained units. Granular alluvium primarily consists of sands and gravels with varying amounts of silt and/or clay. Scattered cobbly zones were encountered in Borings SD-15, SD-16, SD-17 and PE-13 in the station area. Drilling through cobbly zones was usually accompanied by strong rig chatter and slow progress.

Intervals of fine-grained alluvial units generally ranging from less than a foot to 15 feet in thickness, and consisting of lean clays, silts and fat clay mixed with variable amounts of fine to coarse sand and some gravel, are interspersed with the granular alluvial units in the project area.

4.2.1.2 Bedrock

Bedrock strata of the Pliocene Fernando Formation crop out both to the north of the alignment in the City Terrace area of the southern Repetto Hills, and to the northwest of the Los Angeles River Narrows along the south base of the Elysian Hills. The Fernando Formation typically consists of massive to indistinctly bedded siltstone or mudstone and well bedded sandstone (Lamar, 1970). The older Miocene Puente Formation is exposed to the north of the Fernando Formation exposures and underlies much of the Elysian and Repetto Hills. The Puente Formation consists of well-bedded siltstone, claystone and very fine sandstone (Lamar, 1970). In the area shown in Plate 1, the contact between these formations is covered by alluvium over most of its extent. Where exposed, the contact is often difficult to locate accurately because the lithologic change between the formations can be gradational (Lamar, 1970). For engineering purposes the characteristics of both of the formations can be considered similar. We have therefore not attempted to differentiate the Fernando Formation from the Puente Formation during subsurface investigations; i.e. when referring to bedrock information obtained from borings, bedrock is designated as the Fernando/Puente Formation in the text, and on the boring logs by a dual symbol, Tf/Tp.

Bedrock was encountered at various depths ranging from about 28 to 58 feet below the planned station invert in borings SD-16, PE-13, and SD-17 in the First/Lorena Station area. The bedrock materials encountered in the borings consist predominantly of poorly bedded siltstone and claystone. Overall, the bedrock materials range from very soft to soft according to criteria provided by the bureau of reclamation in their "Engineering Geology Field Manual" (Figure A-2, Appendix A). Bedrock was not encountered in the three borings along the tail track tunnel alignment. Thus, bedrock is not anticipated to occur in the station and tunnel excavation in the project area.

4.2.2 Local Faulting and Folding

An east-west trending topographic escarpment (the Coyote Pass escarpment) is located approximately 1,600 feet north of the subject station. The escarpment forms the southern margin of the City Terrace area in the Repetto Hills and is as much as 80 feet high. It can be traced as an intermittent feature from near the channel of the Los Angeles River in the west to the southern base of the Monterey Park Hills near Atlantic Boulevard in the east (Plate 1). The escarpment is highest along the southern edge of the heights of City Terrace and diminishes to an indistinct feature that is less than 20 feet high near its intersection with the tunnel alignment northwest of the station. A second topographic escarpment occurs approximately 3,300 feet south of the station. The southerly escarpment has an east-northwest to northeast trend and its surface expression is relatively subdued when compared to the Coyote Pass escarpment.

Geologic studies following the 1987 Whittier Narrows earthquake (M 5.9) attribute these and similar escarpments in the Elysian and Repetto Hills of central and eastern Los Angeles to ongoing folding and faulting. Seismologic, geodetic, and geomorphic analyses indicate that the escarpments could result from either surface faulting or near-surface folding of weakly consolidated materials that overlie movements on deeply buried (or "blind") thrust faults. Concurrent investigations carried out by GeoTransit Consultants (1996a) to evaluate the Coyote Pass escarpment conclude that the escarpment as well as the southerly escarpment are the result of primarily fold deformation associated with faulting at some unknown depth. The station is located on the north limb of an east-west trending anticline. The southerly escarpment is actually the south flank of this broad fold which has an amplitude of more than one mile. The axis of the fold is roughly centered along Third Street, approximately 1,200 feet to the south of proposed First/Lorena Station.

No documented faults trend toward or cross the proposed station location and there are no Alquist-Priolo earthquake fault zones identified by the State in the area.

4.3 SUBSURFACE STRATIGRAPHY AND GROUNDWATER CONDITIONS

4.3.1 Subsurface Stratigraphy

Based on the subsurface information obtained from this investigation and previous investigations, Plate 2 presents the plan and generalized subsurface cross-sectional profiles showing the interpreted subsurface stratigraphy and proposed station and tail track tunnel profiles in the project area.

The subsurface stratigraphy along the proposed tunnel alignments and in the vicinity of the first/Lorena Station consists of shallow surficial fills (for pavement and structure subgrade) overlying alluvium and bedrock of the Fernando/Puente Formation. The bedrock was encountered 28 to 58 feet below the station invert in three borings (SD-16, PE-13 and SD-17) in the station area and was not encountered in any of the borings drilled along the tail track tunnels.

At the boring locations, the fill underlying existing roadway pavements ranges up to about 7 ½ feet in thickness and consists predominantly of base course, and subbase materials including lean clay, clayey silt, gravelly clay, clayey gravel with varying amounts of sand, and silty sand. Locally thicker layers of fill may exist, especially in the vicinity of underground utilities.

The alluvium below the fill is heterogeneous. Within the depths of exploration the alluvium is predominantly granular with interspersed layers of fine-grained soils. The granular alluvium consists of loose to very dense gravel (with and without sand and/or silt), poorly to well graded sands (with and without gravel and/or silt) and silty sand (with and without gravel) with local zones of cobbles, and occasional clayey sand and clayey gravel. Unified Soil Classification System (USCS) classifications of granular alluvium encountered include GP, GM, GC, SW, SP, SW-SM, SP-SM, SC-SM, SM, and SC.

The fine-grained alluvium consists predominantly of medium stiff to hard lean clays, silts and fat clay with various amounts of sand and silt. USCS classifications of encountered fine-grained alluvium include CL, ML and CH.

The equivalent standard penetration test (SPT) blowcounts in the alluvium range from 3 blows per foot to values in excess of 100 blows per foot. Loose granular soils (SPT blowcounts less than 10) and soft (SPT blowcounts between 2 and 4) to medium stiff (SPT blowcounts equals 4 to 8) fine-grained soils are generally located less than 20 feet BGS within the station area. Elsewhere, the granular alluvium is generally medium to very dense, and the fine-grained alluvium is very stiff to hard.

Based on the information from the five borings drilled in the station area (i.e., Borings SD-15 through SD-18 and PE-13) and the current station profile (Plate 2), the station excavation will encounter about 75 percent granular alluvium and 25 percent fine-grained alluvium. A significant portion of the encountered granular materials above the station invert are anticipated to be susceptible to raveling and running conditions.

Similarly, the soils within the tail track tunnel zone are predominantly granular in nature as shown in Plate 2. In general, the alluvium in this zone is similar to that encountered in the station area in terms of composition and consistency, except no cobbly zones were encountered in any of the tail track tunnel borings (DD-27, DD-27S, DD-28 and DD-62).

4.3.2 Groundwater Levels

Groundwater levels in the project area were monitored in three monitoring wells consisting of two (SD-15 and DD-27S) installed during this investigation, and one (PE-13) installed during the preliminary investigation (GeoTransit Consultants, 1994a). The observed groundwater levels are summarized in Table 3-2. The most recent groundwater level readings are also shown in Plate 2.

In general, the current groundwater levels are approximately 80 to 100 feet BGS in the project area. The currently observed groundwater levels apparently dip toward the south and southeast with a gradient of about 1 percent. Based on current station and tunnel profiles and assuming current groundwater levels remain unchanged during construction, it is anticipated that the bottom of the station excavation will be approximately at the groundwater level while the tunnel excavation will be above groundwater.

The project area is located in a region with known significant fluctuations in historic groundwater level data. Table 4-2 summarizes available groundwater level data from a number of wells in the general area that are maintained by the Los Angeles Department of Public Works. Locations of these wells with respect to the project area are shown in Plate 1.

An evaluation of the record shown in Table 4-2, indicates that groundwater levels in the general region can significantly fluctuate. As shown in Table 4-2, the observed groundwater levels at some of the well locations varied as much as 12 feet in one month (Well No. 2818D), 69 feet in one year (Well No. 2808A), and 183 feet in 10 years (Well No. 2818C). These records imply that the groundwater levels in the project area are likely to be time-dependent and can significantly fluctuate. Potential fluctuation in groundwater levels must be considered in the design and construction of the tunnel and station facilities in the project area.

The presence of occasional less permeable lean clay, fat clay, clayey sand and silt layers interspersed within the granular alluvium indicates the potential existence of local perched groundwater zones within the station excavation depths and tunnel zone.

TABLE 4-2. GROUNDWATER LEVEL RECORDS FROM EXISTING WELLS MAINTAINED BY LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Water Well*	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
			Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2765	259.0	-	109.1	+149.9	3-79	-	-	-	Water well
			90.0	+169.0	10-38				
			113.6	+145.4	4-72				
2775	268.0	100	52.6	+215.4	10-75	-	0-76	76-100+	Water well
			40.2	+227.8	11-34				
			70.8	+197.8	6-40				
2807	200.0	556	156.2	+43.8	11-90	-	0-556+	-	Water well
			126.6	+73.4	3-35				
			286.2	-86.2	4-60				
2808A	185.0	744	196.2	-1.2	11-77	-	0-744+	-	
			192.8	-8.5	4-71				
			220.7	-77.8	12-72				
2808C	197.9	-	241.7	-43.8	4-78	-	-	-	Water well
			239.8	-41.9	4-70				
			284.0	-86.1	10-60				
2818B	188.6	-	162.0	+26.6	4-92	-	-	-	Water well
			131.5	+57.1	2-43				
			388.0	-199.4	5-59				
2818C	187.2	-	191.0	-3.8	10-88	-	-	-	Water well
			155.0	+32.2	1-47				
			338.0	-150.8	7-57				

TABLE 4-2. GROUNDWATER LEVEL RECORDS FROM EXISTING WELLS MAINTAINED BY LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Water Well*	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater(1)			Geologic Unit (depth in feet)			Comments
			Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2818D	184.5	-	351.0 146.0 363.0	-166.5 +38.5 -178.5	7-78 7-44 6-78	-	-	-	Water well
2827A	227.0	680	196.2 192.8 220.9	+30.8 +34.2 +6.3	11-60 4-59 5-51	-	0-680	680+	Destroyed in 1961
2827F	211.1	-	186.8 185.8 352.0	+24.3 +25.3 -140.9	5-83 11-82 4-66	-	-	-	Water well
2827G	200.4	-	184.0 209.0 348.0	+16.4 -8.6 -147.6	4-92 2-51 9-60	-	-	-	Water well
2827J	210.5	-	201.3 133.2 262.8	+9.2 +77.3 -52.3	11-79 11-74 1-62	-	-	-	Water well
2828	192.9	306	182 119.9 228.0	+10.9 +73.4 -35.1	9-51 3-35 3-50	-	0-306+	-	Destroyed in 1957
2828A	193.0	-	189.9 120.0 206.9	+3.1 +7.3 -13.9	9-51 1-35 7-49	-	-	-	

TABLE 4-2. GROUNDWATER LEVEL RECORDS FROM EXISTING WELLS MAINTAINED BY LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Water Well*	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater(1)			Geologic Unit (depth in feet)			Comments
			Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2828D	196.0	-	244.0 202.0 378.0	-48.0 -6.0 -182	2-77 2-51 9-60	-	-	-	Water well
2828E	195.0	-	236.5 210.5 346.5	-41.5 -15.5 -151.5	11-80 4-51 5-61	-	-	-	Water well
2828F	185.8	-	175.0 170.0 323.0	+10.8 +15.8 -137.2	4-92 4-85 9-57	-	-	-	Water well
2828G	193.1	-	170.0 194.0 342.0	+23.1 -0.9 -148.9	4-92 4-77 9-60	-	-	-	Water well
2837A	196.0	407	203.0 107.0 352.5	-7 +89 -156.5	1-68 1-46 9-60	-	0-407+	-	
2837B	196.5	-	158.0 123.2 368.0	+38.5 +73.3 -171.5	4-92 1-35 7-74	-	-	-	Water well
2838A	174.2	500	131.0 131.0 277.0	+43.2 +43.2 -129.8	4-92 4-92 7-78	-	0-500+	-	Water well

TABLE 4-2. GROUNDWATER LEVEL RECORDS FROM EXISTING WELLS MAINTAINED BY LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Water Well*	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater(1)			Geologic Unit (depth in feet)			Comments
			Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2838B	161.6	-	142.0	+19.6	4-92	-	-	-	Water well
			138.4	+23.2	1-50				
			263.4	-101.8	9-57				
2847	190.0	534	201.5	-11.5	11-80	-	0-534+	-	Water well
			116.0	+74.0	2-42				
			360.5	-170.5	5-61				
2847B	203.7	470	145.0	+58.7	4-92	-	0-470+	-	Water well
			145.0	+58.7	4-92				
			323.0	-119.3	7-57				
2847C	228.3	-	200.0	+28.3	4-92	-	-	-	Water well
			150.8	+77.5	4-35				
			322.0	-93.7	9-60				
2857C	228.0	-	212.0	+16.0	11-80	-	-	-	Water well
			212.0	+16.0	2-70				
			336.0	-108.0	8-60				

Note: * Refer to Plate 1 for well locations with respect to the project area.

4.3.3 Soil and Groundwater Contamination

An assessment of the soil and groundwater contamination for the project area has been presented in the Stage II environmental site assessment report for the Eastside Extension prepared by GeoTransit Consultants (1994b). The results of the Stage II environmental site assessment together with the data obtained from the current investigation and available data from other previous investigations were utilized in evaluating the potential for soil and groundwater contamination.

The scope of environmental monitoring in this investigation was limited to field observation of sulfurous and/or hydrocarbon odors and screening soil samples with an OVA for the potential presence of volatile organic compounds (VOCs). The results of OVA monitoring are described in Section 3.6. In general, no significant hydrocarbon and/or sulfur odors were detected from soil samples above the station and tunnel inverts. Significant OVA readings (exceeding 10 ppm above background levels) were noted in two soil samples below the station invert at Boring SD-15 (at 95 feet BGS as compared with the station invert at about 78 feet BGS) and Boring SD-17 (at 105 feet BGS as compared with the station invert at 63 feet BGS).

4.3.3.1 Groundwater Contamination

No evidence of floating product was detected during development and groundwater level monitoring of Monitoring Wells SD-15 and DD-27S. In the Stage II environmental site assessment program (GeoTransit Consultants, 1994b), chemical tests performed on groundwater samples from Monitoring Well EB-12A (approximate Station CR 169+35, i.e., about 1,930 feet from the western terminus of First/Lorena Station) detected no hydrocarbon, sulfide or organic lead contamination.

As described previously, the currently observed groundwater levels are at or below station and tunnel inverts. Thus, no dewatering prior to construction is anticipated. However, the potential for localized groundwater inflows into the excavation exists. Judging from the above observations, most of these groundwater inflows, are likely to have a low potential for contamination. However, the potential for hydrocarbon contamination of some of these inflows cannot be completely precluded due to the proximity of the project area to the existing Boyle Heights oil field. Such inflows, if encountered, must be tested. Proper disposal of contaminated inflowing water, if any, will be required to meet applicable provisions for disposal, including all applicable standards, conditions, and requirements imposed by the City of Los Angeles Sanitation District and the California Regional Water Quality Control Board (RWQCB).

4.3.3.2 Soil Contamination

Results of chemical tests on soil samples from two borings (EB-10 and EB-11) performed in the Stage II environmental site assessment investigation (GeoTransit Consultants, 1994b) and available field observation data from this investigation indicate that the soils within the station and tunnel zone are "clean"; i.e. hydrocarbon, sulfide or organic concentration levels are less than the generally accepted action levels published by the California Department of Health Services and RWQCB (refer to Notes 1 and 2 in Table C-5 in Appendix C). Thus, no special disposal requirements are anticipated. However, the localized significant OVA readings detected from two soil samples below the station invert in Borings SD-15 and SD-17, and the proximity of the project area to the existing Boyle Height oil field indicate that there is a possibility that some excavated materials may locally have hydrocarbon contamination. The locally contaminated materials, if any, will require treatment or disposal at a facility approved for disposal of contaminated waste.

4.3.3.3 Gassy Conditions

The potential accumulation of methane and other gases within oil fields in the Los Angeles Basin is well known. The project area is located about 2,500 feet from the closest known boundaries of the Boyle Heights oil field. Oil fields are known sources of methane and other oil field gases such as hydrogen sulfide. Furthermore, ESI (1994) measured methane concentrations of 3.6 percent and 2.2 percent by volume, or 74 percent and 44 percent of lower explosive limit (LEL) for gas samples from shallow and deep probes, respectively, at Boring MW-6 (located in the First/Lorena station area). Thus, the possibility of encountering gassy conditions in the project area exists.

4.4 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

The engineering properties of subsurface materials, as obtained from results of field and laboratory tests, are summarized in Table 3-6. Blowcount data from drive sampling and standard penetration tests (SPT N-Values) are shown in the borehole logs and presented in Table 3-6.

Table 4-3 presents a summary of the measured/interpreted ranges of relevant geotechnical parameters for the various material types encountered within the project area. For purposes of presentation, the alluvium has been broadly categorized into fine-grained and coarse-grained alluvium. The alluvium within the project area is predominated coarse grained with varying amounts of gravel and local cobbly zones some of the high blowcounts recorded within the coarse grained alluvium are due to the presence of gravels and cobbles within the alluvium and do not reflect the relative denseness of the gravelly and cobbly alluvium. Also, due to the presence of gravels and cobbles larger than the sampler size, the results of gradation, only reflect the gradation of the matrix materials and are therefore finer than the actual gradations of the deposit on some of the gravelly and cobbly alluvium.

No engineering properties are presented for the locally present surficial fill which is considered to have little or no effect on the station design. Station and tunnel excavation and construction will be primarily within the alluvium. Based on available geotechnical data and engineering evaluations, the static and dynamic engineering properties for the alluvium and Fernando/Puente Formation bedrock are summarized in Tables 4-3 and 4-4. The engineering properties are presented and used in the engineering analysis presented in Section 5.0.

The following section provides a description of relevant engineering properties of the subsurface materials in the project area.

4.4.1 Alluvium

4.4.1.1 Grain Size Distribution

The alluvium in the tunnel envelope and station excavation area is heterogeneous and is predominantly coarse-grained. Results of grain size distribution and fines content (percentage passing #200 sieve) tests are summarized in Table 3-6. The bulk of the gradation and fines content tests were performed on selected granular samples within and immediately surrounding the tunnel envelope in tail track tunnel borings, and above the station invert in station borings. This was done primarily to evaluate areas of cohesionless sands and gravels which may be susceptible to raveling/running/flowing conditions. As shown on the boring logs (Appendix A), frequent zones of gravels and scattered zones of cobbles were also encountered in the geotechnical borings.

TABLE 4-3 SUMMARY OF ESTIMATED ENGINEERING PROPERTIES

(FIRST/LORENA STATION & TAIL TRACK TUNNELS)

ENGINEERING CHARACTERISTICS	FINE-GRAINED ALLUVIUM		COARSE-GRAINED ALLUVIUM		Fernando/Puente Formation	
USCS Classification	CL,CH,ML,MH CL-ML		SP,SW,GP,GW,SM,SC GP-GM,GW-GP SP-SM,SW-SM,SC-SM		ML,MH,CL,CH	
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
Equivalent SPT Blow Counts	3->100	-	4->100	-	31->100	-
Insitu Moisture Content (%)	12-33	20	8-25	14	18-32	25
Insitu Dry Density (pcf)	87-121	108	90-133	112	92-114	105
Void Ratio	0.4-0.95	0.57	0.24-0.84	0.48	0.49-0.85	0.62
Fines Content(% passing #200 Sieve) (%)	50-96	-	6-47	-	84-98	-
Specific Gravity	-	2.72	2.63-2.69	2.65	2.72-2.73	2.72
Liquid Limit (%)	24-63	41	-	-	26-69	40
Plasticity Index (%)	8-40	23	-	-	1-31	15
Peak Shear Strength: Cohesion (psf)	600-2000	600	0-1200	0	700-5900	5000
Friction Angle, (degrees)	25-31	28	32-45	35	25-34	25
Uniaxial Compressive Strength (psf)	-	-	-	-	4900-20310	12000
Poisson's Ratio	0.3-0.45	0.40	0.3-0.4	0.30	0.4-0.49	0.4
Static Young's Modulus (10^3 psf)						
0-20'	250-1500	600	250-1500	600	-	-
20'-60'	1000-2400	1500	1000-2400	1500	-	-
Below 60'	1600-4600	2000	1600-4600	2000	1200-3000	1800
pH	6.8-7.9	-	7.0-8.5	-	-	-
Chloride Content (ppm)	57-907	-	90-110	-	-	-
Sulphate Content (ppm)	54-138	-	46-95	-	-	-
Electrical Resistivity (ohms-cm)	1531-<2941>	-	862-3378	-	-	-
Compression Index- Cc'	0.065*-0.082	0.07	0.034*-0.07*	0.05	0.05-0.092*	0.07
Swelling Index-Cs'	0.008*-0.02	0.01	0.004*-0.01*	0.005	0.012*-0.022*	0.012
Rate of Secondary Compression-Cx'	0.0006-0.0013*	0.001	0.00035*-0.0014	0.0004	0.0004-0.0009*	0.0009
Swelling (+)/Collapse (-), %	0.16*-(+0.52)	-	-0.01*-(-0.17)	-	0*-(+0.88)	-

NOTES:

1. Results of gradation, insitu moisture content and insitu dry density tests on gravelly and cobbly alluvium are only representative of the finer matrix materials in the alluvium.
2. Shear strength parameters (cohesion and friction angle) were obtained by direct shear tests.
3. Electrical resistivity tests, in general were conducted at in situ moisture content. The value of <> parenthesis corresponds to the minimum electrical resistivity at adjusted moisture condition of the sample based on DOT CA Test 532.
4. Compression Index (Cc') and Swelling Index (Cs') are based on vertical strain-log stress plots. Rate of Secondary Compression (Cx') is based on vertical strain-log time plot.

* Incorporated data from other segments of the Eastside Extension Alignments (GeoTransit Consultants, 1996).

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TABLE 4-4 ENGINEERING PROPERTIES FOR DYNAMIC ANALYSIS

(First/Lorena Station)

MATERIAL PROPERTY	ALLUVIUM						Fernando/Puente Formation Bedrock	
	0 to 20 feet		20 to 60 feet		Below 60 Feet			
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
Shear Wave Velocity (ft/sec)	500-1200	750	1000-1500	1200	1200-2000	1350	1100-1500	1250
Poissons Ratio	-	0.35	-	0.3	-	0.45	-	0.49
Shear Modulus [1] (10^3 ksf)	1.0-5.8	2.5	4.0-9.1	6.0	5.8-16.0	7.0	4-10	6.0
Youngs Modulus [1] (10^3 ksf)	2.7-15.7	6.0	10.5-23.6	15.0	16.7-46.4	20.0	12-30	18.0
Damping	See Note (2)							

Notes:

[1] Values correspond to small strains (shear strain <= 0.001%).
Apply the reduction factor for other strain values:

Shear Strain (%)	**Reduction Factor**
1.0E-02	0.65
1.0E-01	0.2
1.0E+00	0.05

[2] Recommended damping value is strain-dependent as follows:

Shear Strain (%)	**Damping (%)**
2.0E-03	5
1.0E-02	10
5.0E-02	15
2.0E-02	20

Results of gradation tests show the presence of significant zones of granular alluvium with low fines content (poorly to well graded sands and gravels) within tunnel and station excavations. Such zones exist within a major portion of the tunnel and station area.

4.4.1.2 In Situ Conditions and Index Properties

Laboratory test data indicate that the dry densities of the alluvium range from 87 to 133 pcf and the in situ moisture content ranges from 8 to 33 percent. The relatively wide ranges of density and moisture content values and the large number of soil types encountered in the alluvium are indicative of the nonuniform and heterogeneous nature of the alluvium in the project area.

Atterberg limit tests on the limited fine-grained alluvium encountered indicate that these materials are predominantly silts, lean clay and fat clay with liquid limits ranging from 24 to 63 and plasticity indices ranging from 8 to 40.

4.4.1.3 Shear Strengths

Shear strength parameters for the alluvium were derived based on the results of direct shear tests on selected samples from the current and preliminary investigations. Results of these direct shear tests are graphically presented in Figures 4-5, 4-6, and 4-7.

4.4.1.4 Static and Dynamic Modulus

The dynamic modulus and Poisson's ratio for the alluvium were estimated based on results of pressuremeter tests and two downhole geophysical surveys, available literature on similar materials, available correlations with SPT data, and engineering judgement. Modulus of soil is strain-dependent. The modulus value determined from seismic velocity surveys correspond to small strain ranges appropriate for dynamic loading situations (shear strain about 10^{-3} percent or less). Under static loading, shear strains are expected to be considerably larger and the corresponding moduli should be reduced in accordance with the recommended reduction factors shown in Table 4-4.

4.4.1.5 Consolidation and Collapse Potential

Results of four consolidation tests on fine-grained alluvium and two collapse potential and consolidation tests on finer coarse-grained alluvium are presented in Table 3-6. The measured collapse potential of the two coarse-grain alluvium samples saturated at in-situ overburden pressures are 0.1 percent and 0.17 percent. Thus, the collapse potential of the finer coarse-grained alluvium in the event of saturation is considered small.

4.4.1.6 Corrosion Potential

Results of soluble sulfate content tests (46 to 138 ppm) summarized in Table 4-3 indicate that alluvial materials tested are not corrosive to concrete. Results of electrical resistivity tests (862 to 3,378 ohm-cm) indicate that these materials are corrosive to very corrosive to metals.

4.4.2 Fernando/Puente Formation Bedrock

The Fernando/Puente Formation bedrock was encountered below station invert in two station borings (SD-16 and SD-17) drilled in this investigation, and one boring (PE-13) drilled in the preliminary engineering program. Since the encountered bedrock is apparently at least 28 to 58 feet below the station invert, the most important parameters with respect to station design are its stiffness and consolidation characteristics. Thus, emphasis of laboratory testing on bedrock samples was placed on evaluating these characteristics. Available engineering data on similar bedrock materials obtained from other portions of the Eastside Extension (GeoTransit Consultants, 1996b) were also utilized in developing the engineering properties of the Fernando/Puente Formation bedrock shown in Tables 4-3 and 4-4.

4.4.2.1 In Situ Conditions and Index Properties

The Fernando/Puente Formation bedrock in the project area consists predominantly of very soft to soft (based on Bureau of Reclamation, Engineering Geology Manual Classification) claystone, clayey siltstone and siltstone.

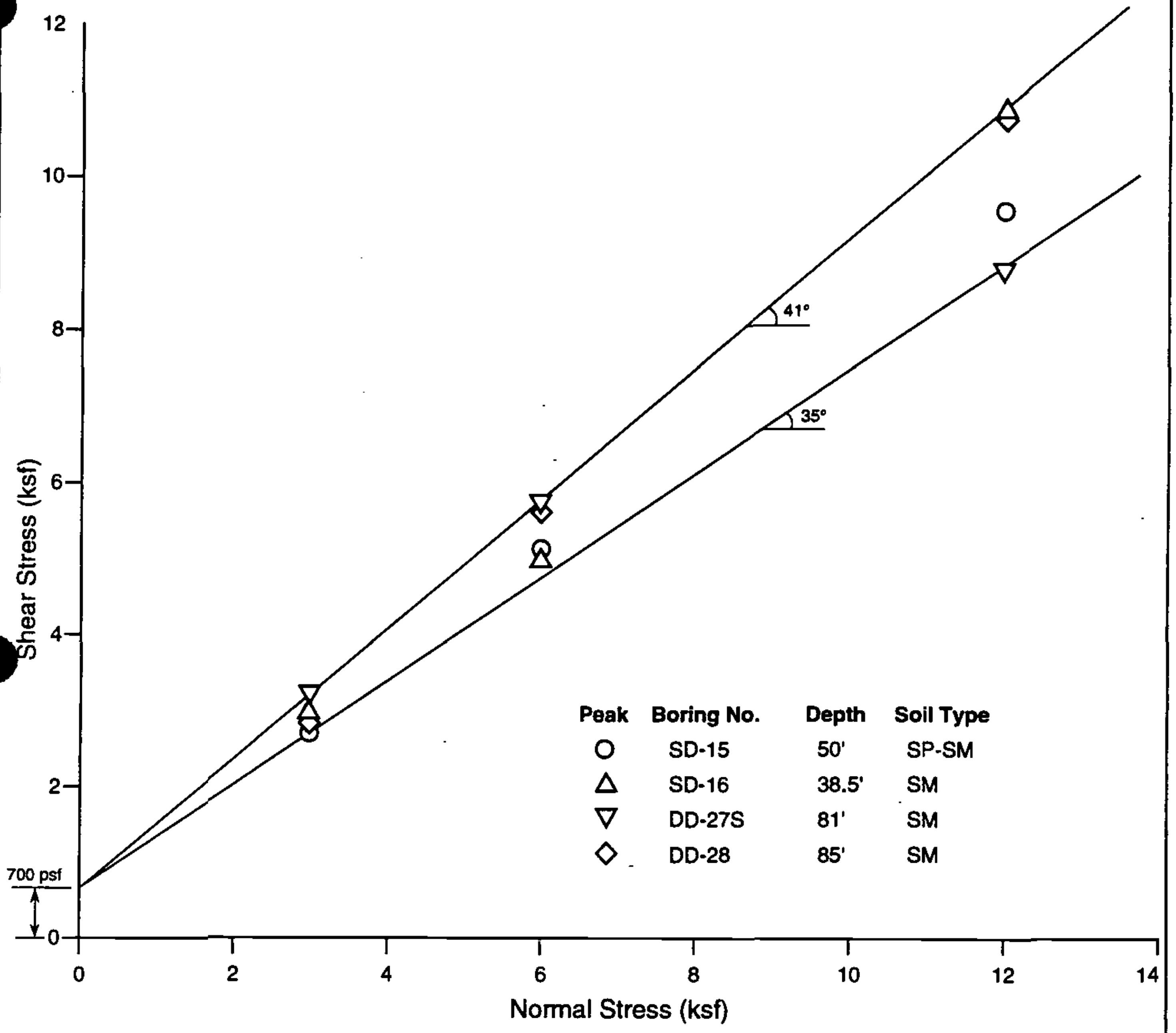
In situ dry density of the bedrock ranges from 92 to 114 pcf, and in situ moisture content ranges from 18 to 32 percent. The fine grained soils derived from the bedrock are predominantly silts, and lean and fat clays with liquid limits ranging from 29 to 69, and plasticity indices ranging from 1 to 31.

4.4.2.2 Static and Dynamic Modulus Characteristics

Static and dynamic modulus characteristics of the Fernando/Puente Formation bedrock were estimated based on data from similar bedrock materials in other portions of the Eastside Extension (GeoTransit Consultants, 1996b). Results are presented in Tables 4-3 and 4-4.

4.4.2.3 Compressibility

Results of a single consolidation test on a sample of claystone are presented in Table 3-6. Compressibility parameters presented in Table 4-3 are based on this test and other tests on similar bedrock materials from other portions of the Eastside Extension (GeoTransit Consultants, 1996b).



Note:
Direct shear tests were conducted on samples taken from First/Lorena Station and Tall Track Tunnels Borings.

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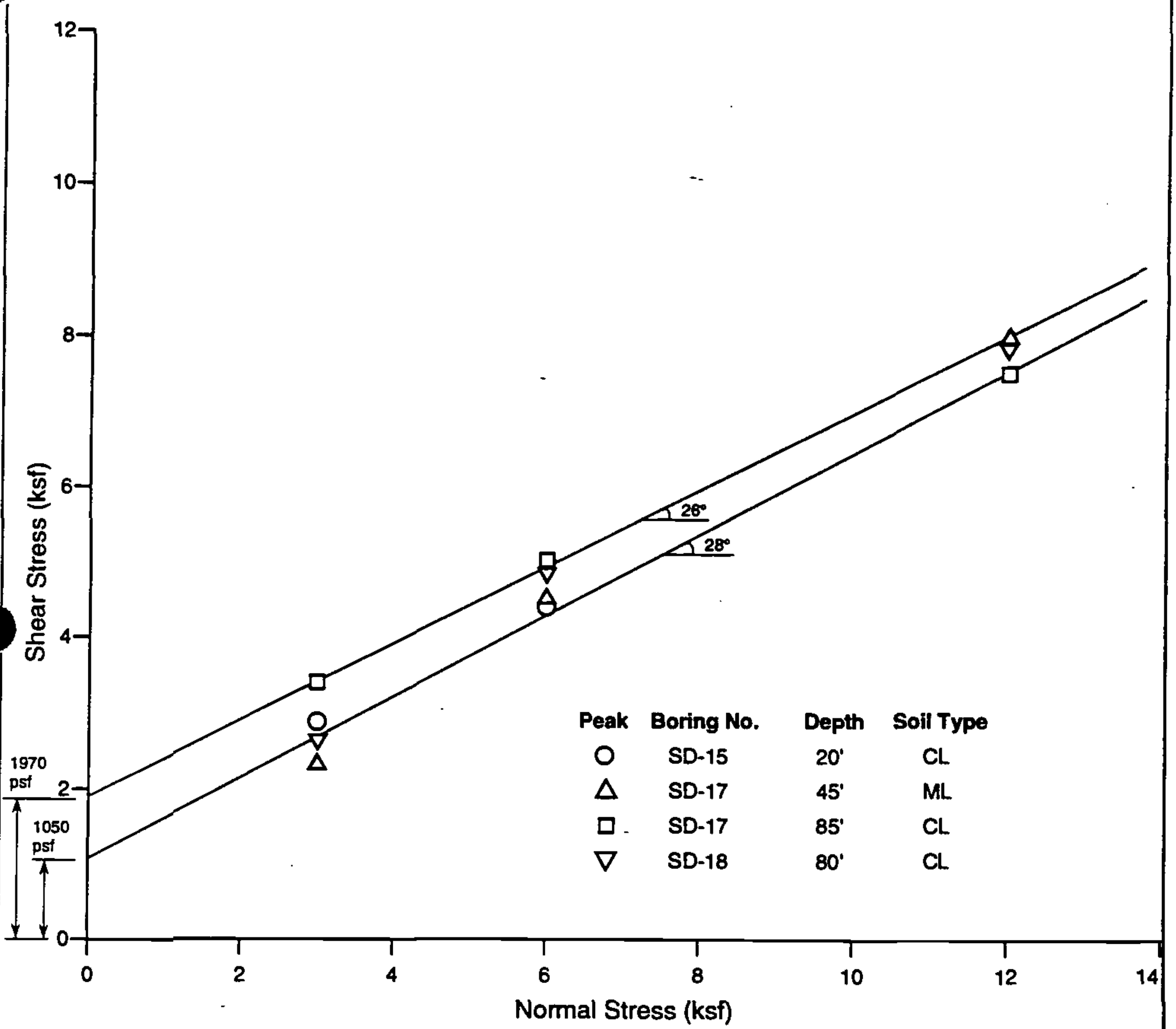
Direct Shear Test Results on Coarse-Grained Alluvium

12-95

Figure 4-5



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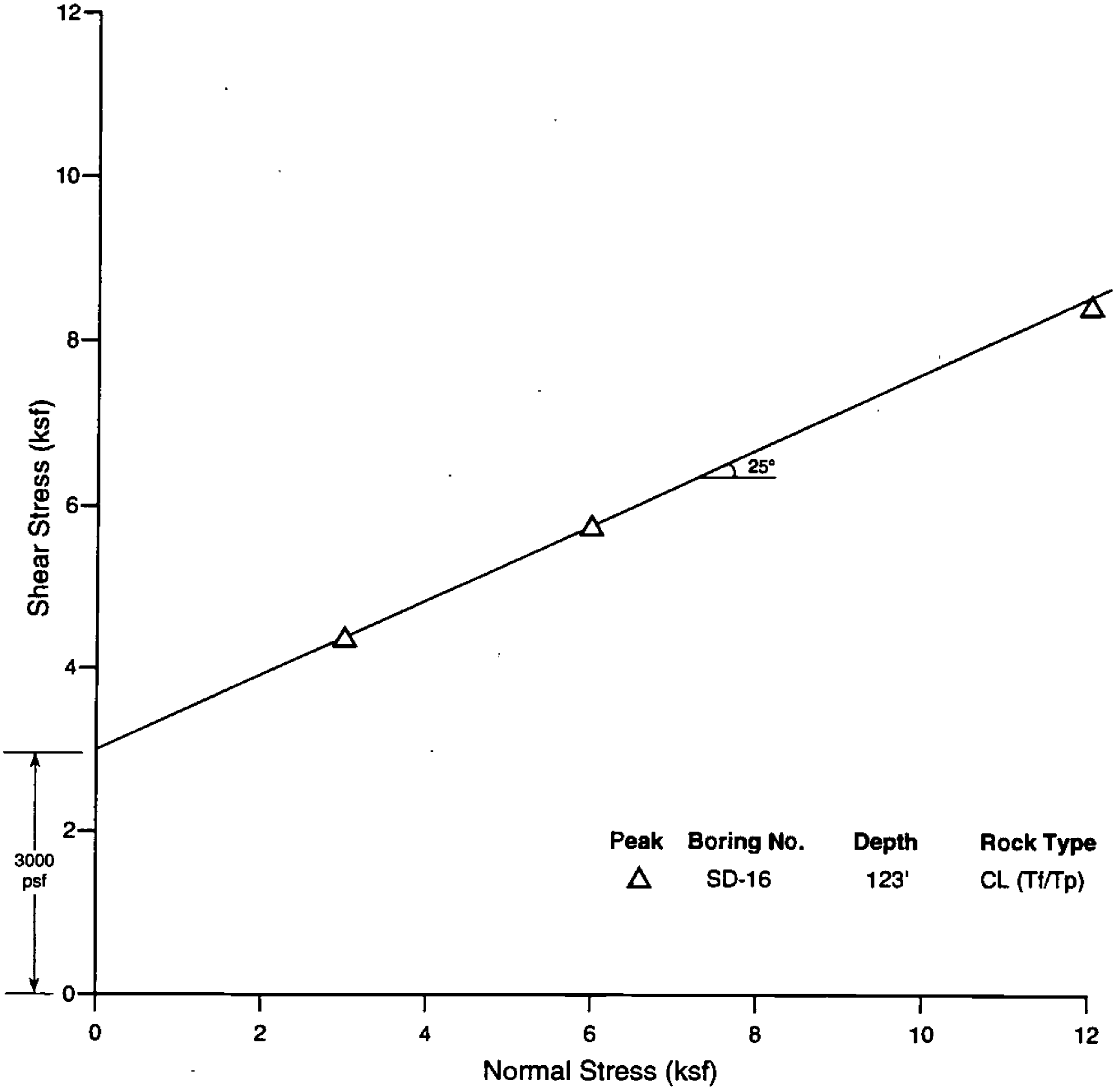
Direct Shear Test Results on Fine-Grained Alluvium

12-95

Figure 4-6

Note:
Direct shear tests were conducted on samples taken from First/Lorena Station and Tail Track Tunnels Borings.



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Eastside Extension
Metro Red Line

Direct Shear Test Results on Fernando/Puente Formation Bedrock

12-95

Figure 4-7

5.0 DESIGN AND CONSTRUCTION

5.1 GENERAL

This section provides a description of geotechnical evaluations and recommendations, and key geotechnical issues for the design and construction of the First/Lorena Station and the connecting Tail Track Tunnels.

5.2 SUMMARY OF SUBSURFACE STRATIGRAPHY

Based on current plans and profiles, the First/Lorena Station and Tail Track Tunnels will be entirely within alluvium. The alluvium is heterogenous and non-uniform. Based on the available subsurface information, the alluvium within the station excavation consists predominantly of granular alluvium that is interbedded with layers of fine-grained alluvium. Granular alluvium within the station excavation consists predominantly of sands and gravels with scattered zones of cobbles, and occasional localized layers of clayey sand and clayey gravel. The fine grained alluvium within the station excavation is predominantly lean clay, silt and fat clay with various amounts of sand and silt. Available subsurface information also indicates that in general, the alluvium within the tunnel envelope is similar to that encountered in the station area in composition and consistency, except that no cobbly zones were encountered in the three tunnel borings.

The granular alluvium would be susceptible to raveling, running/flowing conditions within station/tunnel excavations. The cobbles encountered are typically very hard unweathered granitic and metamorphic rock types.

5.3 GROUNDWATER LEVEL FLUCTUATIONS

Groundwater levels are an important input to assess dewatering needs during construction and evaluate appropriate hydrostatic pressures for the design of tunnel and station structures. As described in Section 4.3.2 the project area is located within a region where the groundwater levels appear to fluctuate significantly as evidenced by the available well records summarized in Table 4-2. This significant groundwater fluctuation indicates that it is difficult to predict the groundwater levels during construction or the maximum groundwater levels that the station structures may experience during their design life. For design purposes, the following groundwater levels were assumed:

- Groundwater levels in the project area during tunnel and station construction will be the same as the most recent groundwater levels measured in this investigation.

- The design groundwater level corresponding to the maximum sustained groundwater level over the design life of the structure can be represented by the groundwater level at Elevation 280 feet, approximately the same elevation as the top of the station box. This level is approximately 50 feet above the current water level in the area.

We understand that EMC applies a load factor of 1.7 on groundwater pressures in tunnel and station design in accordance with the current ACI code requirements, to account for potential future groundwater fluctuations above the design level. The above recommended design groundwater level is considered to be a reasonable estimate. There exists a potential for groundwater levels to periodically rise higher than the recommended design value. To account for this possibility and, at the same time avoid over conservatism in design, the following are recommended:

1. Use the design groundwater level recommended above in conjunction with the current ACI code requirements (i.e. apply a load factor of 1.7).

2. Check the design by using a load factor of one and assume groundwater levels to be at the ground surface.

It is recommended that groundwater levels be monitored prior to and during the construction. The geotechnical evaluation and recommendations, especially those related to groundwater dewatering, should be examined and, if needed, modified to reflect the as-measured groundwater level at the time of construction.

5.4 SEISMIC DESIGN CONSIDERATIONS

A project-specific seismic hazard analyses to determine appropriate design earthquake accelerations for the Eastside Extension was not part of the scope for this investigation. As per EMC's request, the seismic criteria established in a 1983 report entitled "Seismological Investigation & Design Criteria" prepared by Converse and others (1983) for the Metro Rail Project, was adopted in our analyses. For geotechnical analysis and design purposes, the maximum design earthquake (MDE) with a maximum horizontal ground acceleration of 0.6g as recommended in the above report was used. The magnitude of the corresponding design earthquake was estimated at 7.0. An earthquake corresponding to the Operating Design Earthquake (ODE) specified in the 1983 report by Converse and others with a maximum ground acceleration of 0.3g was assumed for estimating the earthquake loading imposed on shored excavations during construction.

There have been significant changes in our understanding of the seismicity of the area, the state of the art in seismic hazard analyses, and local code requirements since 1983. If the seismic design criteria are revised in the light of these changes, the seismic related geotechnical parameters and recommendations presented in this report should be reviewed and modified as appropriate.

5.5 FIRST/LORENA STATION WITH CROSSOVER

5.5.1 General

The planned cut-and-cover construction of the Station with crossover will involve about 60 to 78 feet of excavation from the ground surface to the planned station invert and the construction of the station structure. The excavation will be primarily in the alluvium underlying shallow surficial fills. A detailed description of the alluvium has been presented in Section 4.3 and a summary provided in Section 5.2. The station structure is proposed as a water-tight, rigid reinforced concrete box structure bearing on the alluvium.

The primary geotechnical considerations that require geotechnical engineering evaluation for design and construction of the planned station facilities include the following:

- Excavation methods

- Construction dewatering and related issues
- Excavation-related temporary shoring systems
- Foundation design and recommendations for soil, water and earthquake loading on the permanent station structures
- Evaluation of potential for earthquake-induced liquefaction and its potential effects on the station structure.

5.5.2 Excavation Methods

Station excavation will be primarily in alluvium which comprises roughly 75 percent granular alluvium and 25 percent fine-grained alluvium. Based on the results of this investigation and design and construction experience in similar subsurface conditions, it is anticipated that the cut-and-cover excavation can be achieved using conventional excavation methods. However, suitable excavation equipment to handle the very dense gravelly and cobbly alluvium will be required. Most of the granular alluvium in the station area will run and ravel readily. The fine-grained alluvium will have a tendency to slake if subject to changes in moisture conditions. Thus, timely application of ground support is important to prevent ground loss.

5.5.3 Dewatering and Groundwater Control

The most recent observed groundwater level data indicate that the groundwater table in the station area is about at or slightly below the planned bottom of station excavation. Minor preconstruction dewatering should be anticipated provided the groundwater levels during station construction remain at current levels. The potential for significant fluctuation of the groundwater levels in the area dictates the need for continuing groundwater level monitoring before and during station construction to verify this groundwater level assumption and to re-evaluate the dewatering needs, as appropriate.

Localized groundwater inflows can be anticipated due to the local presence of perched groundwater conditions. If these inflows are large enough, the accumulated inflow may need to be collected and pumped out of the excavation using ditches and sumps.

5.5.4 Sloped Excavation

Compared to shored excavations, sloped excavations will require more construction space and increase the volume of excavated materials. Sloped excavations can be used for the station's structural components that require shallower excavations, or can be used to reduce the height of shoring if sufficient easements can be obtained.

Temporary slopes in alluvium should be no steeper than 1½H:1V (1½ horizontal to 1 vertical). If heavy loads (stored materials, cranes, etc.) are anticipated at the top of the slopes, the slopes must be modified accordingly by taking into consideration the impact of these loads. The construction and proper maintenance of safe, stable slopes are the responsibility of the contractor. Such safe, stable slopes must be based on actual construction conditions and subsurface conditions encountered during excavation.

5.5.5 Shored Excavation and Shoring Support

The planned station excavation will require shoring due to the proximity of the station to existing buildings and roads, and limited construction space along the alignment. Various shoring systems may be appropriate. These include various temporary walls such as sheet pile, soldier pile and lagging, precast, and slurry walls supported by tiebacks, anchors and/or internal bracing struts. The most appropriate shoring system must consider subsurface conditions, excavation geometry, the dewatering scheme (if applicable), construction procedures, characteristics of nearby buildings, and local experience. Based on local practice in the Los Angeles area in subsurface geotechnical conditions similar to those anticipated at the station area, soldier piles with tiebacks and/or internal bracing (struts and wales), and timber, shotcrete or pre-cast reinforced concrete lagging are the most likely shoring systems. The use of slurry wall construction for support of excavations in lieu of soldier piles and lagging would be relatively expensive and may not be practical. Driving of sheet piles may not be feasible due to local presence of gravel and cobbles in the alluvium.

5.5.5.1 Lateral Pressure

Lateral earth pressure on the shoring system depends on the type of shoring system, construction procedures, and subsurface and groundwater conditions. Based on the available laboratory test results, anticipated shoring system, and construction procedures, as well as previously stated engineering assumptions, lateral earth pressures on the soldier pile and lagging walls for the following cases are shown in Figures 5-1 through 5-3:

- Braced sheeting above the excavation
- Cantilevered sheeting above the excavation
- Surcharges from a sloped excavation, existing buildings, construction loads, and earthquake-induced loads
- Active and passive earth pressures on soldier piles below the excavation.

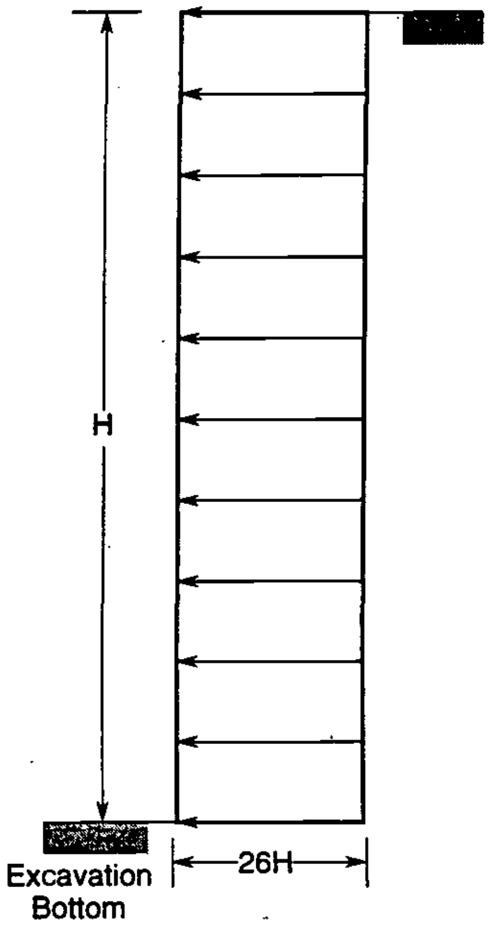
The lateral loading diagrams presented in Figures 5-1 through 5-3 are for use in the design of soldier pile details, tiebacks, or an internal bracing system.

Lateral earth pressures on lagging depend on a number of factors, including subsurface conditions and engineering properties, spacing between soldier piles, and dimensions and configuration of the excavation. For sizing purposes, the lateral earth pressure on lagging can be taken as 50 percent of that recommended for the temporary shoring walls (Figures 5-1 through 5-3), to account for soil arching effects. However, the soldier piles must be designed to carry the full lateral loading.

It is understood that design and construction of an appropriate shoring system to reduce ground loss and disturbance of the site and adjacent buildings, and to provide a safe worksite is the responsibility of the Contractor. Various design considerations are described in the following sections for use as general guidelines.

5.5.5.2 Soldier Piles and Lagging

The soldier piles and lagging walls should be designed to adequately resist lateral and vertical loads imposed by the excavation, existing structures, construction loading, environmental loading (such as earthquake loading), and the shoring system itself. Design considerations, which include pile sizing, embedment depth, spacing, installation, and lagging provisions, should be in compliance with appropriate building codes and city requirements.



Lateral Earth Pressure
On Braced Sheeting

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Metro Red Line

Lateral Earth Pressure on Temporary Excavation Support

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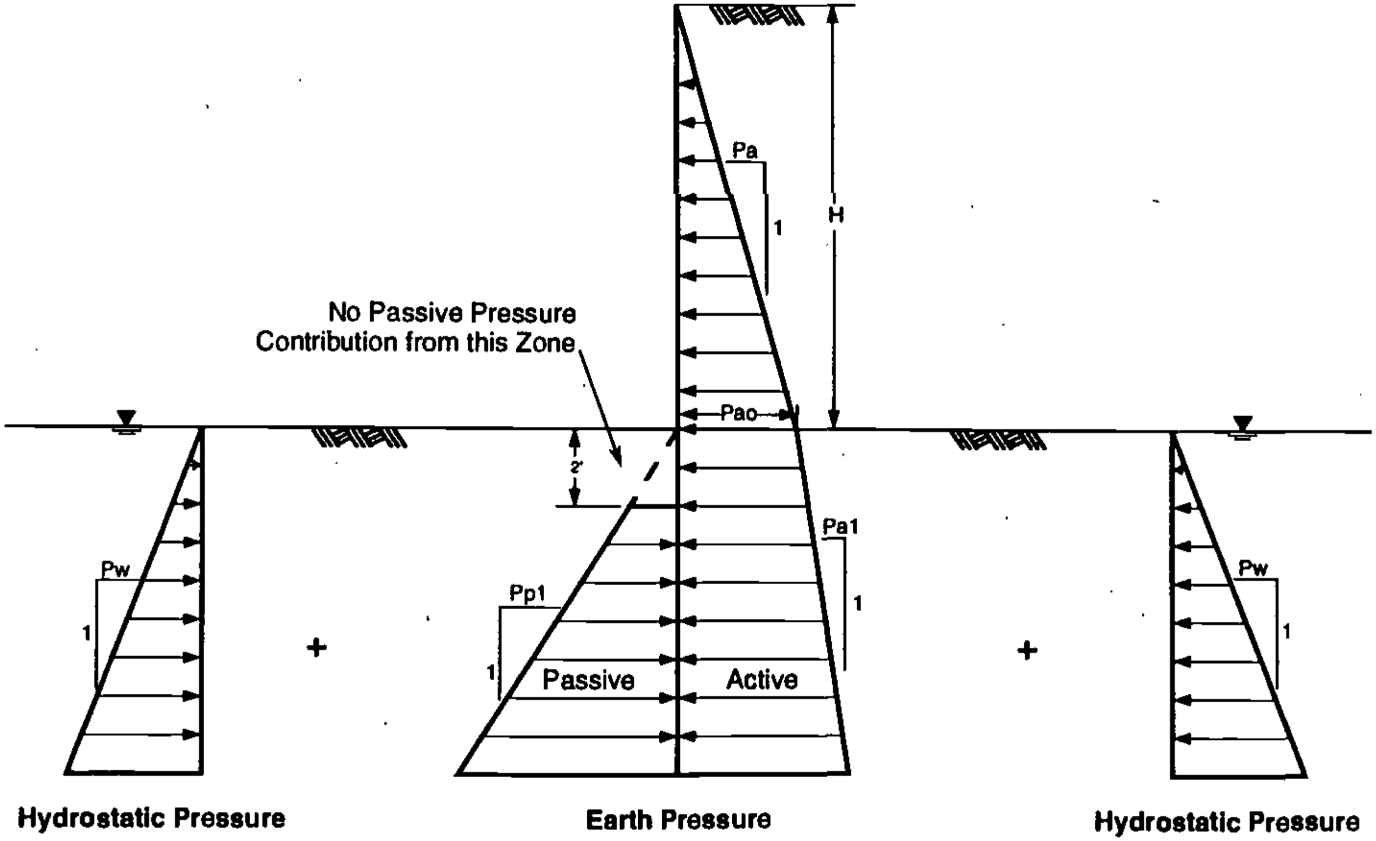
Figure 5-1



Pa = 38 psf/ft
Pao = 38H psf
Pa1 = 20 psf/ft
Pp1 = 200 psf/ft
Pw = 62.4 psf/ft

Notes:
1. Groundwater Level is Assumed at Station Invert.
2. The Penetration Obtained by Using This Pressure Diagram Should be Increased by 20%.

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Lateral Earth Pressure on Temporary Excavation Support

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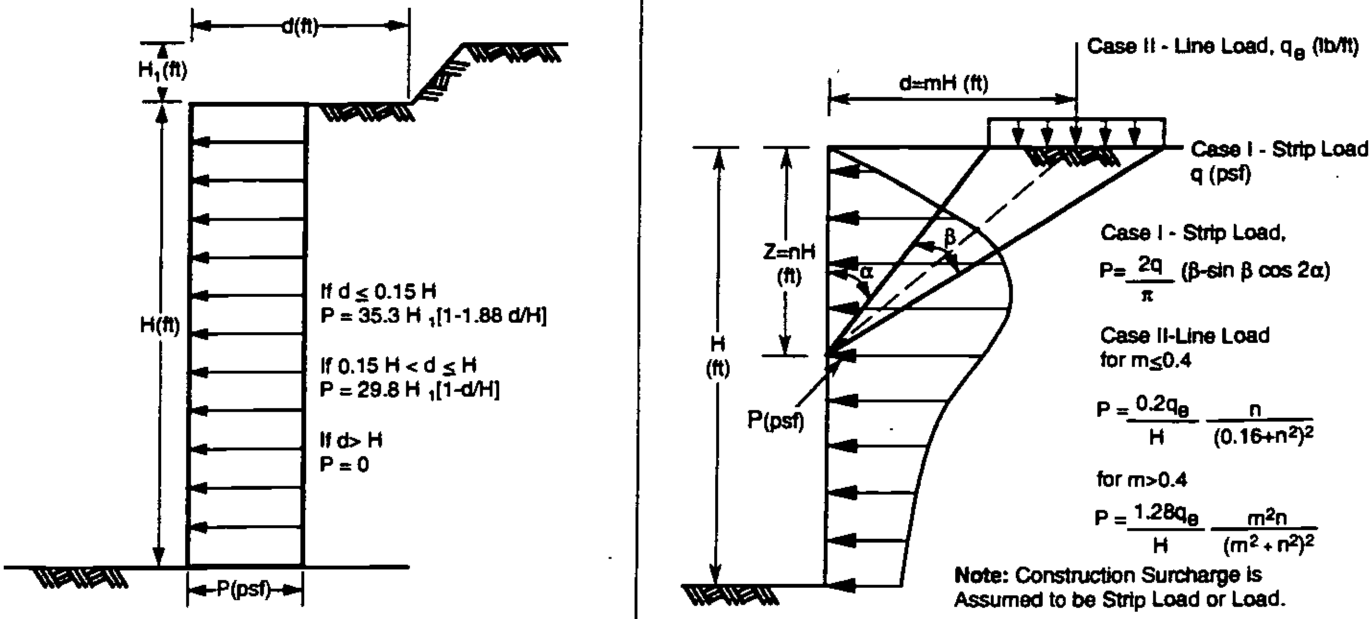
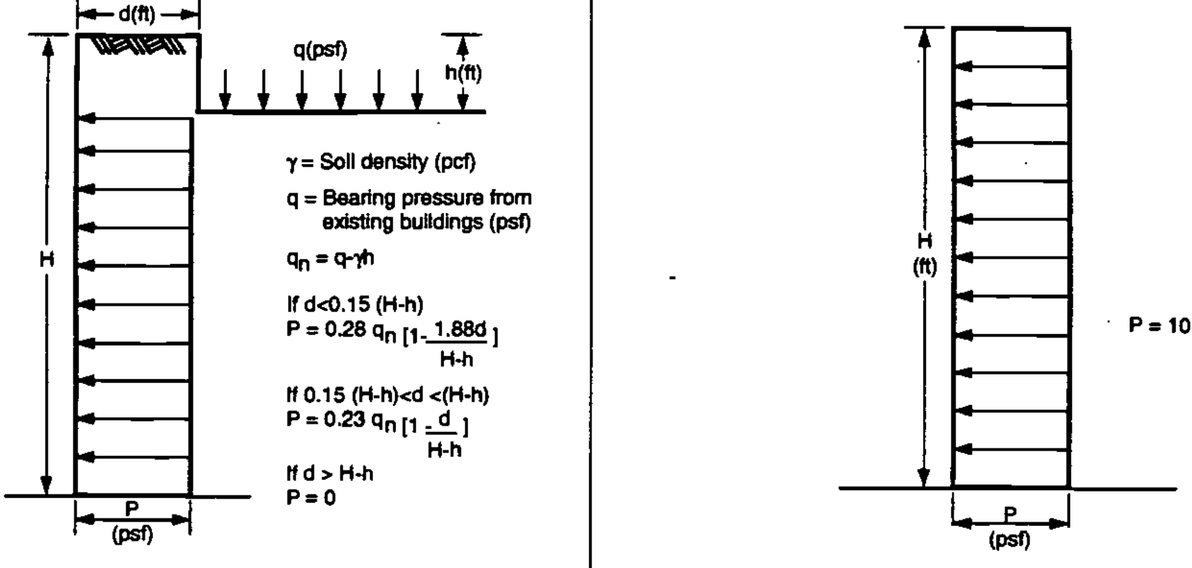
Figure 5-2



(a) Sloped Excavation Surcharge

(b) Construction Surcharge



(c) Building Surcharge

(d) Earthquake Surcharge

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Additional Lateral Earth Pressure on Braced or Cantilevered Sheeting

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Figure 5-3

Pile sizing includes a proper determination of pile size (diameter or cross section) and type (stiffness) so that stresses in the piles are within allowable limits. All anticipated lateral and vertical loads as well as calculated loads from tiebacks or internal bracing should be applied in calculating the pile stresses. The calculated stresses in the pile can be reduced by 20 percent to account for arching effects due to pile flexibility.

The soldier piles should be sufficiently embedded below the excavation depth to safely resist anticipated lateral and vertical loads. The passive resistance should exceed the imposed lateral loads (active resistance minus the resistance from tiebacks or internal bracing) with a reasonable safety factor. The effective excavation width that each pile can support should be taken as 1-1/2 times the soldier pile diameter or half of the pile spacing, whichever is less. It should be noted that piles may undergo some movement before mobilizing the anticipated capacities. It is recommended that pile load tests be performed to verify estimated capacities.

Optimal pile spacing depends on a number of factors including subsurface conditions and engineering properties of subsurface materials, pile sizing, construction procedures and cost. Considering the need for lagging to alleviate soil raveling and minimize ground loss, a pile spacing of 8 feet or less would be reasonable.

Local noise abatement requirements, and the local presence of cobbles in alluvium generally preclude the use of conventional impact driving to install soldier piles. Thus, the soldier piles, if used, would likely be installed in predrilled holes. Slurry or casing will be required to handle potential caving conditions within the granular alluvium.

Lagging between soldier piles will be required. Timber, shotcrete, or pre-cast reinforced concrete members can be used as lagging between soldier piles. The contractor will be responsible for designing, installing and maintaining a suitable lagging system subject to review and acceptance by the owner or his representative. The Contractor must control the temporary height of exposed soil prior to lagging placement to eliminate raveling and ground loss problems. If shotcrete lagging is used, the Contractor must have effective drainage provisions in the lagging design to prevent accumulation of groundwater at the shotcrete-soil interface.

5.5.5.3 Tieback Anchors

Installing tiebacks in the site area will require permission from the owners of adjacent buildings and avoidance of below-grade obstructions such as basements or foundations of adjacent buildings. Many types of tieback anchors exist, including shaft anchors, belled anchors, anchor blocks, and high-pressure grout anchors. In general, the allowable capacity of the tieback anchor should be determined in the field based on load tests. The following recommendations should be considered in the design and installation of tieback anchors.

Effective friction of a tieback anchor can develop only beyond a no-load zone. Our recommendations for the no-load zones on a straight shaft anchor, considering depth of excavation and potential wedge failure planes, are shown in Figure 5-4. The anchors may be installed at inclinations ranging between 20 degrees and 50 degrees below the horizontal. Potential caving conditions are possible in the granular alluvium, so the contractor should use appropriate measures to prevent caving and minimize ground loss. Each tieback anchor must be load tested to 150 percent of the design load in accordance with standard acceptance criteria (FHWA-DP-68-IR, November 1984) or local site-specific experience of the Contractor. The load in the tiebacks must be locked off at 100 percent of the design load, and monitored periodically.

5.5.5.4 Internal Bracing

If braced shoring systems are employed, the strut loads should be determined using the full load diagrams shown in Figures 5-1 through 5-3. The vertical spacing between struts must be appropriately designed to mitigate ground movement potential. All struts must be preloaded to eliminate slack and minimize ground movement. A preload of 25 percent of the design load is recommended. However, it should be noted that a preload of 25 percent of the design load may induce undue loading on basements, if any, of adjacent buildings. This possibility must be analyzed on a case-by-case basis.

Procedures to compensate for the effects of temperature changes on the strut loads should be developed and implemented so that proper strut load levels can be monitored and maintained during construction.

5.5.5.5 Combined Shoring System

If a shoring system which combines external tiebacks and internal bracing or struts and wales is selected for excavation support, the contractor's design must account for the variation in stiffness and deflection characteristics of the support elements which may induce substantially different load distributions.

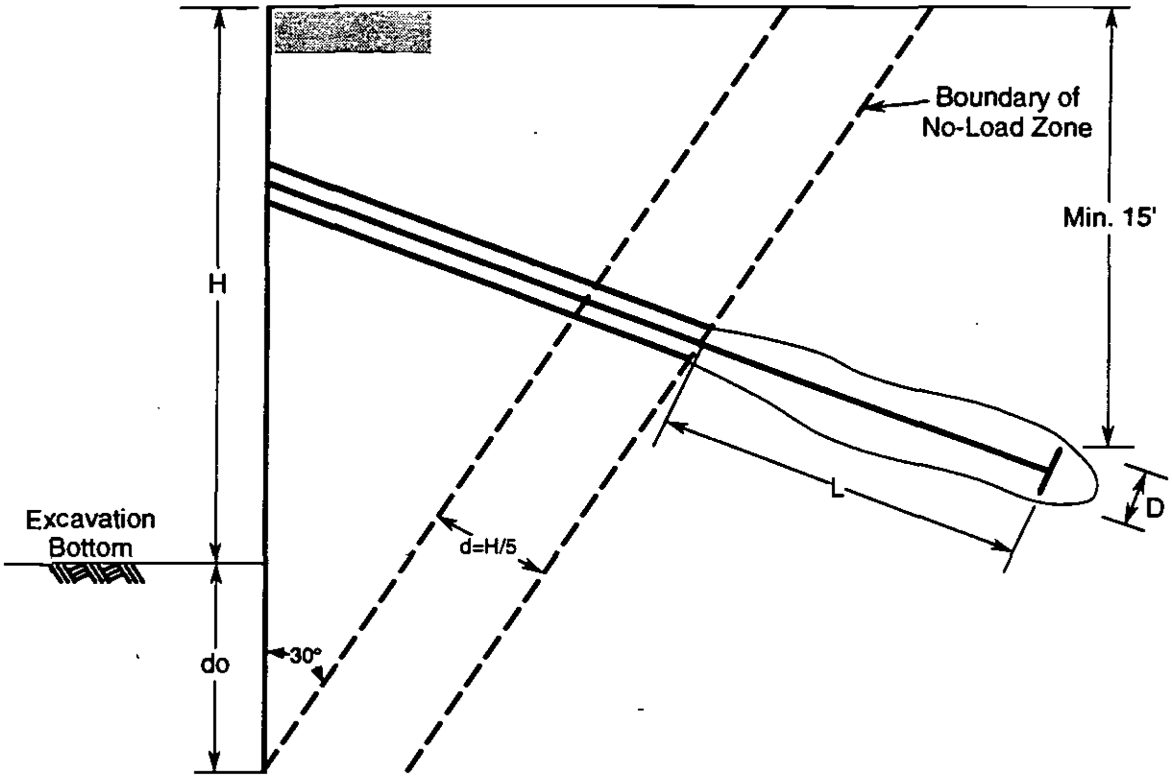
5.5.5.6 Ground Movement

Station excavations will experience ground movements in terms of wall movement and ground heave. The magnitude of wall movement depends on many factors, including the design and construction of shoring systems, construction schedule, specifications, and subsurface conditions. In general, for a well-designed and constructed shoring system, the maximum horizontal wall deflection may be about 0.1 percent to 0.2 percent of the excavation depth. For the First/Lorena Station the corresponding maximum horizontal wall movement for a well designed and constructed shoring system may be about 1 inch to 2 inches. For a shoring system with tiebacks, this maximum horizontal deflection will likely occur near the surface, and decrease with depth. For a well-designed and constructed internally braced system with struts and wales, the maximum horizontal deflection will probably occur near the bottom of the excavation and decrease to about 0.2 inch to 0.5 inch near the surface. It is estimated that, for a well-designed and constructed shoring system, a maximum vertical settlement of about 0.5 inch to 1 inch will probably occur behind the wall to about 25 feet to 50 feet from the wall, and will decrease as the distance from the maximum settlement location increases.

5.5.6 Protection of Adjacent Buildings/Structures and Underpinning

No building structures are directly above the proposed station. Within the zone of influence of the station excavation and as close as 15 to 20 feet away, there exists a number of 1-story residential and commercial buildings and a 3-story shopping plaza (3425 E. First Street).

Station excavation and construction may cause some ground settlement or angular distortion. It is the Contractor's responsibility to carry out the excavation with timely placement of an adequate support system to reduce disturbance and avoid structural damage to the adjacent buildings.



L= Bond length of anchor beyond no-load zone.
do= Depth of penetration required for stability.
do, d, D, H, and L are in feet.

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Anchor Locations for Tieback Walls

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Figure 5-4

In general, the need for underpinning these adjacent buildings will depend on whether their foundations are adequate and whether the buildings can satisfactorily resist anticipated settlement due to excavation related deformation. Figure 5-5 presents rough guidelines to estimate whether adjacent buildings require underpinning. The final need for underpinning should be evaluated by a qualified structural engineer on a case-by case basis. These adjacent buildings, whether underpinned or not, should be monitored for settlement and lateral movement on a regular basis during station construction. If the monitored settlements indicate the potential for excessive settlements beyond the maximum allowable limits preset by the engineers, excavation work should be suspended temporarily and immediate remedial measures taken.

5.5.7 Excavation Heave

The excavation depth of the First/Lorena Station is approximately to 60 to 78 feet. This would mean a maximum stress relief of about 8,000 to 10,000 psf at the bottom of the excavation, resulting in bottom heave due to elastic and consolidation rebounds. Based on the subsurface conditions and results of consolidation tests on subsurface soils (Appendix B), it is estimated that the heave due to elastic rebound will be about 1 to 2 inches, and the consolidation rebound will be about 1 to 2 inches. Elastic rebound will take place during the excavation. The bulk of the consolidation rebound is anticipated to take place within a 1 to 2 month period following the excavation. Rupture of the excavation bottom due to excessive heave is not likely.

5.5.8 Foundation Support

5.5.8.1 Main Station Structure

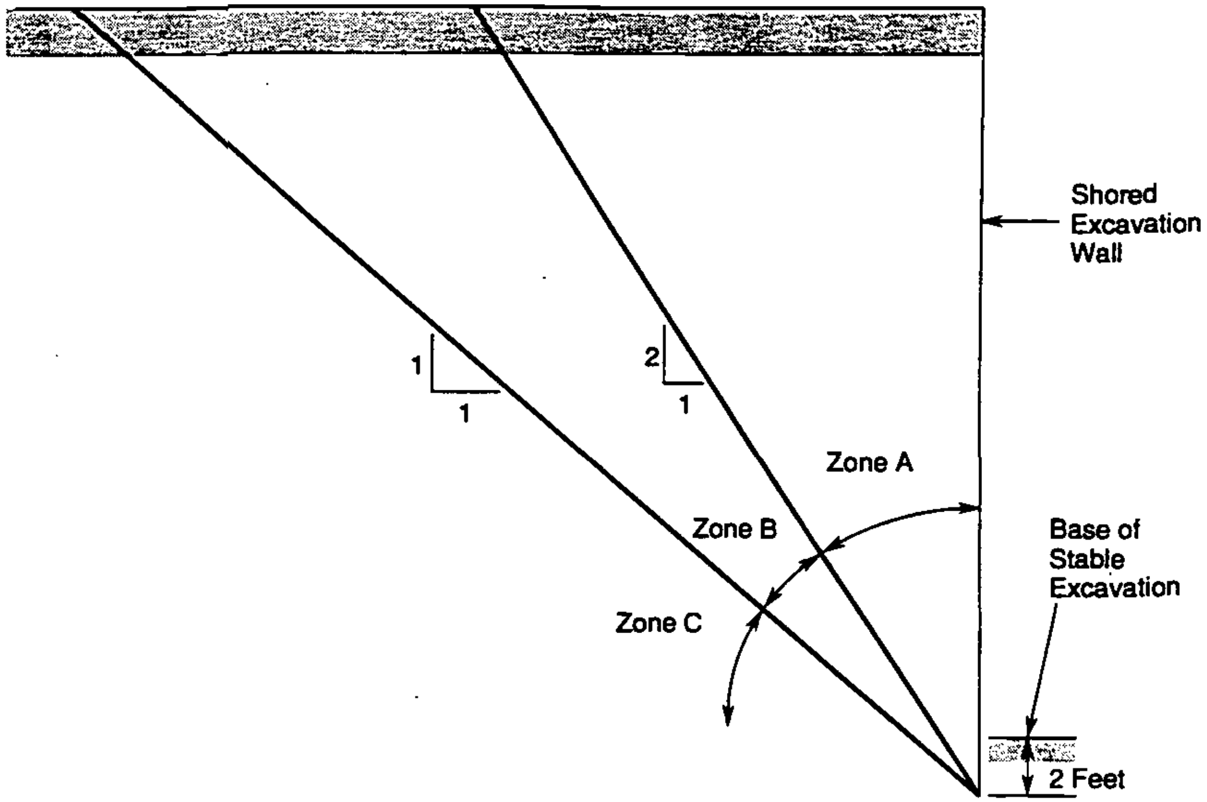
The overall station will be designed and constructed as a relatively rigid box, and the main station housing the rail facilities will be supported on wide, thick slabs that will function as relatively rigid mat foundations. The design of mat foundations is generally governed by settlement considerations rather than by bearing capacity. Available information indicates that the average bearing pressure on the mat foundations from the station and backfill would be on the order 5,000 psf, which is less than the overburden pressure removed by the excavation. Therefore, this anticipated station load can be adequately supported on the alluvium underlying the station mat foundation. An allowable bearing capacity of 5,000 psf may be used for design of mat foundations provided the settlements estimated below are taken into consideration in design.

It is estimated that elastic settlement due to the station load of 5,000 psf will be on the order of about 1 inch to 1½ inch. This settlement would take place during construction. In addition to the immediate elastic settlements, 1 to 1½ inches of consolidation settlement is estimated under the design load of 5,000 psf. Analyses indicates that about 90 percent of the consolidation settlement due to the station load will take place over a period of about 1 to 2 months following construction. Some differential settlement of the structure should be expected due to the nonuniformity of the soils at the site. It is estimated that the differential settlement over the width of the station will be on the order of 1 inch.

If the groundwater levels in the station area rise to the assumed design levels (i.e., about 50 feet above the station invert), the station mat foundation will experience a vertical stress relief of about 3,100 psf, resulting in heave due to elastic and consolidation rebounds. It is estimated that the corresponding elastic rebound will be on the order of about 1/2 inch, and the corresponding consolidation rebound will be on the order of ½ inch. In this case, differential heave over the width of the station is estimated to be about ½ inch.

5.5.8.2 Surface and Near Surface Structures

Near surface structures can be supported on conventional spread footings founded on properly compacted fill or on competent granular alluvium in the site area. All spread footings should be a minimum of 2 feet wide and at least 2 feet below the lowest adjacent grade. Subgrade for shallow footings should be overexcavated to a depth equal to the minimum footing width and replaced with properly compacted fill. Overexcavation must extend a minimum distance equal to half the width or 2 feet, whichever is larger, beyond the perimeter of the footing. Fill must be compacted to a minimum dry density of 95 percent of maximum dry density as determined by the ASTM procedure D-1557. Allowable bearing capacities and estimated total settlement in terms of footing width and bearing pressures for shallow spread footings are graphically presented in Figure 5-6. Some differential settlement between adjacent footings should be anticipated. This differential settlement between adjacent footings is estimated to be one-half of the average total settlements or the differences in total settlement, whichever is larger.



Notes: The following underpinning guidelines are for structure foundations in Zones A, B, and C.

Zone A: Foundations within this zone generally require underpinning

Zone B: Sensitive structure foundations within this zone generally require underpinning

Zone C: Underpinning may not be required

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General Guidelines for Underpinning

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Figure 5-5

In the absence of specific information on loads, dimensions and locations of structures, Figure 5-6 is provided as general guidelines for foundation support needs. Structure-specific foundation recommendations should be provided when such information becomes available.

Figure 5-6 can be used for vertical, concentric loading. Bearing capacity will be reduced under eccentric and/or inclined (combined vertical and horizontal loads) loading conditions. It is recommended that a site-specific and loading specific study be performed for such cases once design loading conditions are known.

Lateral resistance of the footing can be assumed to be provided by passive earth pressure on the side of the footing and friction resistance between the footing and soil. An allowable passive pressure of 250 psf per foot, and an allowable frictional coefficient of 0.4 are appropriate for lateral resistance considerations.

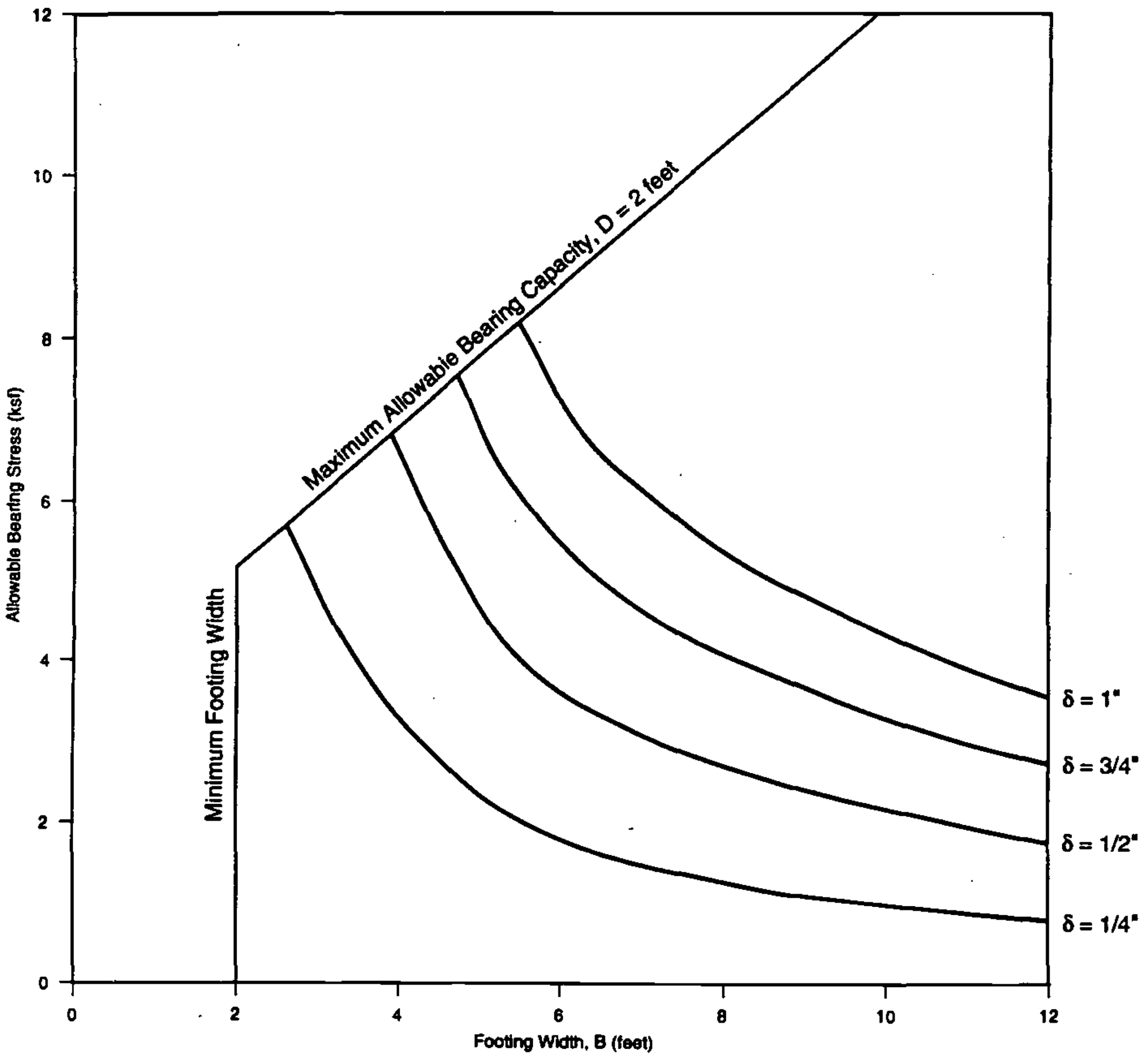
5.5.9 Geotechnical Input for Station Design

5.5.9.1 Geotechnical Parameters

Table 5-1 summarizes recommended geotechnical design parameters for station design based on available data. These design parameters include lateral earth pressure coefficients, unit weight of soil, shear strength, long term maximum groundwater level, modulus of subgrade reaction for shallow foundations and seismic coefficients.

5.5.9.2 Soil Spring Constants for Main Station Structures

Estimated ranges and best estimates of static and dynamic soil spring constants for design of the main station structure are presented in Figure 5-7. It is recommended that the full range of estimated parameters be considered in the structural design to account for the potential variability of the subsurface materials in the station area. An upper bound/lower bound approach is recommended for the structural response analysis of the station box structure. The lower bound values of soil spring constants should be used for evaluation of settlements while the upper bound values should be used for evaluation stresses in the structure.



Explanation:

δ = Immediate Settlement for a Square Footing

D = Footing Embedment Below Adjacent Grade

Note: Factor of safety of 3 is included.

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Allowable Bearing Capacity for Spread Footing in Compacted Granular Fill (Compacted to 95% of Maximum Dry Density)

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Figure 5-6

TABLE 5-1. DESIGN VALUES OF GEOTECHNICAL PARAMETERS FOR UNDERGROUND STRUCTURES (FIRST/LORENA STATION)

Parameter	Design Value	
Geologic Unit	Fine-Grained Alluvium	Coarse-Grained Alluvium
Unit Weight of Soil (pcf)	130	
Void Ratio	0.57	0.48
Cohesion (psf)	600	0
Angle of Internal Friction of Soil (degrees)	28	35
Soil Pressure Coefficient At Rest (Ko) Active (Ka)	 0.45 0.29	
Groundwater Level[1]	50 feet above station invert	
For Support of Mat Foundation: Coefficient of subgrade reaction K_1[2] (tons/ft^3) Footing of Width B, K_b[3] (tons/ft^3)	 70 - 200 $K_b = \frac{K_1}{I} \left(\frac{B+1}{2B}\right)^2$	
Seismic Coefficient ODE[4] Horizontal (Kh) Vertical (Kv) MDE[4] Horizontal (Kh) Vertical (Kv)	 0.3g 0.2g 0.6g 0.4g	

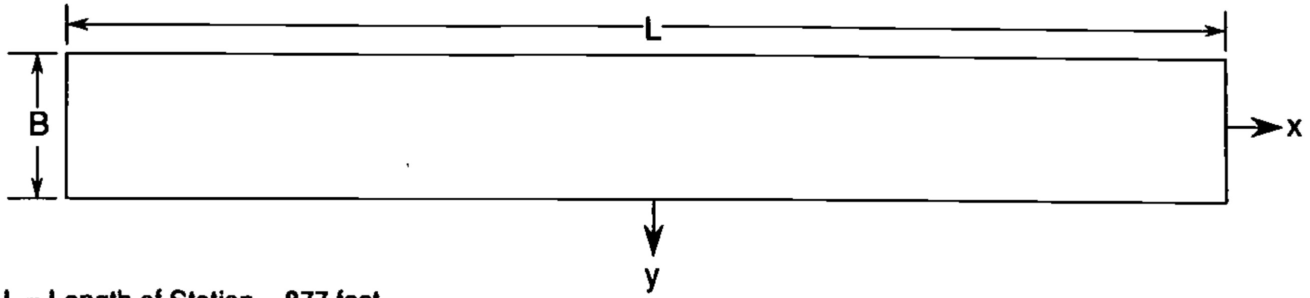
Notes:

(1) Assumed maximum groundwater level during the design life of the station. Current groundwater levels are below station invert (Elevation 230 feet).

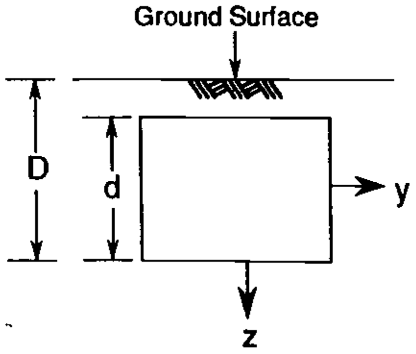
(2) K_1 = Coefficient of subgrade reaction for a 1' x 1' plate.

(3) K_b = Coefficient of subgrade reaction for a mat foundation of width B and length mB. I=1.0 for m=1, I = 1.12 for m = 2, I = 1.6 for m = 5, I = 2 for m $\geq$10.
An upper bound/lower bound approach is recommended for the structural response analysis of the station structure. The lower bound values of modulus of subgrade reaction should be used for evaluation of settlements while the upper bound values should be used for evaluating stresses in the structure.

(4) Operating Design Earthquake (ODE) and Maximum Design Earthquake (MDE) are based on Seismic Investigation and Design Criteria for Metro Rail Project prepared by Converse Consultants, Earth Sciences Associates and Geo/Resource Consultants (1983).



Plan View



Cross Section

L = Length of Station = 877 feet
B = Width of Station = 60 feet
D = Depth of Station Invert = 59.5 to 78 feet
d = Height of Station Wall = 51 feet
k_x, k_y, k_z, = Soil Spring Constants in x, y and z directions, respectively.

Loading Condition	Applicable Condition	Shear Modulus G, 10^6 psf	Poisson's Ratio	Estimated Soil Spring Constant (Stiffness) 10^6 lb/ft		
				k_z	k_y	k_x
Static (high strain)	Estimated Range	0.5 to 1.6	0.3 to 0.4	920 to 2900	1070 to 3400	890 to 2800
	Best Estimate	0.7	0.35	1280	1500	1240
Dynamic* (low strain)	Estimated Range	5 to 16	0.3 to 0.4	9600 to 30500	10900 to 35000	9100 to 29000
	Best Estimate	7	0.35	13400	15300	12700

Note:

* Actual dynamic stiffness is frequency-dependent. Approximate average numbers (over a wide frequency range) are provided for simplification purposes.

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Estimated Soil Spring Constant Values (First/Lorena Station)

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Figure 5-7

5.5.9.3 Lateral Earth Pressures on Permanent Structures

We understand that the station will be designed and constructed as a relatively rigid, water-tight box. The recommended lateral earth pressures including hydrostatic pressure and earthquake loading are shown in Figure 5-8. The following are noted:

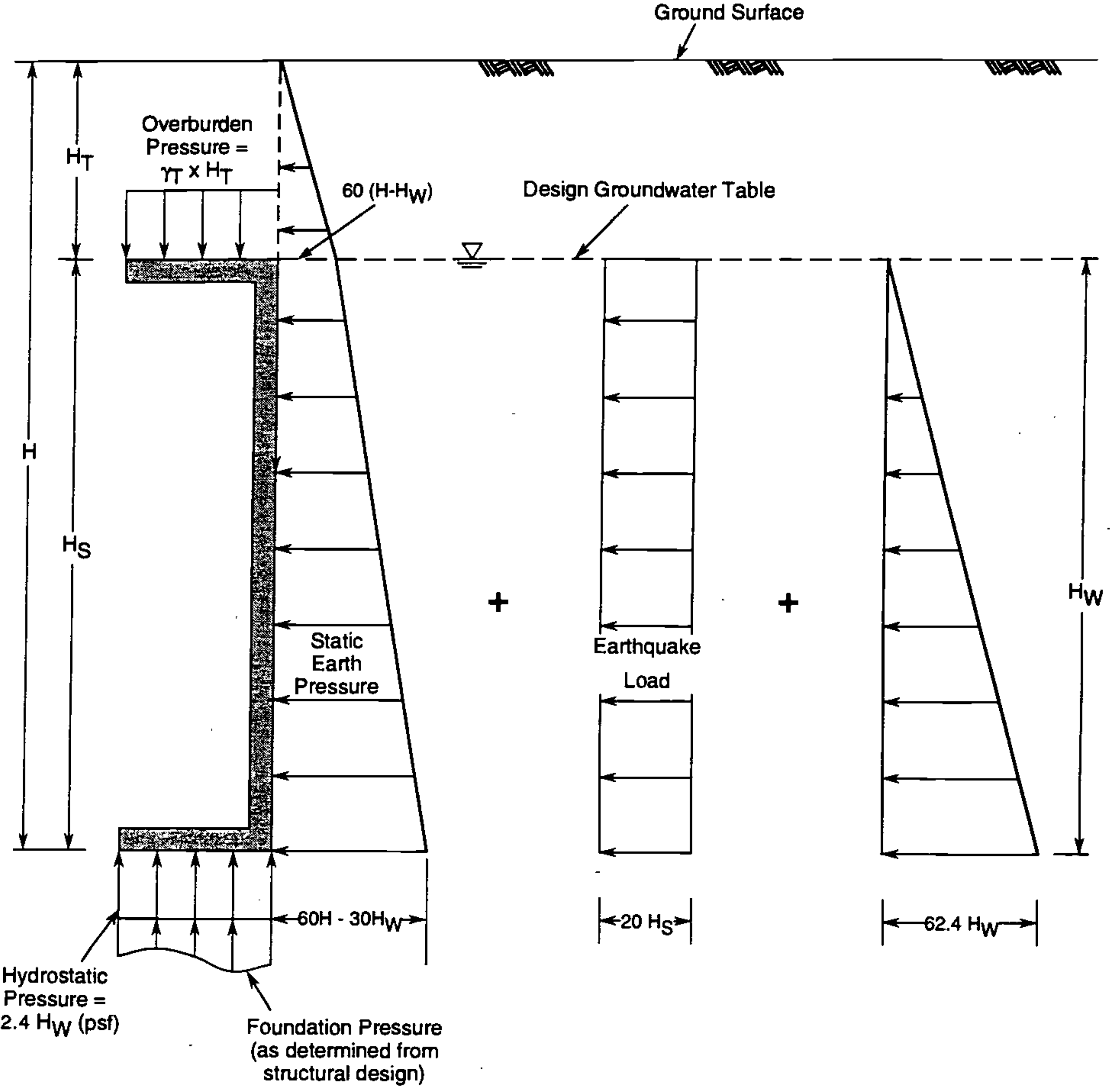
- Assumptions for groundwater levels during the design life of the station and maximum design earthquake (MDE) described in Sections 5.3 and 5.4 were utilized in developing Figure 5-8.

- The Mononobe-Okabe procedure was utilized for developing earthquake induced lateral earth pressures. This procedure assumes that the wall yields or rigidly moves sufficiently for active conditions to develop during earthquake loading. In developing the earthquake-induced earth pressure recommendations, the potential effects of vertical ground acceleration were ignored to avoid over-conservatism.

- Potential surcharge effects from adjacent existing buildings which are not underpinned should be considered in the wall pressure diagrams. Lateral surcharge loads on walls can be calculated in accordance with the recommendation shown in Figure 5-3c. Vertical loads from anticipated traffic and other live loads should be determined and added to the roof loading.

5.5.10 Liquefaction Potential

Liquefaction potential of subsurface soils surrounding the station was evaluated using the procedures by Seed and Idriss (1982), Seed et al (1984) and recent work done by Fear and McRoberts (1995). The assumed highest groundwater levels described in Section 5.3 were utilized in the analysis to account for potential groundwater fluctuations. The results of our analyses indicate that liquefaction of the site soils is unlikely under the assumed maximum design earthquake.



Notes:

(1) All pressure in psf

(2) Station invert elevation is at about 232 feet.

(3) Design groundwater level is at about elevation 280 feet (about 10 to 30 feet below ground surface).

(4) H, H_T, H_S, and H_W are in feet.

(5) γ_T - Total density of soils.

(6) Drawing is schematic.

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Earth Pressure on Below-Grade Permanent Structures

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Figure 5-8

5.5.11 Earthwork

Based on the subsurface conditions encountered in the investigation, the station excavation can be accomplished by conventional and readily available excavation technology. Earthwork and site preparation activities are expected to consist of an excavation for subterranean structures; subgrade preparation for the station floor; foundation preparation for near-surface structures; excavation for utility trenches; subgrade preparation for pavements; and backfill placement for subterranean walls, footings and utility trenches. Major excavations will need to be provided with temporary shoring according to the recommendations presented in Section 5.5.5. Other minor excavations, subgrade preparation and backfill placement must be done in accordance with the guidelines presented in Appendix C. All work must be in compliance with applicable city (Los Angeles), state (California), and federal (Occupational Safety and Health Act) requirements.

In general, the mat foundations must be underlain by a minimum 2-foot thick dense layer of granular material with an in situ density of at least 95 percent of the maximum dry density as per ASTM D1557. In granular alluvium this may be achieved by proof rolling the excavated subgrade with a heavy vibratory roller, or by overexcavating and compacting the 2-foot thick zone below the foundation in 8-inch thick lifts. In fine-grained alluvium, the subgrade must be overexcavated a minimum of 2 feet below design grade and replaced with granular material compacted to required specifications. If the mat is directly placed on the native materials, rock fragments larger than 6 inches are not allowed in the exposed subgrade. The compacted fill blanket should extend a minimum of 5 feet beyond the foundation perimeter.

Materials excavated from non-contaminated granular alluvium (sand, silty sand, gravelly sand, sandy gravel, and gravel) could be stockpiled to be reused as backfill material. Excavated fine-grained alluvium is not suitable as backfill material. If there is insufficient material available on site for backfill, imported granular material could be used for fill subject to the approval of the geotechnical engineer.

5.6 TAIL TRACK TUNNELS

5.6.1 Subsurface Materials in Tunnel Zone

Based on current plans and profiles and measured groundwater levels (Plate 1), the Tail Track Tunnels will be entirely in alluvium and above groundwater. It is anticipated that soft ground/soft rock tunneling methods will be generally applicable.

Tunneling in alluvium along the alignment will encounter raveling and running conditions because of the predominantly granular nature of the alluvium. Slow raveling conditions can be anticipated in silty sand and clayey sand. Fast raveling conditions and running/flowing conditions can be anticipated in cobbles, gravels, gravelly sand, and poorly graded sands above or below groundwater, or well-graded sands below groundwater. Fast raveling and running/flowing conditions are anticipated over a major portion of the tunnel within alluvium.

It is noted that a significant portion of the Tail Track Tunnels is under existing commercial and residential structures. For the Tail Track Tunnels, ground stability and stringent settlement controls are important to help reduce construction impact to the public, especially in the portion of tunnel below the existing commercial and residential structures.

5.6.2 Groundwater Control

The most recent observed groundwater level data indicate that the groundwater table in the Tail Track Tunnel area is about at or below the planned tunnel invert. No preconstruction dewatering is anticipated provided groundwater levels remain at or below the currently measured levels. The potential for significant fluctuation of the groundwater levels in the area dictates the need for continuing groundwater level monitoring before and during tunnel construction to verify groundwater levels and to re-evaluate the dewatering needs, as appropriate. As stated previously, local perched groundwater is possible since fine-grained soils are locally present in the tunnel zones. It is anticipated that small local inflows can be handled by sumps and pumps.

5.6.3 Liquefaction Potential

Liquefaction potential of the subsurface soils surrounding the tunnels was evaluated in accordance with the methodology described in Section 5.5.10. The assumed highest groundwater levels described in Section 5.3 were utilized in the analysis to account for the effects of potential groundwater fluctuations on the liquefaction potential of subsurface materials. The results indicate that liquefaction of the site soils under the assumed maximum design earthquake is unlikely.

5.7 SOIL AND GROUNDWATER CONTAMINATION SUMMARY

5.7.1 Groundwater Contamination

The proposed areas of tunnel and station construction may encounter local perched groundwater conditions. Some of these perched water inflows may be contaminated with hydrocarbons due to the proximity of the project area to the existing Boyle Heights oil field. No evidence of floating hydrocarbon product was detected during development and groundwater level monitoring of Monitoring Wells SD-15 and DD-27S.

5.7.2 Soil Contamination

The majority of the excavated soils is expected to be clean and should not require special handling or disposal. Again, due to the proximity to the Boyle Heights oil field, some of the excavated soils from the station and tunnel areas may locally contain hydrocarbon contamination and require treatment and disposal at approved facilities.

5.8 GASSY CONDITIONS

The proximity of the station and tunnel areas to the Boyle Heights oil field indicates that the potential for encountering oil field gases (methane, hydrogen sulfide, etc.) during construction exists.

Proper ventilation and monitoring of methane, hydrogen sulfide and other toxic/explosive gases will be necessary to satisfy the U.S. and California Occupational Safety and Health Administration (OSHA) requirements and to provide a safe working environment.

5.9 ABANDONED OIL WELLS

Due to the proximity of the project area to the Boyle Heights oil field, there is a potential for the presence of undocumented cased or uncased abandoned oil wells within the tunnel envelope and station excavation limits. In addition to requiring considerable time to move the casings, such abandoned wells, if encountered, may contain large quantities of water or even oil under pressure. The abandoned wells may also contain residual accumulations of hydrogen sulfide, methane or other toxic/explosive gases.

5.10 CORROSIVE SOILS

Results of sulfate content tests indicate that the soils in the project area are not corrosive to concrete. Results of available electrical resistivity tests from the preliminary geotechnical investigations (GeoTransit Consultants, 1994a) and the current investigation (Table 3-6) indicate that most of the subsurface soils are corrosive (2000 to 5000 ohm-cm) to very corrosive (less than 2,000 ohm-cm) to metals.

5.11 FAULT CROSSING

No known active or inactive faults cross or project toward the project area.

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7.0 LIMITATIONS

The conclusions and professional opinions presented in this report were developed by GeoTransit Consultants for Engineering Management Consultant (EMC). GeoTransit Consultants makes no warranty, either expressed or implied, as to its findings, opinions, recommendations, specifications, or professional advice except that these were promulgated after being prepared in accordance with generally accepted standards of care and diligence normally practiced by recognized consulting firms performing services of a similar nature.

Subsurface conditions are, by their nature, uncertain and may vary from those tested in the laboratory, documented in historical documents or encountered at the locations where visual inspections, borings, soundings, test pits, surveys, or other explorations were made by GeoTransit Consultants. The data, interpretations, and recommendations of GeoTransit Consultants are based solely on such information or from information obtained by others in the area covered by this report, and or observations from borings in the area covered by this report.

The data and conclusions contained herein should be considered to relate only to the specific project and location discussed herein. GeoTransit Consultants is not responsible for any conclusions that may be made from these data by others unless we have been given an opportunity to review such conclusions and concur in writing. This report has not been prepared for use by parties other than EMC or its consultants. It may not contain sufficient information for the purposes of other uses.

If any changes are made in the project as outlined in this report, the conclusions contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified and approved in writing by GeoTransit Consultants.

Respectfully submitted,

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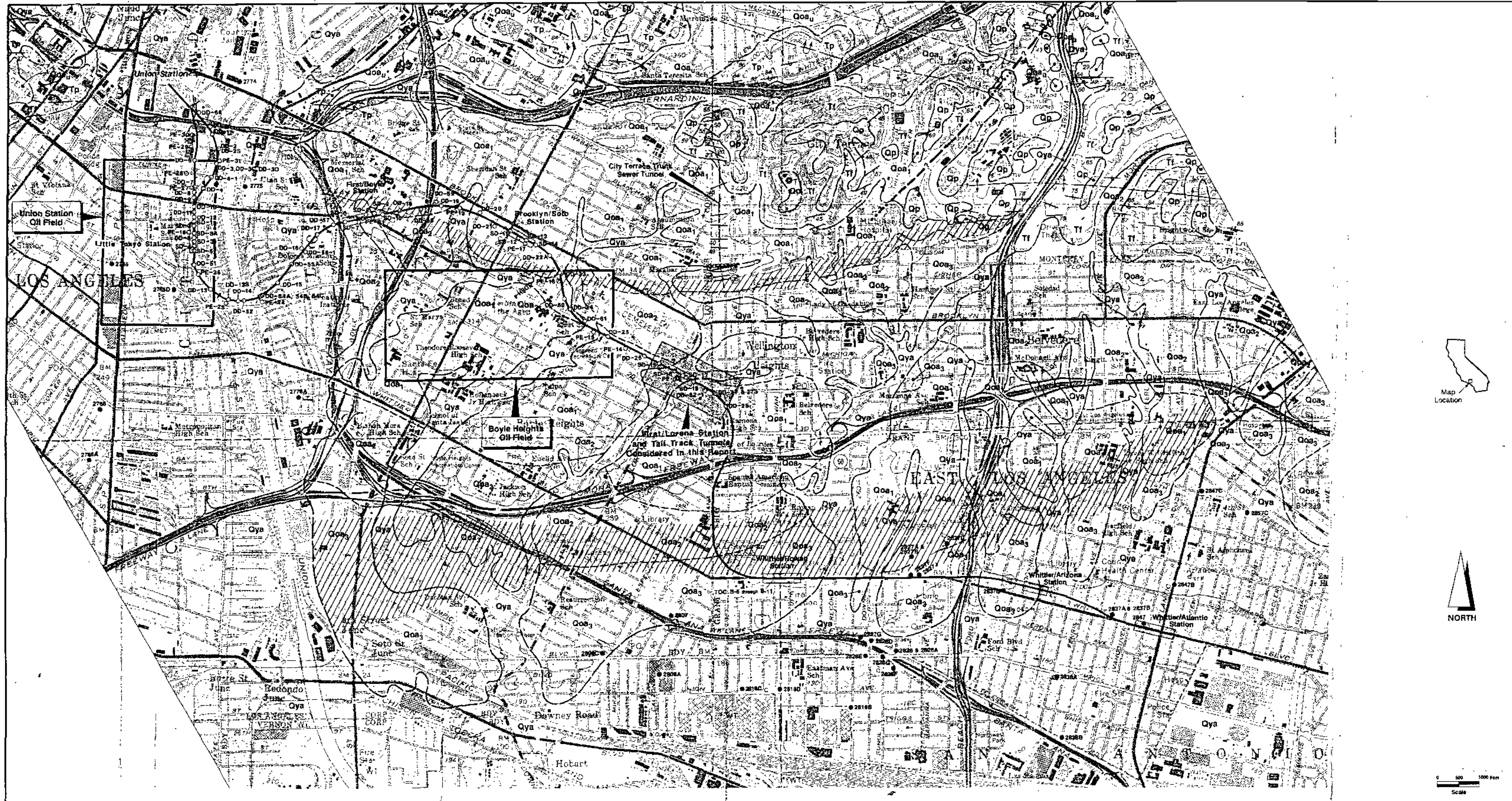

EXPLANATION

GEOLOGIC UNITS

Qya
YOUNGER ALLUVIUM
Q_4 and Q_5 of Bullard and Lettis (1993)

Qalo of Lamar (1970)
Qoa$_u$ | **Qoa$_3$** | **Qoa$_2$** | **Qoa$_1$**
OLDER ALLUVIUM
Q_1, Q_2 and Q_3 of Bullard and Lettis (1993)

Qp
OLDER GRAVEL

Tf
FERNANDO FORMATION

Tp
PUENTE FORMATION

SYMBOLS

▨ Fault/Escarpment

TUNNEL ALIGNMENT; cross ticks are 1,000-feet apart

71 STRIKE AND DIP OF BED

STRIKE OF VERTICAL BED

82 STRIKE AND DIP OF OVERTURNED BED

AXIS OF ANTICLINE, short dashed in older alluvium, dotted where buried, queried where uncertain or continuation unknown

AXIS OF OVERTURNED ANTICLINE

AXIS OF SYNCLINE, short dashed in older alluvium, dotted where buried, queried where uncertain or continuation unknown

GEOLOGIC CONTACT, short dashed between subunits of older alluvium

POSSIBLE FAULT, dotted where buried

○ Project-Specific Geotechnical Borings, Tunnel Borings Denoted by DD-
Station Borings Denoted by SD-
Borings from Preliminary Engineering Program Denoted by PE-

2816D ◆ Water Well; monitored by L.A. Co. Dept. Public Works

SEWER TUNNEL ALIGNMENT

47 (35) Bedding attitude in tunnel by Leroy Crandall and Associates (1979); circled number indicates amount of dip in Pleistocene deposits

Shear zone in tunnel

Geology Adapted and Modified from Lamar (1970) and Bullard and Lettis (1993).

Base Map: U.S. Geological Survey, Los Angeles 7.5 Minute Quadrangle; dated 1966, photorevised 1981

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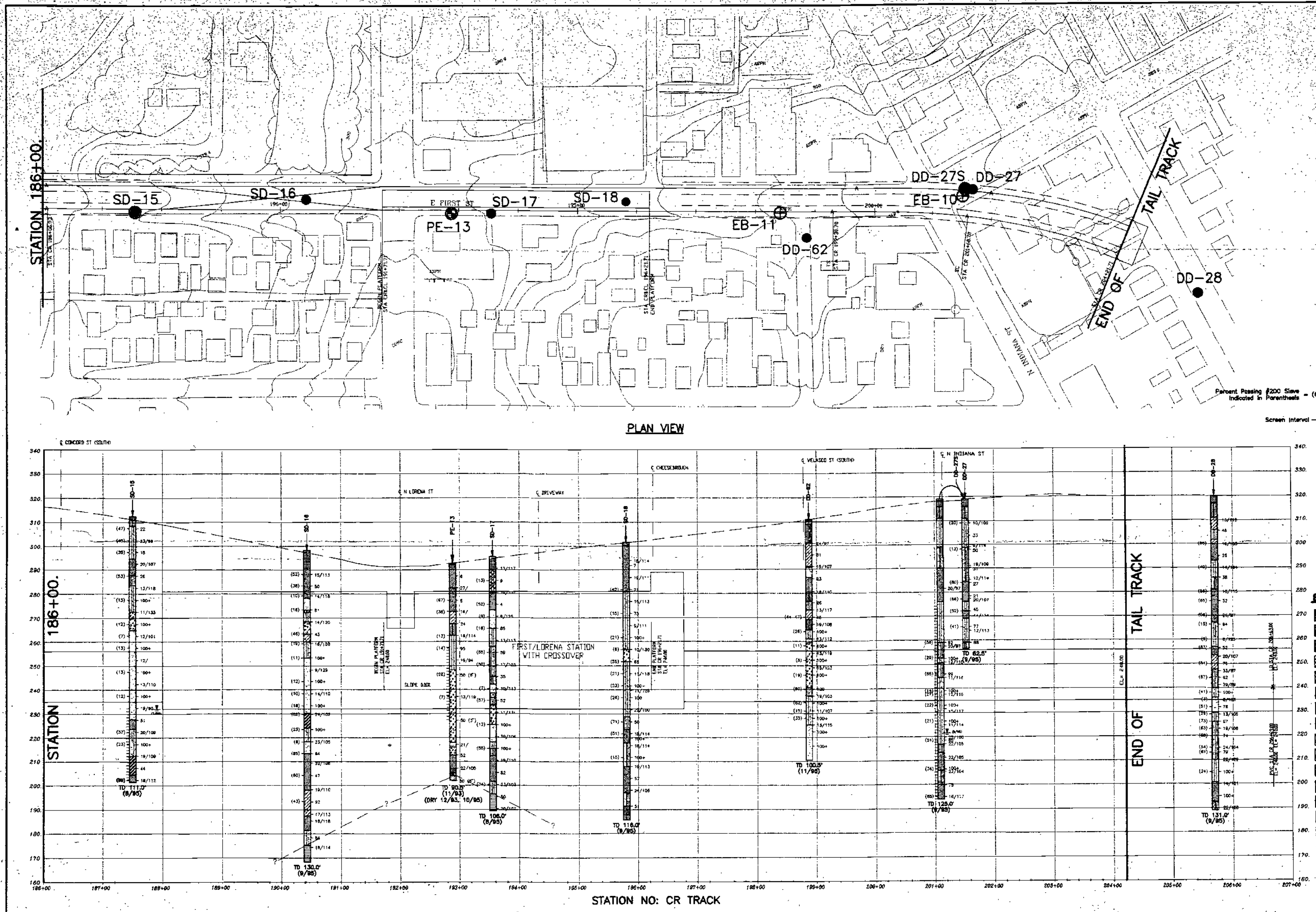
Geotechnical Investigation
Eastside Extension
Metro Red Line

Key Plans and Geology

5-96

Plate 1



Percent Passing #200 Sieve Indicated in Parenthesis = (66)

Screen Interval

Groundwater Level and Date Measured

18/102 = Moisture Content (%) / Dry Density (pcf)

16/ = Moisture Content (%) Only

21 = SPT Blow Count for Final 12 Inches of Penetration

21 (7") = SPT Blow Count for 7 Inches of Penetration

TD 90.5' Total Depth of Boring

(11/93) Date Drilled

(DRY 12/93) Dry Hole and Date Measured

● Rotary Wash Borings, this investigation

● Rotary Wash Borings with Monitoring Well, this investigation

● Rotary Wash Borings with Monitoring Well, preliminary investigation (GeoTransit Consultants, 1994a)

⊕ Environmental Boring (GeoTransit Consultants; 1994b)

SYMBOLS

▥ SILT, SILT with SAND (ML)
▦ SANDY SILT (ML)
▩ SILTY CLAY, CLAYEY SILT, SILTY CLAY with SAND (CL-ML, CL/ML, ML/CL)
▨ LEAN CLAY, LEAN CLAY with SAND (CL)
▩ SANDY LEAN CLAY, GRAVELLY LEAN CLAY (CL)
▨ FAT CLAY, FAT CLAY with SAND (CH)
▨ SANDY FAT CLAY, GRAVELLY FAT CLAY (CH)
▒ SAND, SAND with SILT, SAND with CLAY, SAND with GRAVEL (SP, SW, SP-SM, SP-SC, SW-SM, SW-SC)
▒ SILTY SAND, SILTY SAND with GRAVEL (SM)
▨ CLAYEY SAND, CLAYEY SAND with GRAVEL (SC)
▒ GRAVEL, GRAVEL with SAND, GRAVEL with SILT, GRAVEL with CLAY (GP, GW, GP-GM, GP-GC, GW-GM, GW-GC)
▩ SILTY GRAVEL, CLAYEY GRAVEL (GM, GC, GC-GM)
▤ SILTSTONE
☰ CLAYSTONE
▦ FILL
—?— BEDROCK CONTACT, queried where uncertian

REV.	DATE	BY	SUB	APP	DESCRIPTION
2	01-05-96				FINAL DRAFT
1	12-06-95				ISSUED PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED IN PART THROUGH A GRANT FROM THE U.S. DEPARTMENT OF TRANSPORTATION, URBAN MASS TRANSPORTATION ADMINISTRATION, UNDER THE URBAN MASS TRANSPORTATION ACT OF 1964, AS AMENDED, AND IN PART BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

DESIGNED BY:
DRAWN BY:
CHECKED BY:
IN CHARGE:
DATE:

Metropolitan Transportation Authority
METRO RED LINE

GeoTransit Consultants
SUBMITTED:

PLAN AND PROFILE
FIRST/LORENA STATION &
TAIL TRACK TUNNELS
EASTSIDE EXTENSION

PLATE 2

PROJECT NO: 95-8347
DRAWING NO:
REV.
SCALE: HORIZ: 1'=80'
VERT: 1'=20'
SHEET NO: 1 OF 1

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

FIELD EXPLORATION

GENERAL

The field exploration program, consisting of eight rotary wash borings (SD-15, SD-16, SD-17, SD-18, DD-27, DD-27S, DD-28 and DD-62), was conducted from August 25 to November 15 and November 16, 1995. Prior to this investigation, one geotechnical boring (PE-13) was drilled in November 1993. Mud rotary drilling was performed by the drilling subcontractor C & L Drilling, Inc. Logging of borings and soil sampling were performed by geologists or engineers supervised by a Registered Geotechnical Engineer (RGE) or a Certified Engineering Geologist (CEG) from GeoTransit Consultants. Two monitoring wells were installed during this investigation at selected boring locations to monitor groundwater levels and one monitoring well was installed previously. The boring locations were established in the field by a GeoTransit Consultant engineer by tape measurements from adjacent street curbs and other fixed features. The approximate locations of exploration points along the alignment are shown in Plate 2.

BORINGS AND MONITORING WELL INSTALLATION

Exploratory borings were drilled using a truck-mounted MAYHEW-1000 mud rotary rig. The mud rotary borings for the geotechnical exploration were approximately 5 to 6 inches in diameter and the depths ranged from 62.5 feet to 131.0 feet below ground surface (BGS). In general, soil samples were obtained at 5-foot-depth intervals and at significant changes in stratigraphy, by alternately using standard split-spoon samplers (Standard Penetration Test Method) and 2.4-inch inner diameter California-type drive samplers lined with 1-inch high brass rings. A cone-type plastic catcher and a sample catcher assembly consisting of a brass ring containing 8 metallic teeth and a rubber donut were used with the split-spoon samplers and California drive samplers, respectively.

Standard Penetration Tests (SPT) were performed according to the American Society for Testing Materials (ASTM) Standard Procedure D1586. This method consists of driving the standard split-spoon sampler 18 inches into the soil with a 140-pound hammer falling 30 inches. Blow counts were recorded for each 6-inch driving increment. The number of blows required to drive a standard split-spoon sampler for the last 12 of 18 inches is called the standard penetration resistance.

The blows were terminated when one of the following occurred:

- A total of 100 blows was reached for penetration of 12 inches or less
- No obvious sampler advance was observed during driving
- Sampler was advanced 18 inches.

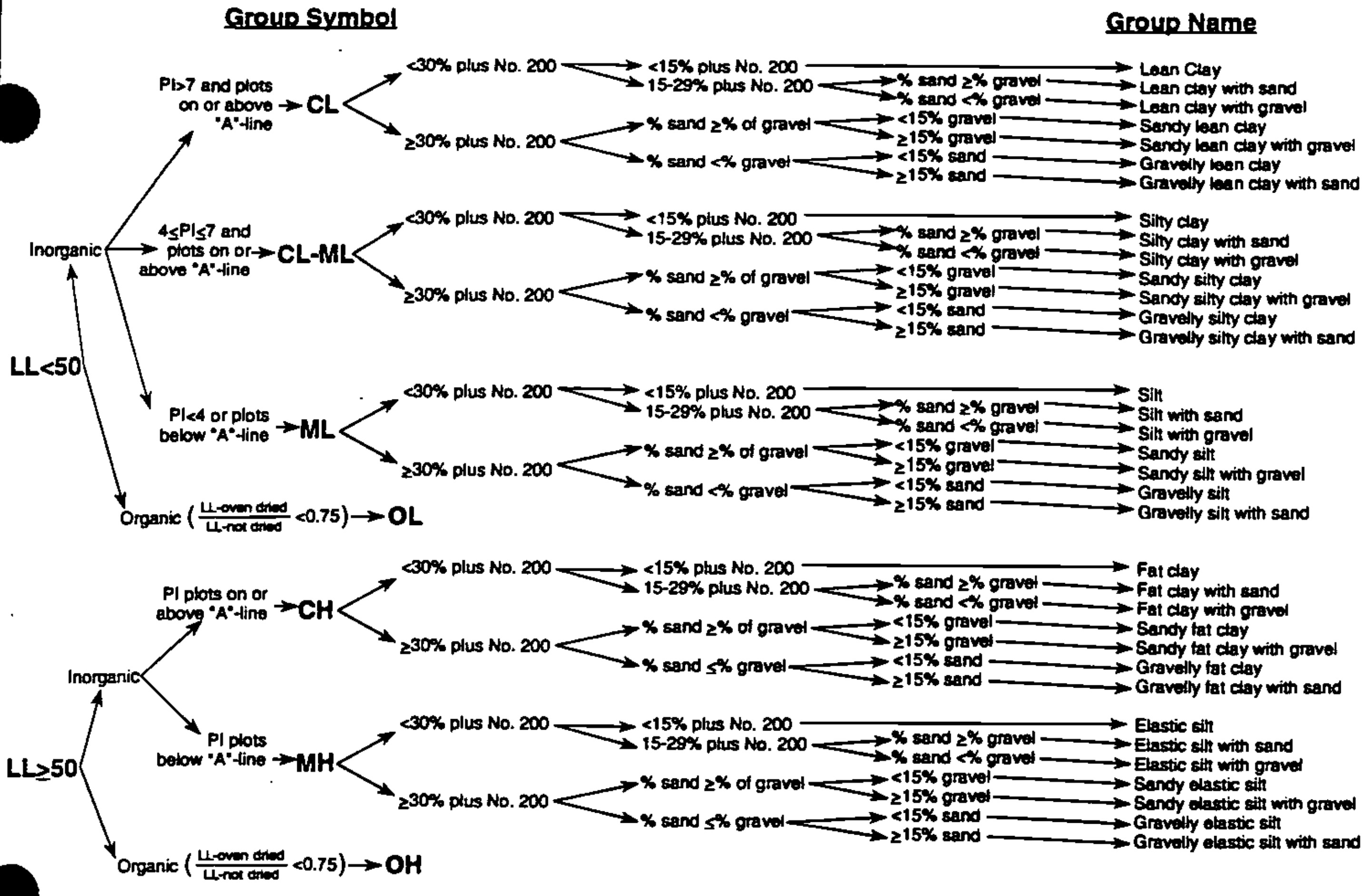
Split spoon samples were examined and carefully removed from the spoon, logged, sealed and transported to the laboratory for testing.

Relatively undisturbed soil samples were obtained with the California-type drive sampler by driving the sampler with a 300- or 400-pound downhole hammer falling 18 inches. Hammer weights and the corresponding drop heights used to drive the sampler are indicated in boring logs. Blow counts were recorded for each 6-inch driving increment. The 12-inch long samples were examined and the middle six rings were collected in a plastic tube for transportation to the laboratory for testing. In order to compare the blow counts on a uniform basis, the California sampler downhole hammer blow counts were corrected for sampler dimensions, different hammer weights, height of fall, and buoyancy and viscous drag in the bentonite drilling mud. The corrected blow counts designated as "Equivalent SPT Value" are presented in Table 3-6.

Two monitoring wells were installed during this investigation after the completion of soil sampling at selected station locations to monitor groundwater levels. One monitoring wells installed previously. With the rotary-wash drilling method, observation of groundwater levels during drilling is generally not possible since the borings are filled with drilling fluid. Sometimes, estimates of the groundwater level were made by observing the moisture conditions of the samples.

Water from local fire hydrants was used to flush each boring to remove or thin the drilling fluid prior to monitoring well installation. Backfill material consisting of #2/12 Monterey /1/2-inch size bentonite pellets/sluff was placed at the bottom of borings. Following this, the PVC casing assembly was inserted into the boring, filter sand was placed to about 1.5 to 10 feet above the top of the screened interval, and 1/2-inch size bentonite pellets were then dropped in to form a 1.5 to 3.5-foot thick bentonite plug. Cement grout was then pumped into the annulus from the top of the bentonite plug to within 12 inches to 18 inches of the ground surface. A circular, 8-inch diameter metal traffic box was installed at the monitoring well location flush with the ground surface. Monitoring well installation diagrams are presented in Figures A-3B, A-7B and A-11B.

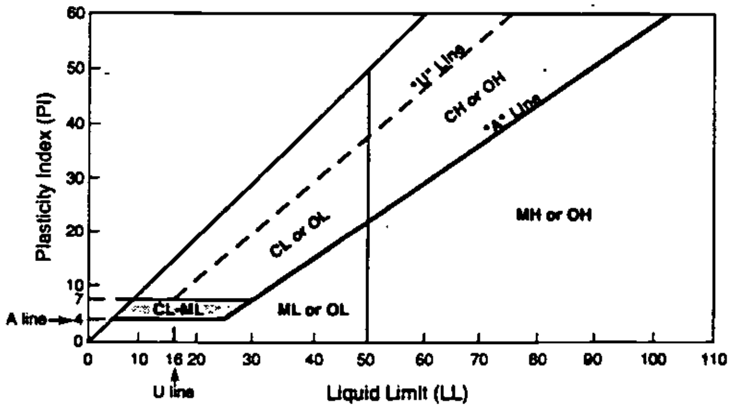
The logs of borings accompany this appendix. Stratification lines on the logs represent approximate boundaries between predominant soil types. Geologic and engineering descriptions and material classifications used on the boring logs are in general accordance with the Unified Soil Classification System (USCS) and the American Society for Testing and Materials (ASTM D2487 and D2488) Methods. The material was visually classified in the field and independently verified in the laboratory using the general guidelines presented in ASTM D2488. For samples on which laboratory classification (gradation and/or Atterberg Limits) were performed, material classification was done in general accordance with ASTM D2487. Figure A-1 presents the Laboratory and Field Soil Classification Systems used in material classification, as presented on the logs. Figure A-2 presents an explanation for the log symbols and terminology.



Flow Chart for Classifying Fine-Grained Soils (50% or More Passes No. 200 Sieve)

	Laboratory Classification Criteria	
Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC 5% to 12% Borderline cases requiring use of dual symbols	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 4 $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting all gradation requirements for GW	
	Atterberg limits plot below "A" line or PI less than 4	Atterberg limits plot above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
	Atterberg limits plot above "A" line and PI greater than 7	
	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 6 $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting all gradation requirements for SW	
	Atterberg limits plot below "A" line or PI less than 4	Atterberg limits plot above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
	Atterberg limits plot above "A" line with PI greater than 7	

From: ASTM 2487 ^{b}All Sieve Sizes on this Chart are U.S. Standard



Plasticity Chart for Laboratory Classification of Fine Grained Soils

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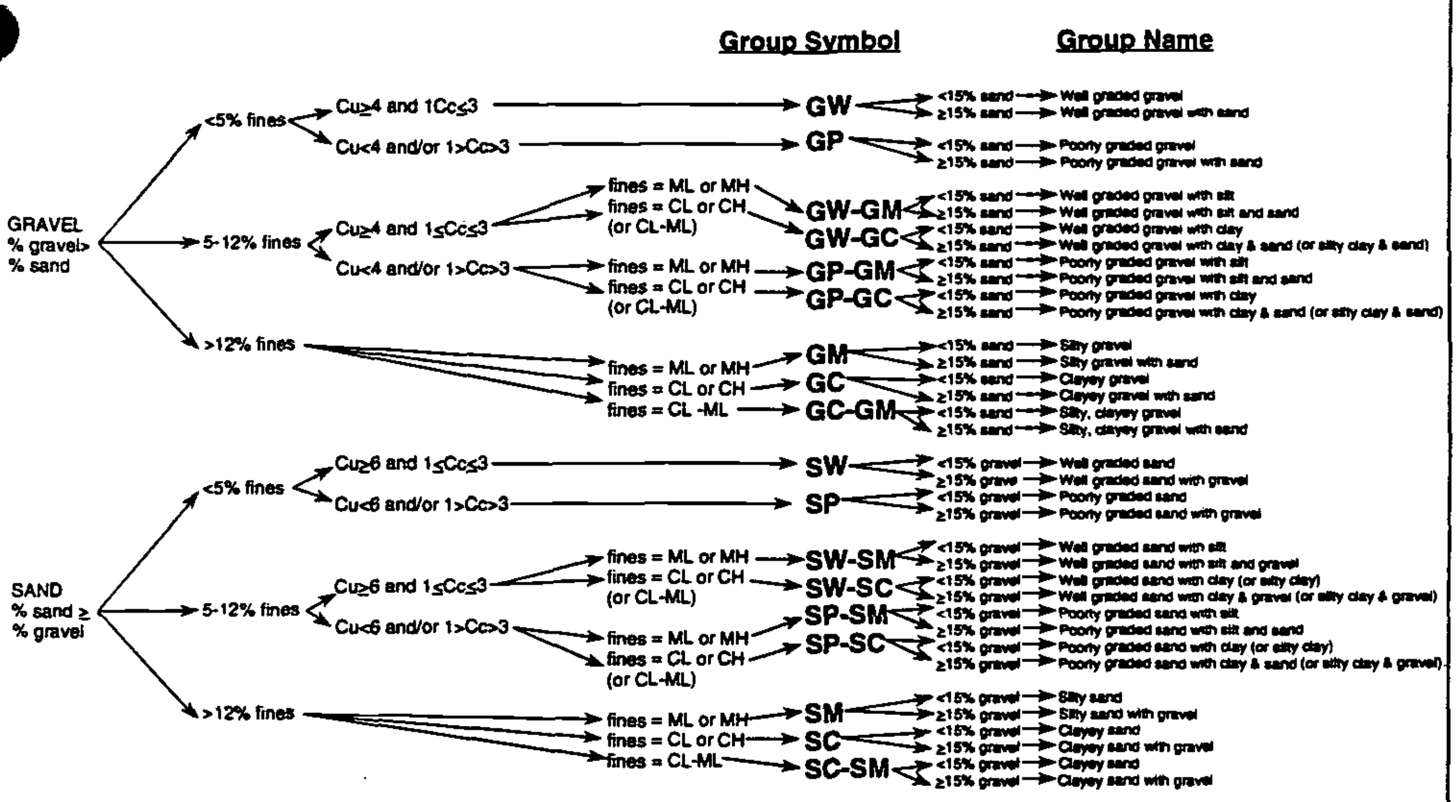
Laboratory Soil Classification System (Sheet 1 of 4)

4-96

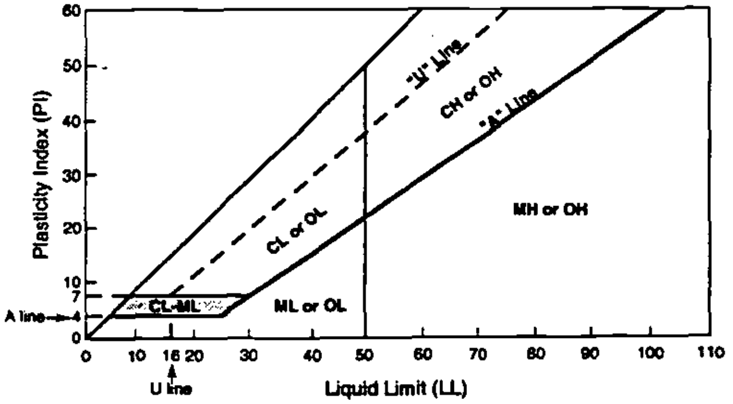
Figure A-1



Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)

	Laboratory Classification Criteria	
Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC 5% to 12% Borderline cases requiring use of dual symbols	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 4 $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting all gradation requirements for GW	
	Atterberg limits plot below "A" line or PI less than 4	Atterberg limits plot above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
	Atterberg limits plot above "A" line and PI greater than 7	
	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 6 $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting all gradation requirements for SW	
	Atterberg limits plot below "A" line or PI less than 4	Atterberg limits plot above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
	Atterberg limits plot above "A" line with PI greater than 7	

From: ASTM 2487 [b]All Sieve Sizes on this Chart are U.S. Standard



Plasticity Chart for Laboratory Classification of Fine Grained Soils

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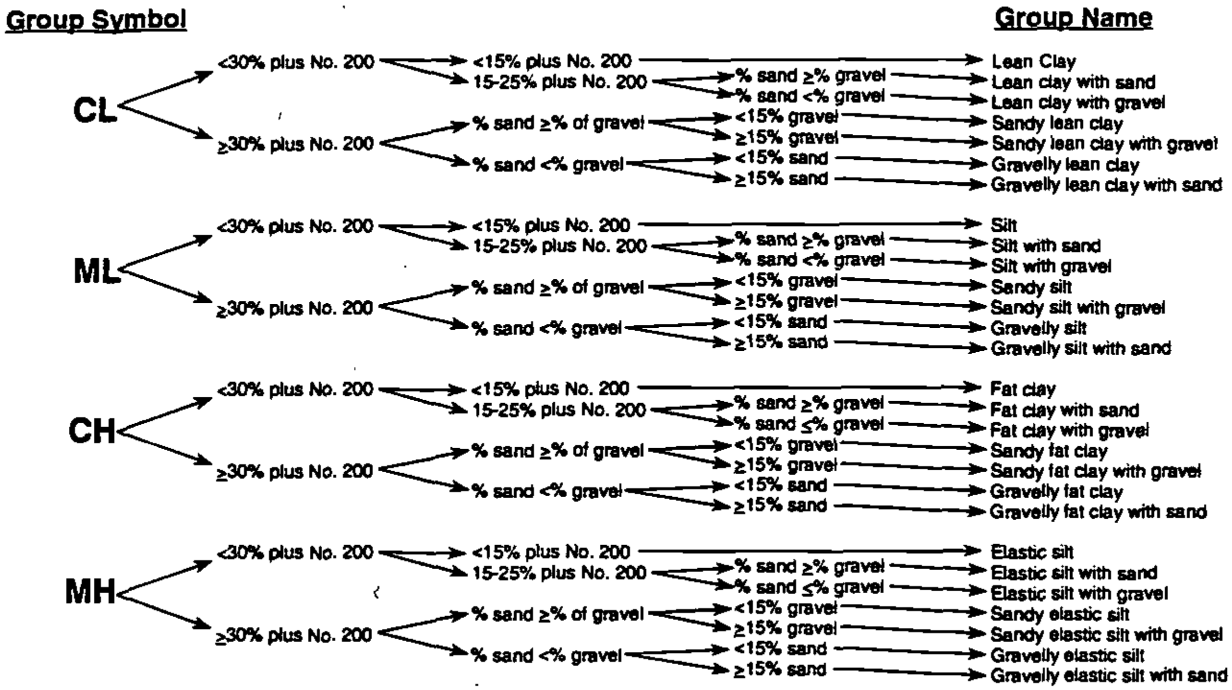
Project No.: 95-8347-04

Geotechnical Investigation
Eastside Extension
Metro Red Line

Laboratory Soil Classification System (Sheet 2 of 4)

4-96

Figure A-1



Note: Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5%

Flow Chart for Identifying Fine-Grained Soils (More than 50% Fines)

From: ASTM D2488

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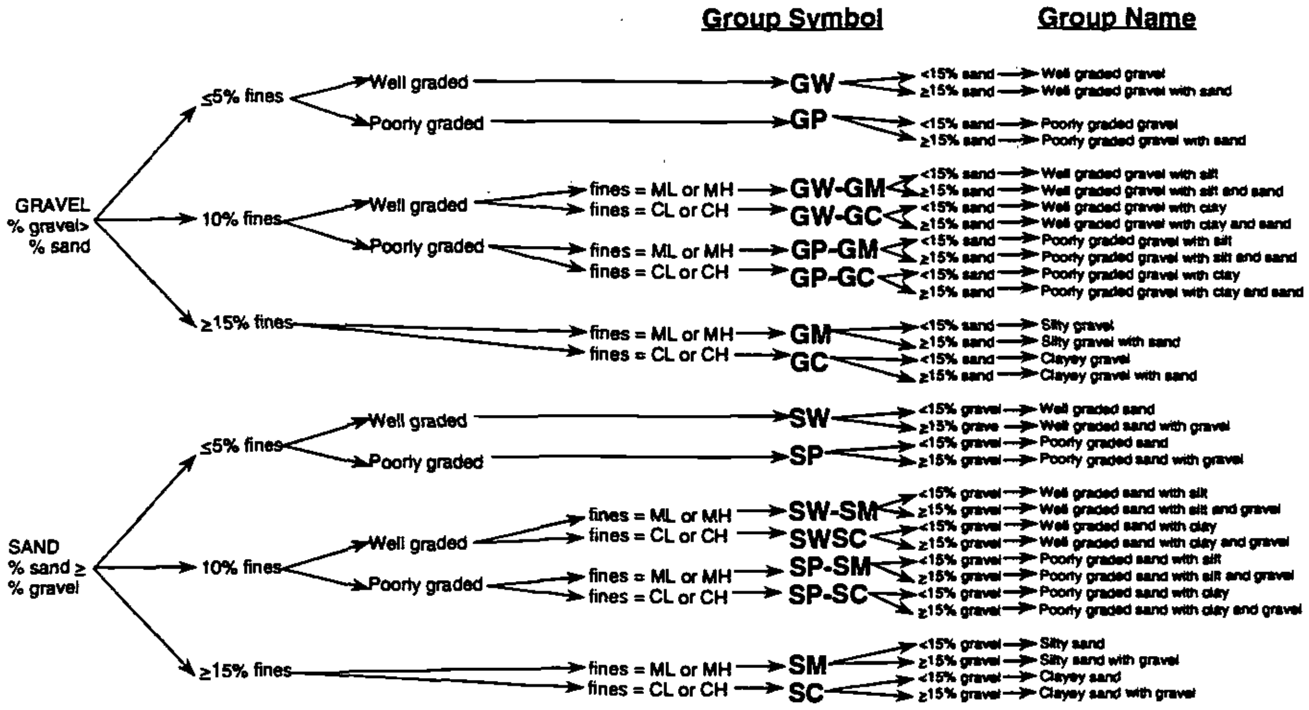
Eastside Extension
Metro Red Line

Field Soil Classification System (Sheet 3 of 4)

4-98

Figure A-1



Note: Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5%

Flow Chart for Identifying Coarse-Grained Soils (Less than 50% Fines)

From: ASTM D2488

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Field Soil
Classification System
(Sheet 4 of 4)

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Figure A-1

SAMPLE/TYPE

S	STANDARD SPLIT SPOON SAMPLE
D	$2\frac{1}{2}$" DIA., 12" DRIVE SAMPLE
P	$2\frac{7}{8}$" DIA., 36" PITCHER SAMPLE
B	BAG SAMPLE

SIZE PROPORTIONS

FINE-GRAINED SOILS (50% or more passes no. 200 sieve)		COARSE-GRAINED SOILS (more than 50% retained on no. 200 sieve)	
<u>Coarse-grained Proportions</u>		<u>Fine-grained Proportions</u>	
TRACE	<5%	TRACE	<5%
SOME	5 TO 15%	WITH	5 TO 15%
WITH	15 TO 30%	USE MODIFIER	≥15%
USE MODIFIER	>30%		

SOIL MOISTURE

DRY - ABSENCE OF WATER, DRY TO TOUCH

MOIST - DAMP BUT NO VISIBLE WATER

WET - VISIBLE FREE WATER, USUALLY
SOIL IS BELOW WATER TABLE

EXPLANATIONS

BLOW COUNT (PENETRATION RESISTANCE)

- BLOW COUNTS FOR 6" INTERVALS EXCEPT AS NOTED

BLOW COUNTS FOR THE CALIFORNIA DRIVE SAMPLER ARE OBTAINED WITH EITHER A 300-, A 380- OR A 400- POUND DOWNHOLE HAMMER FALLING 18 INCHES. EQUIVALENT SPT VALUES ARE OBTAINED BY APPLYING APPROPRIATE CORRECTIONS, AS INDICATED ON FIGURE A-2 (SHEETS 2 OF 4 AND 3 OF 4)

RECOVERY (INCHES RECOVERED/INCHES TOTAL SAMPLE)

MOISTURE CONTENT (%) - LABORATORY DETERMINED MOISTURE CONTENT

PMT-	PRESSURE METER TEST
PPM -	PARTS PER MILLION
PCF -	POUNDS PER CUBIC FOOT
OVA -	ORGANIC VAPOR ANALYZER
[BACKGROUND] -	BACKGROUND OVA READING
N/A -	NOT APPLICABLE
/ -	DENOTES ALTERNATING SOIL TYPES IN A LAYER (EXAMPLE: SP/SW i.e. ALTERNATING POORLY AND WELL GRADED SANDS IN A PREDOMINANTLY SANDY LAYER)

TERMS FOR THE DESCRIPTION OF BEDROCK (SHEET 4 OF 4)

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Eastside Extension
Metro Red Line

Explanation of Boring Logs
(Sheet 1 of 4)

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Figure A-2

EQUIVALENT SPT BLOWCOUNTS FOR CALIFORNIA DRIVE SAMPLES

FOR CALIFORNIA DRIVE SAMPLES, EQUIVALENT SPT VALUES ARE OBTAINED BY APPLYING THE APPROPRIATE CORRECTIONS FOR DIFFERENT HAMMER WEIGHTS, HAMMER DROP, SAMPLER DIMENSIONS, AND BUOYANCY AND VISCOUS DRAG WITHIN THE DRILLING MUD. THESE CORRECTIONS ARE INDICATED BELOW:

FINE-GRAINED SOILS

EQUATION 1

$$N_{SPT} = N \times \frac{W \times H\,(2.0^2 - 1.375^2)}{4200\,(D_o^2 - D_i^2)}$$

WHERE:

R_C = SAMPLER HAMMER RATIO (FT/LB)

W = WEIGHT OF DOWNHOLE HAMMER (300 LBS, 380 LBS OR 400 LBS)

H = HAMMER DROP (18-INCHES)

Do, Di = OUTER (3.25 INCHES) AND INNER (2.41 INCHES) DIAMETERS OF THE CALIFORNIA DRIVE SAMPLER

EQUATION 2 (for Standard Penetration Hammer)

$$R_C = 4.185 \times 10^{-5}$$

BASED ON EQUATIONS 1 AND 2 AND SHEAR STRENGTH OF SOIL (FOUNDATION ENGINEERING HANDBOOK, WINTERKORN & FANG, FIGURE 1-22); AND ASSUMING A 20% REDUCTION FOR THE BUOYANCY AND VISCOUS DRAG WITHIN THE DRILLING MUD, EQUIVALENT SPT VALUES ARE OBTAINED AS FOLLOWS:

FOR 300 LBS DOWNHOLE HAMMER:	$N_{SPT} = 0.65\ N\ (N \leq 48)$	$N_{SPT} = 0.50\ N\ (N > 48)$
FOR 380 LBS DOWNHOLE HAMMER:	$N_{SPT} = 0.75\ N\ (N \leq 39)$	$N_{SPT} = 0.63\ N\ (N > 39)$
FOR 400 LBS DOWNHOLE HAMMER:	$N_{SPT} = 0.77\ N\ (N \leq 37)$	$N_{SPT} = 0.66\ N\ (N > 37)$

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Metro Red Line

Explanation of Boring Logs
(Sheet 2 of 4)

12-95

Figure A-2

EQUIVALENT SPT BLOWCOUNTS FOR CALIFORNIA DRIVE SAMPLES (CONTINUED)

FOR CALIFORNIA DRIVE SAMPLES, EQUIVALENT SPT VALUES ARE OBTAINED BY APPLYING THE APPROPRIATE CORRECTIONS FOR DIFFERENT HAMMER WEIGHTS, HAMMER DROP, SAMPLER DIMENSIONS, AND BUOYANCY AND VISCOUS DRAG WITHIN THE DRILLING MUD. THESE CORRECTIONS ARE INDICATED BELOW:

COARSE-GRAINED SOILS

EQUATION 1

$$N_{SPT} = N\text{x} \frac{W \text{ x } H \, (2.0^2 - 1.375^2)}{4200 \, (D_o^2 - D_i^2)}$$

WHERE:

N_{SPT} = EQUIVALENT SPT BLOW COUNTS

N = DOWNHOLE HAMMER BLOW COUNTS FOR 12" PENETRATION OF CALIFORNIA DRIVE SAMPLER

W = WEIGHT OF DOWNHOLE HAMMER (300 LBS, 380 LBS OR 400 LBS)

H = HAMMER DROP (18-INCHES)

D_o, D_i = OUTER (3.25 INCHES) AND INNER (2.41 INCHES) DIAMETERS OF THE CALIFORNIA DRIVE SAMPLER

ASSUMING A 20% REDUCTION FOR THE BUOYANCY AND VISCOUS DRAG WITHIN THE DRILLING MUD, EQUIVALENT SPT VALUES OBTAINED FROM EQUATION 1 ARE AS FOLLOWS:

FOR 300 LBS DOWNHOLE HAMMER: N_{SPT} = 0.46 N

FOR 380 LBS DOWNHOLE HAMMER: N_{SPT} = 0.58 N

FOR 400 LBS DOWNHOLE HAMMER: N_{SPT} = 0.61 N

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Geotechnical Investigation
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Metro Red Line

Explanation of Boring Logs
(Sheet 3 of 4)

12-95

Figure A-2

Terms for the Description of Bedrock

ROCK STRENGTH

DESCRIPTOR	CRITERIA
Extremely Hard	Core, fragment, or exposure cannot be scratched with knife or sharp pick, can only be chipped with repeated heavy hammer blows.
Very Hard	Cannot be scratched with knife or sharp pick. Core or fragment breaks with repeated hammer blows.
Hard	Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.
Moderately Hard	Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.
Moderately Soft	Can be grooved 1/16 inch (2mm) deep by knife or sharp pick with moderate or heavy pressure. Core or fragment breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved with fingernail, or carved with a knife. Breaks with light manual pressure.

Any bedrock unit softer than H7, very soft, is to be described using USBR 5000 consistency descriptors.

Note: Although "sharp pick" is included in these definitions, descriptions of ability to be scratched, grooved or gouged by a knife is the preferred criteria.

DISCONTINUITY ROUGHNESS

CLASSIFICATION	DESCRIPTION
Smooth	Appears smooth and is essentially smooth to the touch. May be slickensides.
Slightly Rough	Asperities (undulations) on the fracture surfaces are visible and can be distinctly felt.
Medium Rough	Asperities are clearly visible and fracture surface feels abrasive.
Rough	Large angular asperities can be seen. Some ridges and high angular steps evident.
Very Rough	Near-vertical steps and ridges occur on the fracture surface.

FRACTURE FILLING

DESCRIPTION	DEFINITION
Clean	No film or coating
Very Thin	0 - 1 mm
Moderately Thin	1 - 3 mm
Thin	3 - 10 mm
Moderately Thick	10 - 30 mm
Thick	> 30 mm

Note:

1) Terminology adopted and modified from Engineering Geology Field Manual, U.S. Department of the Interior Bureau of Reclamation.
2) Color description from the Geological Society of America's Rock Color Chart.
3) For descriptions of weak sedimentary rocks also see key to soil definitions.

WEATHERING

DESIGNATION	ABBREVIATION	DESCRIPTION
Fresh	W1	Rock shows no discoloration, loss of strength or other effects due to weathering.
Slightly Weathered	W3	Rock is partly discolored, notably near fracture surfaces. Fractures may be open and stained. Intact rock is not noticeably weaker than fresh rock.
Moderately Weathered	W5	Rock is predominately discolored. Fractures are open, contain filling and may have greater discoloration with alteration penetrating inwards. Intact rock is noticeably weaker than fresh rock.
Intensely Weathered	W7	Rock is discolored throughout. Fractures discolored and contain thick filling. Rock is partly friable but lithorelicts are present. Original rock fabric is mainly preserved.
Decomposed	W9	Discolored or oxidized throughout, resistant minerals such as quartz may be unaltered, complete separation of grain boundaries, resembles soil but remnant rock structure may be preserved.
Residual Soil	RS	Rock is completely weathered to soil and original structure, and texture is completely destroyed.

DISCONTINUITY SPACING

SPACING	FRACTURE	BEDDING & FOLIATION
Less than 1/2 inch (<12 mm)	Extremely Close	Laminated
0.05 to 0.1 foot (12 mm to 30 mm)	Very Close	Very Thin
0.1 foot to 0.3 foot (30 mm to 1 cm)	Close	Thin
0.3 foot to 1 foot (1 cm to 30 cm)	Moderately Close	Medium
1 foot to 3 feet (30 cm to 1 m)	Wide	Thick
More than 3 feet (<1 m)	Very Wide	Very Thick

SEPARATION OF FRACTURE WALLS (FRACTURE OPENNESS)

DESCRIPTION	SEPARATION OF WALL (mm)
Tight	0
Slightly Open	1.0
Moderately Open	1.0 - 3.0
Open	3.0 - 10.0
Moderately Wide	10.0 - 30.0
Wide	> 30.0

GeoTransit Consultants

Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

Explanation of Boring Logs
(Sheet 4 of 4)

12-95

Figure A-2

BORING LOGS

MEMORANDUM

95-8347

Date: June 20, 1995

To: Bomi Ghadiali, EMC

From: Suji Somasundaram, Geotransit Consultants

Subject: Hammer Weight Calibration
Eastside Extension

Attached please find calibration certificates for the two 140 pound hammers (serial numbers MW8701 and MW8702) submitted to us by C & L Drilling, for the borings within the Union Station to Little Tokyo station segment. We understand that Class F - tolerance indicates ± 2.3 grams.

C & L DRILLING CO.

Contractors License #568520

600 E. LAMBERT RD.
LA HABRA, CALIFORNIA 90631
(310) 691-3533 FAX (310) 690-0264

FAX TRANSMITTAL

DATE 6-6-95

COMPANY Earth Technology

ATTENTION Grant Miller

FAX# 714-724-1557

OF PAGES TRANSMITTED Cover + 2

REMARKS

Calibration - SPT hammers

IF YOU DO NOT RECEIVE ALL OF THE PAGES INDICATED ABOVE, PLEASE CALL (310) 691-3533, AND WE WILL RE-TRANSMIT THE MISSING PAGES.

EMPIRE Scale co

12065 CLARK STREET
SANTA FE SPRINGS, CA 90670

(310) 946-0761
FAX (310) 944-0770

June 5, 1995

C & L Drilling
600 E. Lambert Rd
La Habra, CA 90631

Service Order # 22556
P.O.# Verbal - Odell
S/N MW8701

CERTIFICATION OF GUARANTEED CALIBRATION

This is to certify that the Drill Head was thoroughly tested before packing and shipment using weights directly traceable to National Institute of Standards and Technology. This equipment guaranteed accurate to within Class "F" Tolerance.
Complies to Mil-Spec. 45662A
MS-12572

ACTUAL WEIGHT: 140.0 Lbs.

Service Technician

EMPIRE Scale co.

12065 CLARK STREET
SANTA FE SPRINGS, CA 90670

(310) 946-0761
FAX (310) 944-0770

June 5, 1995

C & L Drilling
600 E. Lambert Rd.
La Habra, CA 90631

Service Order # 22356
P.O.# Verbal - Odell
S/N MW8702

CERTIFICATION OF GUARANTEED CALIBRATION

This is to certify that the Drill Head was thoroughly tested before packing and shipment using weights directly traceable to National Institute of Standards and Technology. This equipment guaranteed accurate to within Class "F" Tolerance .
Complies to Mil-Spec. 45662A
MS-12572

ACTUAL WEIGHT: 140.0 Lbs.

Service Technician

GeoTransit Consultants

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**		
Project Number:	**95-8347**	Boring Number: **SD-15**	Sheet 1 of 5
Boring Location:	**On 1st St., 105' E of Concord**	Elevation and Datum(feet): **312.0**	
Health and Safety:	**Level D**	Date Started: **8/26/95**	Date Finished: **8/26/95**
Drilling Equipment:	**Mayhew 1000**	Total Depth (feet): **113.0**	Depth to Bedrock(feet): **None**
Drilling Method:	**Rotary Wash**	Number of Samples: **22**	Depth to Water (feet): **79.1**
Boring Diameter:	**5 inches**	Completion Information: **Installed Monitoring Well**	
Hammer Information: **SPT Hammer: 140-lb and 30-inch drop** **Downhole Hammer: 400-lb and 18-inch drop**		Logged By: **C. Marshall Payne**	Checked By: **Gamini Weeratunga**

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		Asphalt and Concrete with base										06:50
		ALLUVIUM LEAN CLAY/SILT; reddish brown, moist, stiff, trace fine-grained sand	CL/ML	Qa								
5		SILTY SAND; yellowish, brown, moist, medium dense, fine-grained sand with silt, micaceous, trace pea gravel	SM		1	S	9 12 10	14/18	7.2 [5]			07:00
		increasing silt content, sandy silt (ML)										
10		silty sand, medium dense, 1-2" thick layers of fine-grained sand			2	D	7 9	12/12	8.4 [6.8]	96	23	07:10
15		increasing sand content,			3	S	8 8 7	8/18	6.0 [5.2]			07:15
		SANDY LEAN CLAY; reddish brown, moist, stiff, low to medium plasticity clay with fine-grained sand	CL									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **SD-15** Sheet 2 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	Drilling Time
		SANDY LEAN CLAY; reddish brown, moist, stiff, low to medium plasticity clay with fine-grained sand	CL	Qa	4	D	7 11	3/12	7.0 [6.8]	107	20	07:25
25		SANDY SILT; yellowish brown, moist, very stiff, silt with fine-grained sand	ML		5	S	12 14 12	12/18	5.5 [5.2]			07:30
30		SILTY SAND; reddish brown, slightly moist, dense, fine- to medium-grained sand with silt	SM		6	D	20 30	3/12	6.6 [6.6]	118	13	07:35
35		SILTY SAND with GRAVEL; light gray to gray, slightly moist, very dense, fine- to coarse-grained sand with gravel to 1" in size, subangular	SM		7	S	45 65 10 /0"	4/12	5.2 [5.2]			07:43
40		SAND/GRAVEL; light gray, moist, very dense, fine- to coarse-grained sand and subangular to subrounded gravel, estimated gravel/cobble layer 4-6" thick (drill rig chatter)	SP/GP		8	D	40 60	2/12	6.8 [6.6]	133	11	08:00
45		trace silt			9	S	65/6" 50/2"	3/8	5.3 [5.2]			08:15
		SAND with SILT; yellowish brown, moist, medium dense, fine- to medium-grained sand, some silt	SP-SM									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

GeoTransit Consultants

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number: 95-8347	Boring Number: SD-15		Sheet 3 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	
		SAND with SILT; yellowish brown, moist, medium dense, fine- to medium-grained sand, some silt	SP-SM	Qa	10	D	20 25	8/12	6.5 [6.4]	101	12	08:30
		SAND with GRAVEL; yellowish brown, moist, very dense, fine- to medium-grained sand and subangular to subrounded gravel, 4-6" thick layers with varying gravel content, some silt	SP									
55					11	S	75/6" 75/3"	6/9	5.2 [5.2]			08:48
60					12	D	100/6"	3.5/6	6.4 [6.4]		12	09:03
65		fine- to medium-grained sand, some gravel to 3/8" in size, trace silt			13	S	65/6" 90/6"	8/12	5.3 [5.2]		10	09:35
70		increasing gravel content			14	D	65 85	7/12	6.4 [6.2]	110	13	09:50
75		well graded fine- to medium-grained sand with (SW), some gravel up to 3/8" size, trace silt			15	S	145/6"	6/6	5.6 [5.3]			10:00
▼												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**
Project Number:	**95-8347**
Boring Number:	**SD-15**
Sheet	4 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Samples Blow Count (no./6 inches)	Samples Recovery (inches/inches)	Samples OVA (ppm) [Background]	Samples Dry Density (pcf)	Samples Moisture Content(%)	Drilling Time
		SAND with GRAVEL; gray/yellowish brown, wet, very dense, fine- to medium-grained sand with gravel up to 1" size, trace silt	SP	Qa	16	D	55 70	8/12	6.4 [6]	90	19	10:20
85		LEAN CLAY; medium brown, moist, hard, low plastic clay, some mottled iron-oxide stain, some silt, trace fine-grained sand	CL		17	S	15 23 28	18/18	6.2 [5.5]			10:40
		SILTY SAND; reddish brown and light gray, moist, medium dense, fine- grained sand with silt	SM									
90					18	D	18 25	10/12	6.2 [6.0]	109	20	10:55
		light gray to yellowish brown, dense to very dense, 1-2" thick graded layers of fine-grained sand and fine- to medium-grained sand										
95					19	S	35 80	10/12	26.4 [5.5]			11:10
100					20	D	25 30	11/12	6.0 [5.9]	109	19	12:25
		FAT CLAY; medium to dark yellowish brown, moist, hard, high plasticity clay, some iron-oxide staining, micaceous	CH									
105					21	S	17 21 23	18/18	5.6 [5.5]			11:45
		SANDY LEAN CLAY; dark yellowish brown, moist, very stiff, low plastic clasts with fine-grained sand, some gravel	CL									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

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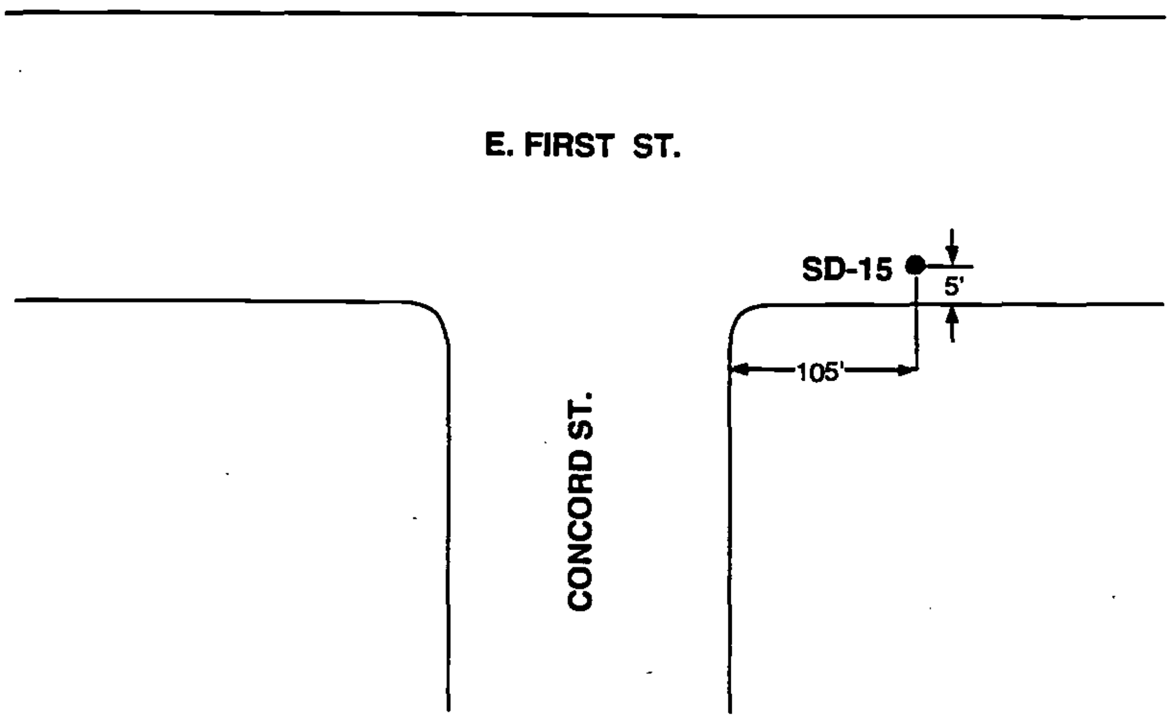
Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**
Project Number: **95-8347**	Boring Number: **SD-15** Sheet 5 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
	▨	SANDY LEAN CLAY; dark yellowish brown, moist, hard, low plasticity clay with fine-grained sand, some gravel	CL	Qa	22	D	20 25	12/12	6.0 [5.6]	112	18	12:00
		End of boring at 113 feet. Groundwater level was measured at a depth of 79.1 feet on 10/18/95.										
115												
120												
125												
130												
135												
140												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate bounderies between subsurface material types and the transitions may be gradual.



North

Not to Scale

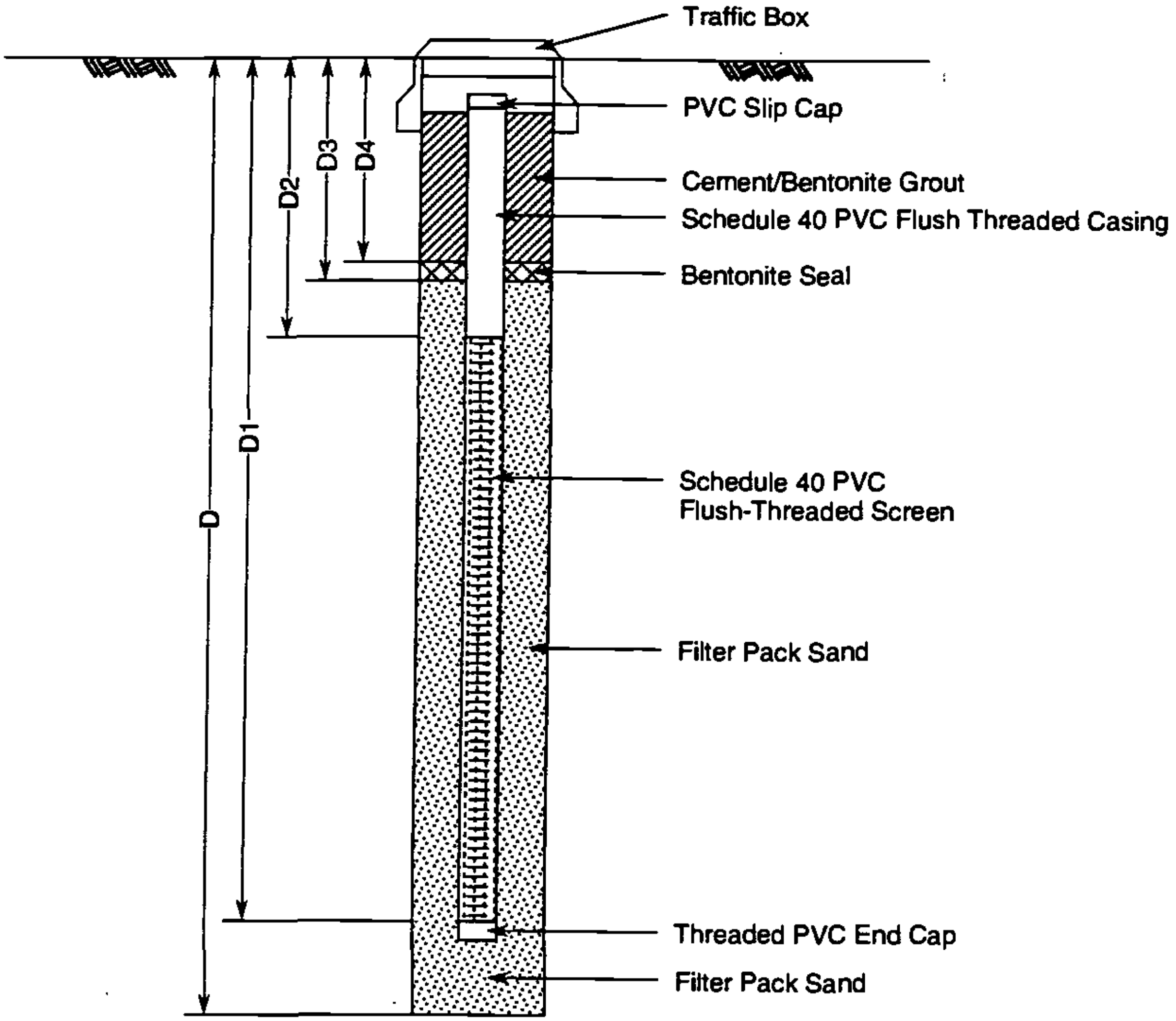
GeoTransit Consultants

Project No.: 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
SD-15

12-95

Figure A-3A



Total Depth (D)	=	113	Feet
Total Depth of Casing (D1)	=	110	Feet
Depth to Top of Well Screen (D2)	=	80	Feet
Depth to Bottom of Top Seal (D3)	=	70	Feet
Depth to Top of Top Seal (D4)	=	65	Feet
Well Casing Diameter	=	2	Inches
Well Screen Slot Size	=	0.01	Inch
Filter Pack Sand Type	=	#20	Medium Aquarium Sand
Bentonite Seal Type	=	1/2	Inch Pellets

GeoTransit Consultants

Project No. 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Monitoring Well Installation Schematic
SD-15

12-95

Figure A-3B

Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 **Boring Number:** SD-16 **Sheet** 1 **of** 5

Boring Location: On 1st St., 127 W of Lorena **Elevation and Datum(feet):** 298.0

Health and Safety: Level D **Date Started:** 9/8/95 **Date Finished:** 9/9/95

Drilling Equipment: Mayhew 1000 **Total Depth (feet):** 130.0 **Depth to Bedrock(feet):** 123

Drilling Method: Rotary Wash **Number of Samples:** 29 **Depth to Water (feet):** Unknown

Boring Diameter: 5 inches **Completion Information:** Grouted to surface

Hammer Information:
SPT Hammer: 140-lb and 30-inch drop
Downhole Hammer: 400-lb and 18-inch drop

Logged By: Greg Renfrew **Checked By:** Gamini Weeratunga

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		ASPHALT										
		ARTIFICIAL FILL GRAVELLY LEAN CLAY with SAND; dark yellowish brown, moist, stiff, low plastic clay with fine gravel (crushed rock) angular chips, some fine- to medium grained-sand	GC	af		B			.7 [0]			
												09:33
5												
		ALLUVIUM SANDY LEAN CLAY; dark yellowish brown, moist, very stiff, low plastic clay with fine- to medium-grained sand, micaceous, rootlet casts, trace gravel	CL	Qa								
10					1	D	11 14	10/12	2.7 [0]	113	15	09:40
15		increased sand content (SC), dense			2	S	13 26 24	16/18	0 [0]			09:50
		2" to 3" thick layer of gravel (GP) up to 3/4" in size, some fine- to			3	D	20 37	6/12	0 [0]	118	14	09:56

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** | Boring Number: **SD-16** | Sheet 2 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		medium-grained sand	SP-SM	Qa								
		SAND with SILT; light yellowish brown, moist, dense, fine- to medium-grained sand, some gravel upto 3/4" size, subangular, some silt										
25		GRAVEL with SAND; yellowish brown, moist, very dense, fine gravel, subangular	GP		4	S	17 25 56	16/18	0 [0]			10:06
		SILTY SAND; light yellowish brown, moist, very dense, fine- to medium- grained sand with silt, some gravel up to 3/4" in size	SM									
30		SAND/GRAVEL; yellowish brown, moist, very dense, fine- to medium gravel up to 1-1/2" in size, fine to medium-grained sand, trace silt	SP/GP		5	D	35 75	9/12	0 [0]	120	14	10:16
35		CLAYEY SAND; dark yellowish brown, moist, dense, fine-grained sand with low plastic clay, some silt, micaceous	SC		6	S	19 22 21	16/18	0 [0]			10:32
		SILTY SAND; dark yellowish brown, moist, very dense, fine- to medium-grained sand with silt, trace fine gravel	SM		7	D	30 35	10/12	0 [0]	108	16	10:40
40												
		fine- to coarse-grained sand										
45		SAND with SILT and GRAVEL; yellowish, brown, moist, dense fine- to coarse-grained sand, some fine to coarse gravel up to 2" in size, some silt	SW-SM		8	S	40 85	6/12	0 [0]			10:49

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **SD-16** Sheet 3 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SAND with SILT and GRAVEL; yellowish, brown, moist, dense fine- to coarse-grained sand, some fine to coarse gravel up to 2" in size, some silt	SW-SM	Qa	9	D	65 100 /5"	8/11	0 [0]	129	9	11:11
55					10	S	100 /6"	6/6	0 [0]			11:16
60		3-1/2" diameter cobbles			11	D	125/6"	2/6	0 [0]	110	14	11:30
65		increased sand and silt content			12	S	45 57 /5"	10/11	0 [0]			11:44
		FAT CLAY; dark yellowish brown, moist, very stiff, high plasticity clay, some fine-grained sand	CH		13	D	17 22			102	24	11:58
70												
					14	D	15 19	0/12				12:06
		graded with coarse-grained sand										
75		SILTY SAND; dark yellowish brown, very dense, fine-grained sand with silt, micaceous	SM		15	S	43 68	15/18	0 [0]			12:14

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number: 95-8347	Boring Number: SD-16	Sheet 4 of 5	

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SAND with SILT; yellowish brown, wet, dense, micaceous, fine grained-sand, some silt	SP-SM	Qa	16	D	40 63	4/12	0 [0]	105	23	12:30
85		LEAN CLAY; dark yellowish brown, moist, hard, low to medium plasticity clay, some fine-grained sand	CL		17	S	15 32 32	16/18	0 [0]			12:43
					18	D	14 20	9/12	0 [0]	108	20	
90												12:58
		increased sand content to 101.5', SANDY CLAY (CL)			19	S	15 19 28	18/18	0 [0]			
95												13:12
100					20	D	17 21	9/12	0 [0]	110	19	13:25
		CLAYEY SAND; dark yellowish brown, moist, dense to very dense, fine- to medium-grained sand and low plasticity clay	SC									
105		layers of silty sand (SM)			21	S	22 30 62	18/18	0 [0]			13:45

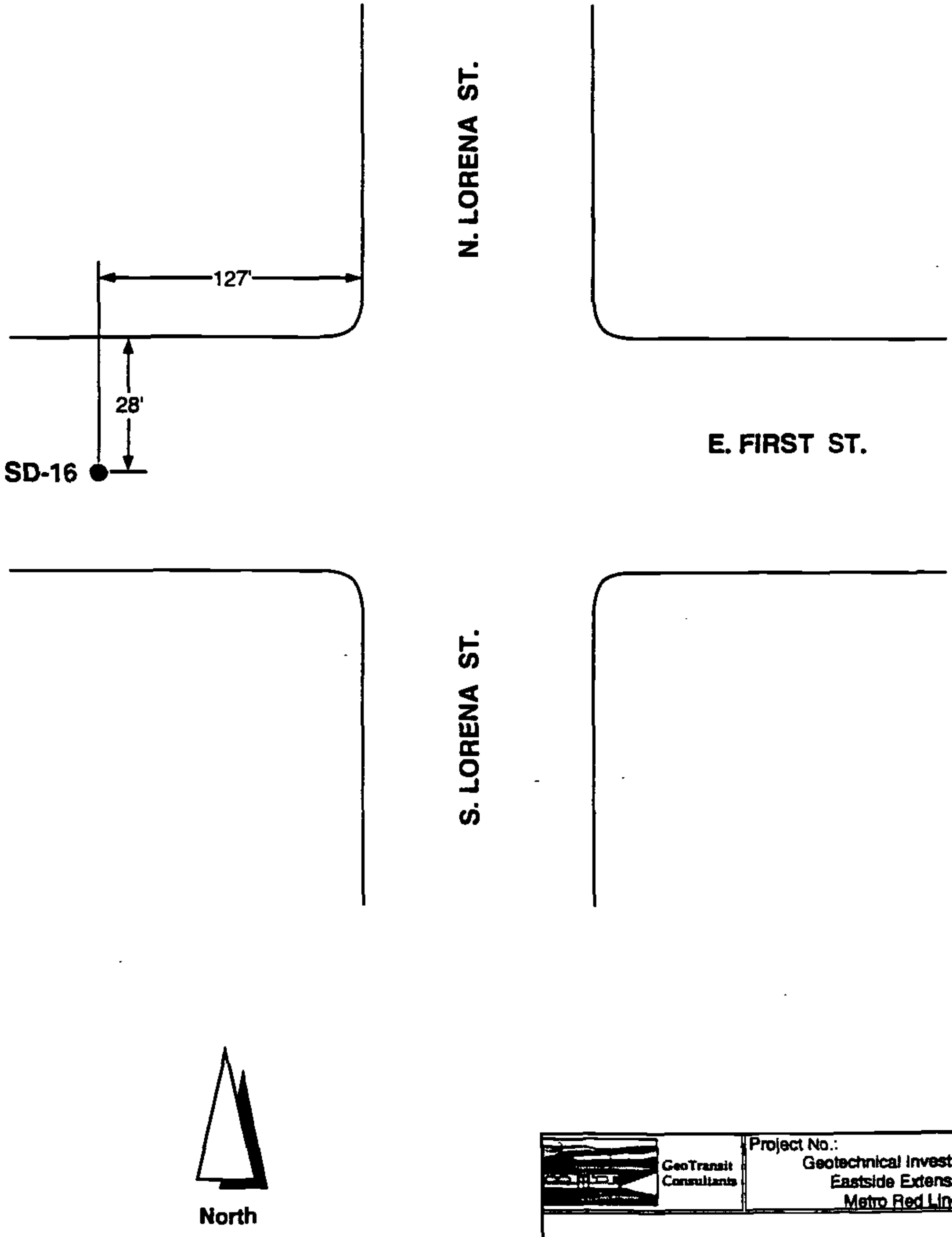
Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

GeoTransit Consultants

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**		
Project Number:	**95-8347**	Boring Number: **SD-16**	Sheet 5 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Samples Blow Count (no./6 inches)	Samples Recovery (inches/inches)	Samples OVA (ppm) [Background]	Samples Dry Density (pcf)	Samples Moisture Content (%)	Drilling Time
	▨	CLAYEY SAND; see description above	SC	Qa	22	D	18 35	9/12	0 [0]	113	17	13:54
	▨	SANDY LEAN CLAY; dark yellowish brown, moist, hard, low to medium plasticity clay with fine-grained sand, interlayered	CL									
	▨				23	D	20 30	11/12	0 [0]	118	16	06:52
115	▨				24	T		32/36	0 [0]			07:58
	░	SILTY SAND; brown, moist, dense to very dense, fine-grained sand with silt	SM									
120	░				25	S	19 32 32	15/18	0 [0]			08:23
	░				26	D	19 28	9/12	0 [0]			08:40
	░				27	T		16/36	0 [0]			
125	▤	FERNANDO/PUENTE FORMATION CLAYSTONE; yellowish, brown, gray, moist, very soft bedrock, poorly laminated, manganase stain, low to medium plasticity clay, some fine-grained sand, white shell fragments up to coarse sand size (no reaction with HCl)	CL	Tf/Tp								09:13
	▤				28	T		16/36	0 [0]			09:40
	▤	medium-grained sand										
130		End of boring at 130 feet.										
135												
140												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



Not to Scale

GeoTransit Consultants

Project No.: 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
SD-16

12-95

Figure A-4

GeoTransit Consultants

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN				
Project Number:	95-8347	Boring Number:	SD-17	Sheet 1 of 4	
Boring Location:	On 1st St., 130' E of Lorena	Elevation and Datum(feet):	294.0		
Health and Safety:	Level D	Date Started:	8/25/95	Date Finished:	8/25/95
Drilling Equipment:	Mayhew 1000	Total Depth (feet):	106.0	Depth to Bedrock(feet):	99.0
Drilling Method:	Rotary Wash	Number of Samples:	21	Depth to Water (feet):	Unknown
Boring Diameter:	5 inches	Completion Information:	Grouted to surface		
Hammer Information: SPT Hammer: 140-lb and 30-inch drop Downhole Hammer: 400-lb and 18-inch drop		Logged By: C. Mashall Payne		Checked By: Grant Miller	

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
	■	ASPHALT										09:15
	▨	ARTICIAL FILL CLAY with SAND; yellowish red, slightly moist, stiff, medium plasticity clay with sand	CL	af								
5	▨				1	D	3 3	12/12	20 [5.2]	117	13	09:20
	░	ALLUVIUM SILTY GRAVEL with SAND; reddish brown, moist, loose, gravel to 1/2" in size, subangular, fine- to medium-grained sand	GM	Qa								
10	░				2	S	3 4 5	3/18	4.3 [4.4]			09:40
	░	mostly gravel (GP)										
15	▨	SANDY LEAN CLAY/LEAN CLAY; reddish brown, moist, soft, medium plasticity clay, some fine- to medium-grained sand	CL		3	D	2 2	12/12	5.0 [4.8]	107	19	09:45
	▨	pieces of wood										

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: SD-17	Sheet 2 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SANDY LEAN CLAY; reddish brown, moist, stiff, medium plasticity clay with fine- to medium-grained sand	CL	Qa	4	S	2 2 2	4/12	6.0 [4.5]			09:55
		SAND with SILT; reddish brown, moist, medium dense, fine- to medium-grained sand, some 1/4" to 3/4" size gravel, subangular, some silt	SP-SM									
25					5	D	14 20	12/12	5.6 [5.0]	125	9	10:00
30		SILTY SAND; yellowish brown, moist, very dense, fine- to medium-grained sand with silt, some gravel	SM		6	S	30 35 50	10/18	4.8 [4.4]			10:05
		[drill chatter, slow drilling], cobbles, cobble layer estimated to be 1 to 2' thick										
35					7	D	35 60	12/12	5.0 [4.8]	113	13	10:15
		SANDY SILT; yellowish and reddish brown, moist, hard, silt with fine- to medium-grained sand, micaceous, trace gravel, some iron-oxide stains	ML									
40					8	S	11 21 38	18/18	5.2 [4.4]		29	10:25
45		increased sand content, fine grained, SILTY SAND to SANDY SILT (SM/ML)			9	D	12 16	12/12	5.2 [4.8]	103	17	10:35

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

GeoTransit Consultants

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**		
Project Number:	**95-8347**	Boring Number: **SD-17**	Sheet 3 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Samples Blow Count (no./6 inches)	Samples Recovery (inches/inches)	Samples OVA (ppm) [Background]	Samples Dry Density (pcf)	Samples Moisture Content(%)	Drilling Time
		SILTY SAND/SANDY SILT; yellowish to reddish brown, moist, dense/hard, fine-grained sand with silt, micaceous, trace gravel	SM/ML	Qa	10	S	14 17 18	8/18	4.8 [4.4]			10:45
55		SAND with SILT; slight reddish brown, moist, dense to very dense, fine- to medium-grained sand, some coarse sand, some silt	SP-SM		11	D	35 75	12/12	4.7 [4.8]	113	10	11:00
		GRAVEL/SAND	GP/SP									
60		SANDY SILT; yellowish brown, moist, hard, silt with fine-grained sand, some medium-grained sand, micaceous, trace gravel up to 3/4" in size, subangular	ML		12	S	18 22 30	12/18	4.6 [4.4]			11:15
65		SAND/GRAVEL; yellowish brown, wet, dense; interlayered fine- to medium-grained sand and gravel (6" to 1' thick layers), subangular gravel to 3/4" in size	SP/GP		13	D	45 80	12/12	5.2 [4.8]	131	11	11:30
70		SAND with SILT and GRAVEL; yellowish brown, moist, dense to very dense, fine- to medium-grained sand, some gravel to 3/4" in size, some silt	SW-SM		14	S	30 50 55	8/18	4.3 [4.1]			11:40
75		decreased silt content, fine- to medium-grained sand and gravel interlayered fine to medium-grained sand and gravel			15	D	25 45	12/12	5.2 [4.5]	106	19	11:55
		SILTY SAND; gray brown, moist, very dense, fine-grained sand with silt	SM									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

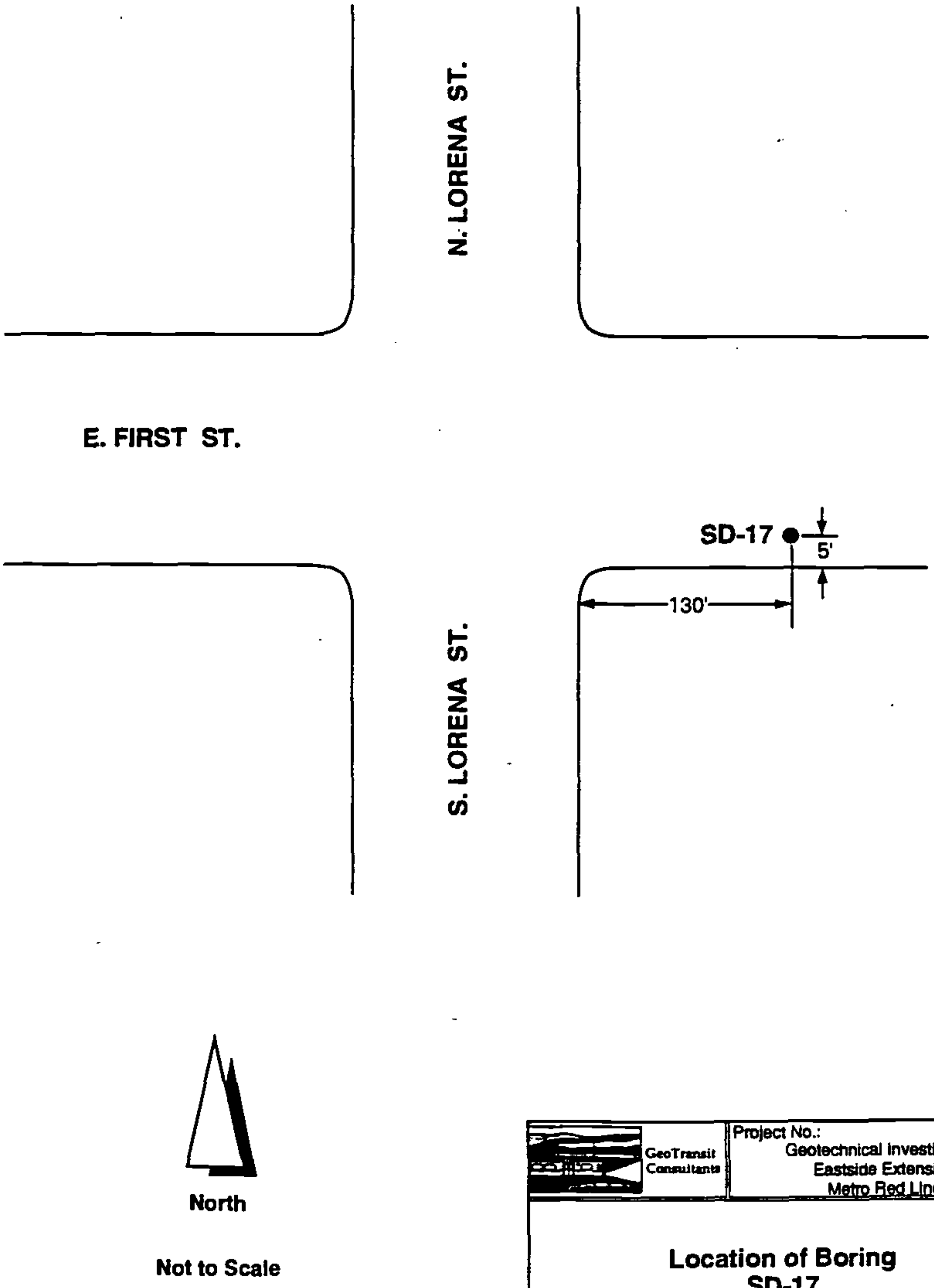
GeoTransit Consultants

Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 **Boring Number:** SD-17 Sheet 4 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SANDY SILT; grayish/yellowish brown, moist, hard, fine-grained sand	ML	Qa	16	S	30 60 65	13/18	5.4 [4.1]			12:00
85		LEAN CLAY; dark reddish brown, slightly moist, very stiff to hard, some pea gravel fragments up to 1/2" in size, some fine-grained sand	CL		17	D	16 23	12/12	5.8 [4.4]	110	19	12:20
90		dark yellowish brown to dark reddish brown (mottled)			18	S	17 37 45	16/18	4.6 [4.1]			12:30
95		SILTY SAND; yellowish brown, slightly moist, dense, fine-grained sand with silt	SM		19	D	30 45	12/12	4.9 [4.6]	103	23	12:55
100		FERNANDO/PUENTE FORMATION CLAYSTONE; mottled light to medium gray and brown, slightly moist, very soft bedrock, medium plasticity clay, appears chaotic, massive, no structure, some fragmentation	CL	Tf/Tp	20	S	14 20 30	18/18	5.0 [3.9]			13:15
105		dark greenish gray, unweathered, unoxidized, massive, very soft bedrock, micaceous, no structure			21	D	22 28	12/12	52.0 [4.4]	107	20	13:30
		End of boring at 106 feet.										

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



GeoTransit Consultants

Project No.: 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring SD-17

12-95

Figure A-5

GeoTransit Consultants

Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 **Boring Number:** SD-18 **Sheet** 1 **of** 5

Boring Location: 1st/Lorena Street **Elevation and Datum(feet):** 302.0

Health and Safety: Level D **Date Started:** 9/6/95 **Date Finished:** 9/7/95

Drilling Equipment: Mayhew 1000 **Total Depth (feet):** 116.0 **Depth to Bedrock(feet):** None

Drilling Method: Rotary Wash **Number of Samples:** 27 **Depth to Water (feet):** Unknown

Boring Diameter: 5 inches **Completion Information:** Grouted to Surface

Hammer Information:
SPT Hammer: 140-lb and 30-inch drop
Downhole Hammer: 400-lb and 18-inch drop

Logged By: Greg Renfrew **Checked By:** Gamini Weeratunga

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
	■	ASPHALT		af								
	▨	ALLUVIUM SANDY LEAN CLAY; yellowish red, moist, medium stiff to stiff, low plasticity clay with fine-grained sand, some medium-grained sand	CL	Qa	1	B			5.3 [4.4]			10:55
5	▨											
	▨				1	D	4 4	10/12	4.9 [4.4]	114	18	11:05
10	○	1' thick gravel layer; yellowish brown			2	S	2 3 4	8/18	4.7 [4.5]			11:14
15	▨	stiff, trace gravel up to 3/4" in size, subrounded			3	D	7 11	12/12	4.4 [4.3]	111	19	11:20

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **SD-18** Sheet 2 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Samples: Type	Samples: Blow Count (no./6 inches)	Samples: Recovery (inches/inches)	Samples: OVA (ppm) [Background]	Samples: Dry Density (pcf)	Samples: Moisture Content(%)	Drilling Time
		SANDY LEAN CLAY; same as above	CL	Qa	4	S	3 7 14	14/18	4.7 [4.5]			11:24
		SILTY SAND; yellowish brown, moist to wet, dense, fine- to medium- grained sand with silt, pockets of fine- to medium-grained sand with fine gravel, subrounded to subangular	SM									
25					5	D	25 45	5/12	4.3 [4.3]	113	15	11:32
		mostly sand, fine- to coarse-grained, some gravel										
30		increased gravel content, very dense, fine- to medium-grained sand, trace coarse-grained sand, fine to coarse gravel up to 3/4" in size, locally well graded			6	S	14 38 35	15/18	4.9 [4.6]			11:43
35		interlayered SAND and SILTY SAND (SP/SM), pockets of GRAVEL to SILTY GRAVEL (GP/GM)			7	D	25 30	12/12	4.7 [4.2]	111	9	11:50
40		coarse gravel up to 2" in size			8	S	12 63 65	12/18	5.1 [4.5]			12:00
45		GRAVEL with SILT; yellowish brown, moist, dense, gravel up to 1-1/2" in size, some silt and fine-grained sand	GP-GM		9	D	35 50	5/12	5.0 [4.5]	130	10	12:14

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GeoTransit Consultants

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number: 95-8347	Boring Number: SD-18		Sheet 3 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	
		SILTY SAND; yellowish brown, moist, very dense, fine-grained sand with silt, trace fine gravel (quartzose), micaceous	SM	Qa	10	S	21 25 40	14/18	6.6 [5.0]			12:29
55					11	D	35 68	8/12	4.6 [4.2]	118	11	12:43
60					12	S	39 90/5"	4/11	5.2 [5.2]			12:52
		increased gravel content (about 20%), fine- to coarse-grained sand, gravel up to 1-1/2" in size										
					13	D	100/6"	3/6	4.5 [4.4]	125	11	13:07
		decreased gravel content, fine- to medium-grained sand, trace fine gravel, subangular, micaceous, mostly fine-grained sand with silt										
65					14	S	35 48 52	15/18	5.3 [5.0]			13:21
70					15	D	25 40	5/12	4.6 [4.6]	100	22	13:36
		gradational contact										
		SANDY SILT; yellowish brown, moist, hard, silt with fine-grained sand, trace medium sand, micaceous, poorly graded	ML									
75					16	S	14 21 29	18/18	5.7 [5.0]			13:45
		SANDY LEAN CLAY; see the description below	CL									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **SD-18** Sheet 4 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	
	▨	SANDY LEAN CLAY; yellowish brown, moist, hard, fine-grained sand with low plasticity clay	CL	Qa	17	D	18 25	11/12	4.5 [4.5]	114	18	
					18	S	37 75	14/18	10.0 [8.3]			09:45
		increased clay content										
	░	SAND/SAND with SILT; yellowish brown, moist, very dense, fine- to medium-grained sand, some silt, poorly graded, micaceous	SP/SM		19	D	40 65	10/12	7.7 [7.2]	114	18	
85												10:01
90		increased silt content, SILTY SAND (SM)			20	S	90 10/.1"	6/6.1	8.7 [8.1]			10:08
	▨	SANDY LEAN CLAY; yellowish brown, slightly moist, hard, low plasticity clay with fine-grained sand, micaceous, mottled with darker stain, carbonaceous stain	CL		21	D	25 28	12/12	7.7 [7.5]	113	19	10:26
95												
		increased clay content, mostly clay, some fine-grained sand, trace medium-grained sand			22	S	25 24 33	18/18	8.2 [7.9]			10:43
100												
					23	D	17 25	12/12	7.8 [7.6]	105	24	11:00
105	░	SAND/SILTY SAND; yellowish brown, moist, medium dense, fine-grained sand, some silt, micaceous	SP/SM									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

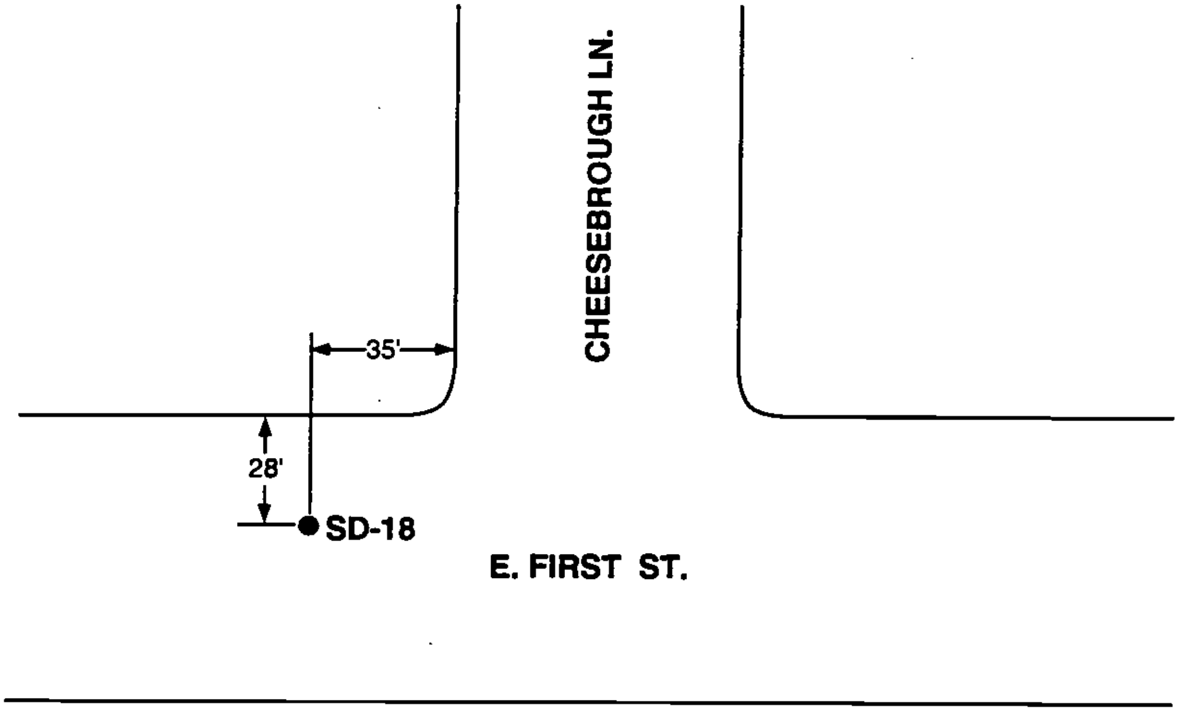
GeoTransit Consultants

Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 **Boring Number:** SD-18 **Sheet** 5 **of** 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	Drilling Time
	⬚	SAND with SILT; see description above	SP/SM	Qa	24	S	18 25 26	18/18	8.3 [7.8]			11:18
	▨	LEAN CLAY/SILT; light olive brown, moist, hard, low plasticty clay and silt, some fine-grained sand, laminated, trace of yellowish brown stain	CL/ML									
	▥	SANDY SILT/SILTY SAND; olive gray, moist, very stiff/medium dense, silt with fine-grained sand, 1" thick high plasticity clay layer	ML/SM		25	D	14 18	7/12	7.7 [7.6]			11:35
115	▥				26	D	16 19	6/12	7.8 [7.6]			11:45
		End of boring at 116 feet.										
120												
125												
130												
135												
140												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



North

Not to Scale

GeoTransit Consultants

Project No.: 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
SD-18

12-95

Figure A-6

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **DD-27S** Sheet 1 of 5

Boring Location: **1st/Indiana (3' E of DD-27)** Elevation and Datum(feet): **319.0**

Health and Safety: **Level D** Date Started: **9/15/95** Date Finished: **9/15/95**

Drilling Equipment: **Mayhew 1000** Total Depth (feet): **127.0** Depth to Bedrock(feet): **None**

Drilling Method: **Rotary Wash** Number of Samples: **21** Depth to Water (feet): **97.3**

Boring Diameter: **5 inches** Completion Information: **Installed Monitoring Well**

Hammer Information:
SPT Hammer: 140-lb and 30-inch drop
Downhole Hammer: 400-lb and 18-inch drop

Logged By: **Greg Renfrew**

Checked By: **Gamini Weeratunga**

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		ASPHALT										
		CLAYEY GRAVEL with SAND	GC	af								
		SANDY LEAN CLAY/SANDY SILT; strong brown	ML/CL	Qa								
5												
		SILTY SAND with GRAVEL; strong brown, fine- to medium-grained sand with silt	SM									
10												
15												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-27S	Sheet 2 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SAND/SILTY SAND; strong brown, fine- to medium-grained sand	SP/SM									
25												
		SANDY LEAN CLAY; strong brown, low plasticity clay with fine- to medium-grained sand	CL									
30												
35												
					18	P		18/36	4.3 [3.9]	97	20	10:09
40												10:21
		SILTY SAND; yellowish brown, fine- to medium-grained sand with silt	SM									
45												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**
Project Number:	**95-8347**
Boring Number:	**DD-27S**
Sheet	3 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Samples Blow Count (no./6 inches)	Samples Recovery (inches/inches)	Samples OVA (ppm) [Background]	Samples Dry Density (pcf)	Samples Moisture Content (%)	Drilling Time
		SILTY SAND; yellowish brown, fine- to medium-grained sand with silt	SM	Qa								
55												
60		SANDY SILT; yellowish brown to light gray, moist, hard, silt with fine-grained sand, micaceous	SM/ML		19	S	31 52 40	16/18	4.8 [4.2]			10:36
					20	D	25 31	8/12	4.3 [3.9]	97	20	11:07
					6	P						
65		layer of clay										
		SILTY SAND; yellowish brown, slightly moist, very dense, fine- to medium-grained sand with silt, iron-oxide staining	SM		21	S	26 59 41/4"	10/16	0.8 [.5]			12:24
		SANDY SILT; yellowish brown, moist, hard, silt with fine-grained sand	SM/ML		22	D	40 44	6/12	4.4 [4.0]	110	12	12:38
					7	P						
70												
					23	S	24 48 51	14/18	4.6 [4.3]			14:00
75		SILTY SAND with GRAVEL; yellowish brown, moist, dense, fine- to coarse-grained sand with silt, some fine gravel, trace coarse gravel up to 1-1/2" in size, subangular to subrounded	SM		24	D	24 44	9/12	4.1 [4.0]	116	11	14:14
					8	P						
					25	S	30/54	10/16	.9			

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number: 95-8347	Boring Number: DD-27S		Sheet 4 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SILTY SAND with GRAVEL; see the description above	SM	Qa			58/4"		[.4]			15:30
		increased sand content, silty sand; light olive brown, micaceous			26	D	36 46		4.6 [4.5]	116	12	
					9	P						15:45
85		increased gravel content, yellowish brown, gravel up to 1-1/4" in size, subangular, quartzose clasts			27	S	61 100/5"	9/11	.7 [3]			
		SILTY SAND; strong brown, slightly moist, dense to very dense, fine- to medium-grained sand with silt, iron-oxide stain, trace fine gravel, subangular to subrounded	SM		28	D	25 36	6/12	4.3 [4.2]	117	10	17:00 08:15
					10	P						
90												
					29	S	29 52 88	16/18	5.3 [4.3]			10:14
					30	D	34 40	8/12	4.1 [4.0]	114	11	10:26
95		light yellowish brown, pockets of fine- to medium-grained SAND (SP/SM), micaceous			11	P						
▼												
100		iron-oxide stained, trace fine gravel, 1" thick layer of SANDY SILT (ML)			31	D	20 25	8/12	5.0 [4.6]	100	25	11:45
					32	S	25 10 29	18/18	5 [5]			11:53
					33	D	20 26	8/12	4.3 [4.4]	105	22	12:12
		SANDY SILT; olive brown, moist, hard, silt with fine-grained sand, micaceous	SM/ML									
105												
		SANDY LEAN CLAY; dark yellowish brown to grayish brown, mottled, moist, hard, low to medium plasticity clay with fine-grained sand, trace coarse-grained sand, subrounded to rounded	CL		34	D	15 20	9/12		105	22	13:12
					12	P						

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

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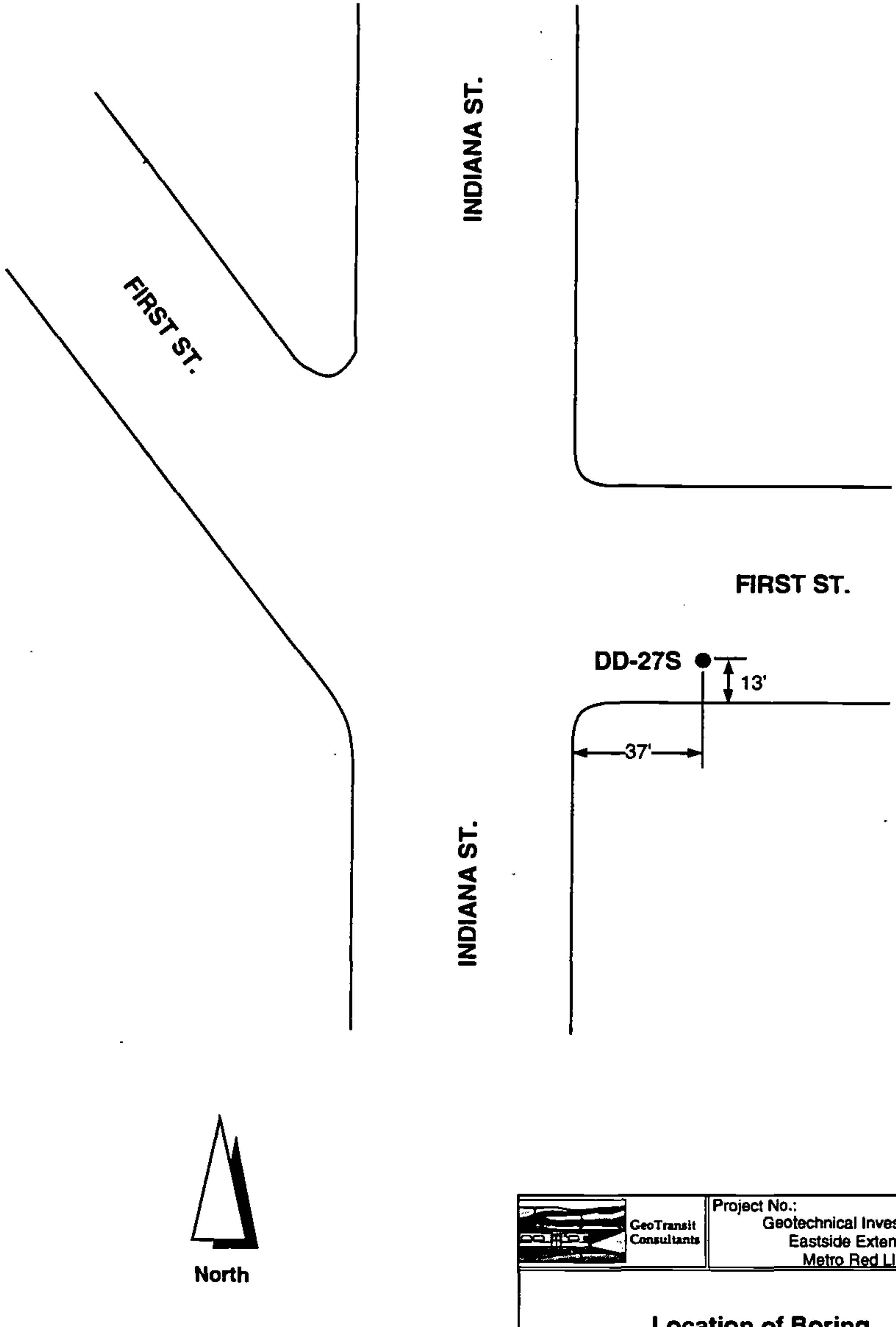
Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-27S	Sheet 5 of 5

Depth (feet)	Lithology	Description	USCS Classi fication	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
	▨	SANDY LEAN CLAY; dark yellowish brown to grayish brown mottled, moist, hard, low to medium plasticity clay with fine-grained sand	CL	Qa	35	S	34 100/4"	10/10	14.0 [4.5]			14:53
115		SAND/SILTY SAND with GRAVEL; grayish brown, moist, dense to very dense, fine- to coarse-grained sand, some silt, gravel up to 5/8" in size, rounded to subrounded, 8" thick layer of clay	SP/SM		36	D	25 30	9/12		104	22	15:01
		decreased gravel content, trace pea gravel, subangular, iron-oxide stain, micaceous										
120	▨	SANDY LEAN CLAY; strong brown, moist, hard, low plasticity clay with fine-grained sand	CL		37	S	18 35 40	18/18	5.0 [3.8]			15:22
125	▨	increased clay content, SANDY LEAN CLAY (CL)			38	D	15 31	8/12	6.0 [4.0]	117	16	15:43
130		End of boring at 127 feet. Groundwater was measured at a depth of 97.3 feet. Note: Soil description in the top 60 feet in Boring DD-27S was based on examination of cuttings. Refer to Boring DD-27 (which is 3 feet east of Boring DD-27S) for detailed description of soils in the top 60 feet.										
135												
140												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



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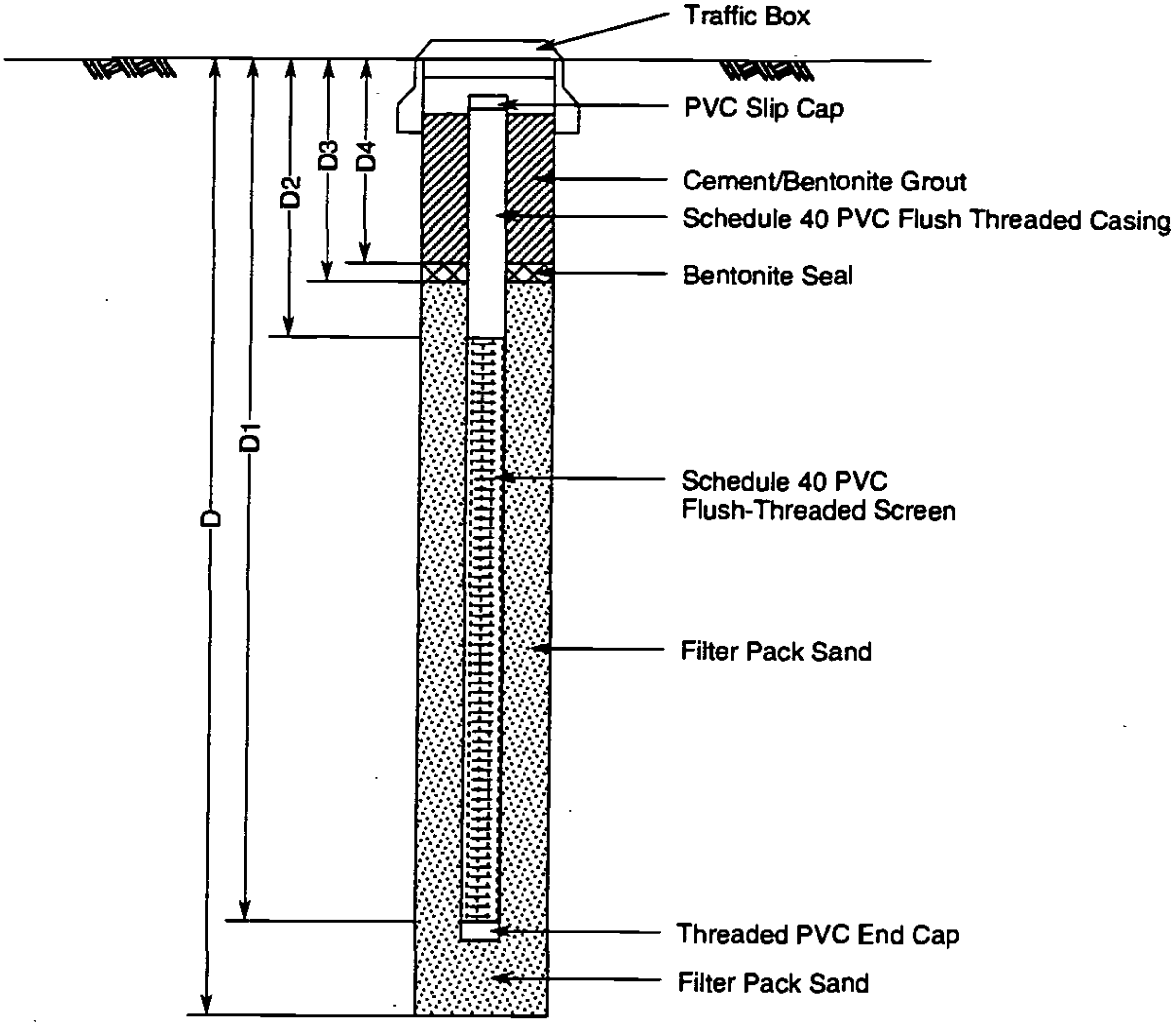
Project No.: 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
DD-27S

12-95

Figure A-7A



Total Depth (D) = 127 Feet
Total Depth of Casing (D1) = 125 Feet
Depth to Top of Well Screen (D2) = 45 Feet
Depth to Bottom of Top Seal (D3) = 42 Feet
Depth to Top of Top Seal (D4) = 39 Feet
Well Casing Diameter = 2 Inches
Well Screen Slot Size = 0.01 Inch
Filter Pack Sand Type = #2/12 Monterey
Bentonite Seal Type = 1/2 Inch Pellets

GeoTransit Consultants

Project No. 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Monitoring Well Installation Schematic
DD-27S

12-95

Figure A-7B

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**		
Project Number: **95-8347**	Boring Number: **DD-27**		Sheet 1 of 3
Boring Location:	**On 1st St., 40' E of Indiana**	Elevation and Datum(feet): **318.0**	
Health and Safety:	**Level D**	Date Started: **9/13/95**	Date Finished: **9/14/95**
Drilling Equipment:	**Mayhew 1000**	Total Depth (feet): **62.5**	Depth to Bedrock(feet): **None**
Drilling Method:	**Rotary Wash**	Number of Samples: **17**	Depth to Water (feet): **Unknown**
Boring Diameter:	**5 inches**	Completion Information: **Grouted to surface**	
Hammer Information: **SPT Hammer: 140-lb and 30-inch drop** **Downhole Hammer: 400-lb and 18-inch drop**		Logged By: **Greg Renfrew**	Checked By: **Gamini Weeratunga**

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		Asphalt and Concrete										
		ARTIFICIAL FILL CLAYEY GRAVEL with SAND; reddish, yellow, moist, dense, fine to coarse gravel up to 1-1/4" in size, subangular, fine- to coarse-grained sand	GC	af		B			1.0 [0.5]			10:05
5		ALLUVIUM SANDY LEAN CLAY/SANDY SILT; reddish yellow, moist, very stiff, low to medium plasticity silt and clay with fine- to coarse-grained sand, trace fine gravel, subangular to angular	CL/ML	Qa								
		gradational contact SILTY SAND; strong brown, moist, medium dense to dense, fine- to medium-grained sand with silt, some fine to coarse gravel up to 3/4" in size, subangular	SM									
10					1	D	13 15	6/12	.9 [.7]	109	10	10:30
15		dense, fine- to coarse-grained sand with gravel up to 1-1/4" in size, trace coarse-grained sand			2	S	15 16 17	11/18	0.6 [.5]			10:38

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 | **Boring Number:** DD-27 | **Sheet** 2 **of** 3

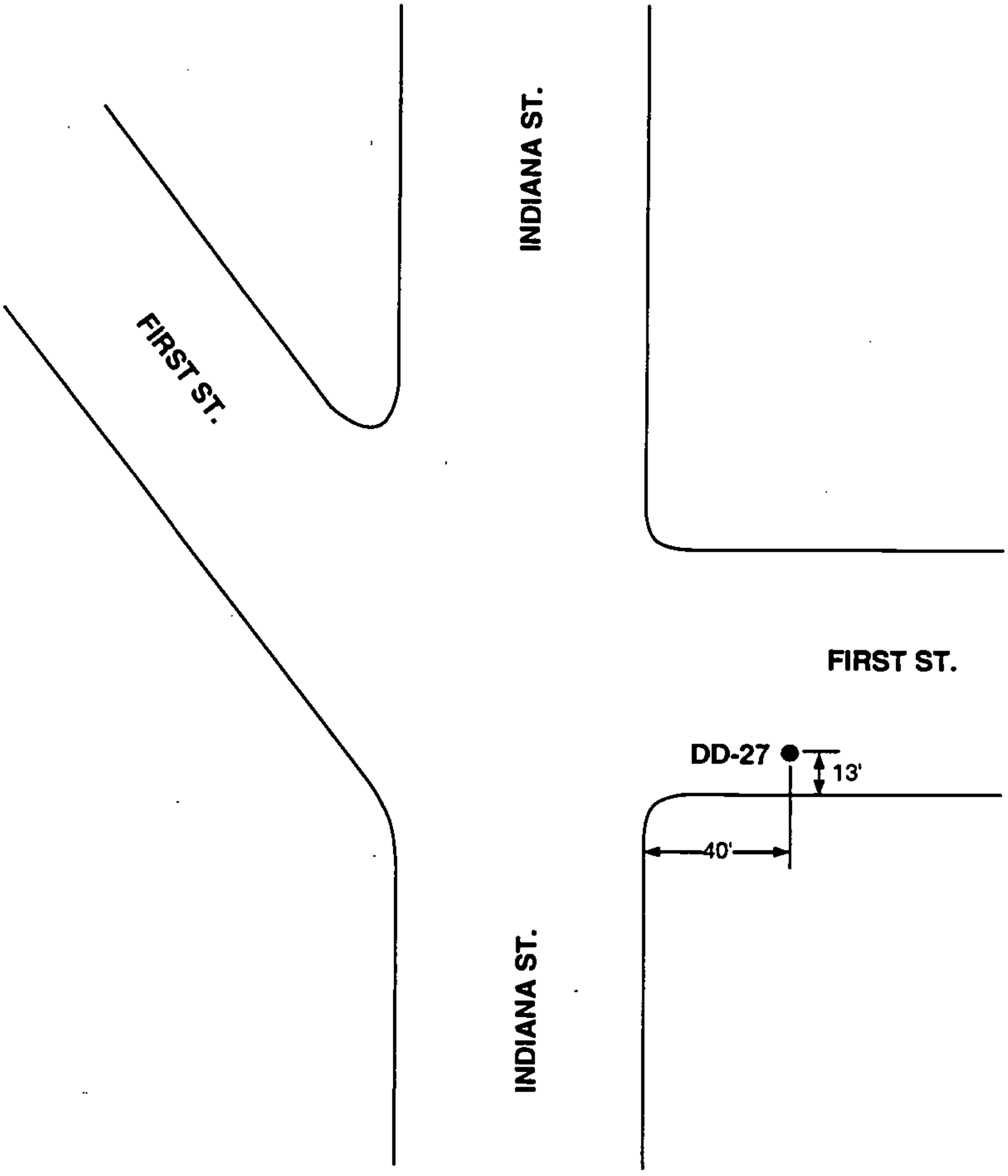
Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SAND/SILTY SAND; strong brown, moist, dense to very dense, fine- to coarse-grained sand, some silt, some gravel up to 1-1/2" in size, subangular	SP/SM	Qa	3	D	35 36	6/12	1.1 [.5]			10:41
					4	S	14 20 30	7/8		114	9	10:47
					5	D	19 31	6/12	1.2 [.8]			11:00
					1	P						
		fine- to medium-grained sand, layer of fine gravel										
25												
									1.0 [.5]			
		increasing gravel content, mostly GRAVEL (GP)			6	D	10 14	6/12		109	19	12:58
		SAND/SILTY SAND; olive brown, moist, dense, fine- to medium-grained sand, trace gravel up to 3/4" in size, subrounded, micaceous	SP/SM		7	S	4 11 20	10/18	1.0 [1.8]			
30												1:02
		layer of CLAYEY SAND (SC)			8	D	11 11	8/12	.4 [.3]	114	12	
35		LEAN CLAY/SANDY LEAN CLAY; strong brown, moist, very stiff, medium plasticity clay with fine- to medium-grained sand	CL		9	S	19 13 14	16/18	.4 [.3]			1:05 1:09
					2	P						
		increased sand content, SANDY LEAN CLAY										
40												
					10	S	6 10 11	16/18	.5 [.5]			2:39
					11	D	10 15	10/12	1.0 [1.0]	107	20	2:46
		SILTY SAND; dark yellowish brown, moist, dense, fine- to medium-grained sand with silt, layers of sandy clay	SM		3	P						
45												
		SANDY LEAN CLAY; dark yellowish, moist, hard, fine- to medium-grained sand with low plasticity clay, micaceous	CL/SC		12	S	8 22 23	7/18	.5 [.2]			4:15
		some silt			13	D	14 19	7/12	.5 [.2]	114	14	
					4	P						4:28

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, whare available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-27	Sheet 3 of 3

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SILTY SAND; yellowish, brown, moist, dense, fine- to medium-grained sand with silt, some clay	SM	Qa								
		silt, trace gravel up to 1" in size, subangular, trace coarse-grained sand			14	S	35 52 25	12/18	.8 [.2]			7:15
55					15	D	15 22	8/12	4.1 [4.0]	113	12	7:29
					5	P						
60		SILTY SAND/SANDY SILT; yellowish brown, moist, dense/hard, fine-grained sand and silt	SM/ML		16	S	29 39 49	2/18				8:54
					17	D	32 57	0/12				
		End of boring at 62.5 feet. (Boring abondoned due to entrapment of a steel ring. See boring DD-27S for depths below 60 feet.)										
65												
70												
75												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



North

Not to Scale

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Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
DD-27

12-95

Figure A-8

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Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number: 95-8347	Boring Number: DD-28		Sheet 1 of 5
Boring Location:	On Alma, 328' S of First Street	Elevation and Datum(feet): 320.0	
Health and Safety:	Level D	Date Started: 9/12/95	Date Finished: 9/12/95
Drilling Equipment:	Mayhew 1000	Total Depth (feet): 131.0	Depth to Bedrock(feet): None
Drilling Method:	Rotary Wash	Number of Samples: 32	Depth to Water (feet): Unknown
Boring Diameter:	5 inches	Completion Information: Grouted to Surface	
Hammer Information: SPT Hammer: 140-lb and 30-inch drop Downhole Hammer: 400-lb and 18-inch drop		Logged By: Greg Renfrew	Checked By: Gamini Weeratunga

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		ASPHALT										
		FILL GRAVELLY CLAY with SAND; dark brown, moist, very loose	CL	af		B						
5		ALLUVIUM SANDY LEAN CLAY/SANDY SILT; strong brown, moist, hard, low to medium plasticity clay and silt with fine- to medium-grained sand	CL/ML	Qa					6.2 [5.1]			
10		CLAYEY SAND; strong brown, moist, medium dense, fine- to medium-grained sand with low plasticity clay, some coarse-grained sand and fine gravel	SC		1	D	14 16	6/6	5.5 [5.1]	118	13	07:24
15		SILTY SAND; yellowish brown, moist, dense, fine- to coarse-grained sand with silt, some fine gravel	SM		2	S	5 22 26	14/18	5.7 [5.4]			07:31
		CLAYEY SAND; yellowish brown, moist, fine-grained sand with low plasticy clay, trace medium-grained sand	SC									

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-28	Sheet 2 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SANDY LEAN CLAY; yellowish brown, moist, very stiff, low plasticity clay with fine-grained sand, trace medium-grained sand	CL	Qa	3	D	9 11	9/12	5.5 [5.2]	103	16	07:35
25		6" thick layer of SILT (ML); dark yellowish brown, dark mottled stain			4	S	12 10 15	14/18	6.9 [5.2]			07:40
		SILTY SAND; dark yellowish brown, moist, medium dense, fine-grained sand with silt, dark mottled stain	SM									
30					5	D	11 16	8/12	5.4 [5.2]	101	14	07:52
		increased sand content										
35		SAND/SILTY SAND; yellowish brown, moist, dense, fine-grained sand, some silt, quartzose	SP/SM		6	S	14 17 21	14/18	6.1 [5.1]			08:00
40		SANDY LEAN CLAY; strong brown, moist, hard, low plasticity clay with fine-grained sand, some medium to coarse-grained sand, some silt	CL		7	D	12 16	8/12	5.3 [5.0]	115	16	08:00
45		1' thick layer of SILTY SAND (SM); yellowish brown, medium dense, fine grained			8	S	7 16 16	13/18	6.4 [5.6]			08:11

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-28	Sheet 3 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SANDY LEAN CLAY; yellowish brown, moist, very stiff, fine-grained sand with low plasticity clay, layers of CLAYEY SAND (SC), some silt, micaceous, trace coarse gravel up to 2" in size, subrounded to subangular	CL	Qa	9	D	8 14	8/12	6.0 [5.1]	97	26	08:20
		SILTY SAND with GRAVEL; yellowish brown, moist, very dense, fine- to medium-grained sand with fine to coarse gravel and silt, granitic gravel up to 2" in size, subangular	SM		10	S	21 47 47	16/18	6.8 [5.6]			08:25
55												
60		decreased silt and gravel content, SAND with SILT (SP-SM)			11	D	30 40	8/12	5.1 [5.0]	125	8	08:46
		SANDY LEAN CLAY; dark yellowish brown, moist, hard, low plasticity clay with fine-grained sand, some silt, micaceous, 3" thick LEAN CLAY layer, iron-oxide stain	CL		12	S	11 25 27	14/18	7.6 [5.9]			08:58
65												
		CLAYEY SAND; yellowish brown, moist, medium dense to dense, fine- to medium-grained sand with low plasticity clay, some silt, pockets of poorly graded sand	SC		13	D	20 38	8/12	5.1 [5.0]	107	20	09:06
70		iron-oxide stains, increased clay content			14	S	25 44 32	9/18	6.0 [6.0]			09:27
		SANDY SILT; olive gray moist, hard, silt with fine-grained sand, a layer of fine gravel, layer of silty sand	ML		15	D	14 25	7/12	6.0 [5.9]	87	33	09:38
75												
					16	S	16 29 33	12/18	7.5 [5.9]			09:45
		a layer of SANDY SILT/SILTY SAND (ML/SM), silt and fine-grained sand										
					17	D	15 21	7/12	5.8 [5.2]	89	29	10:00

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

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Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-28	Sheet 4 of 5

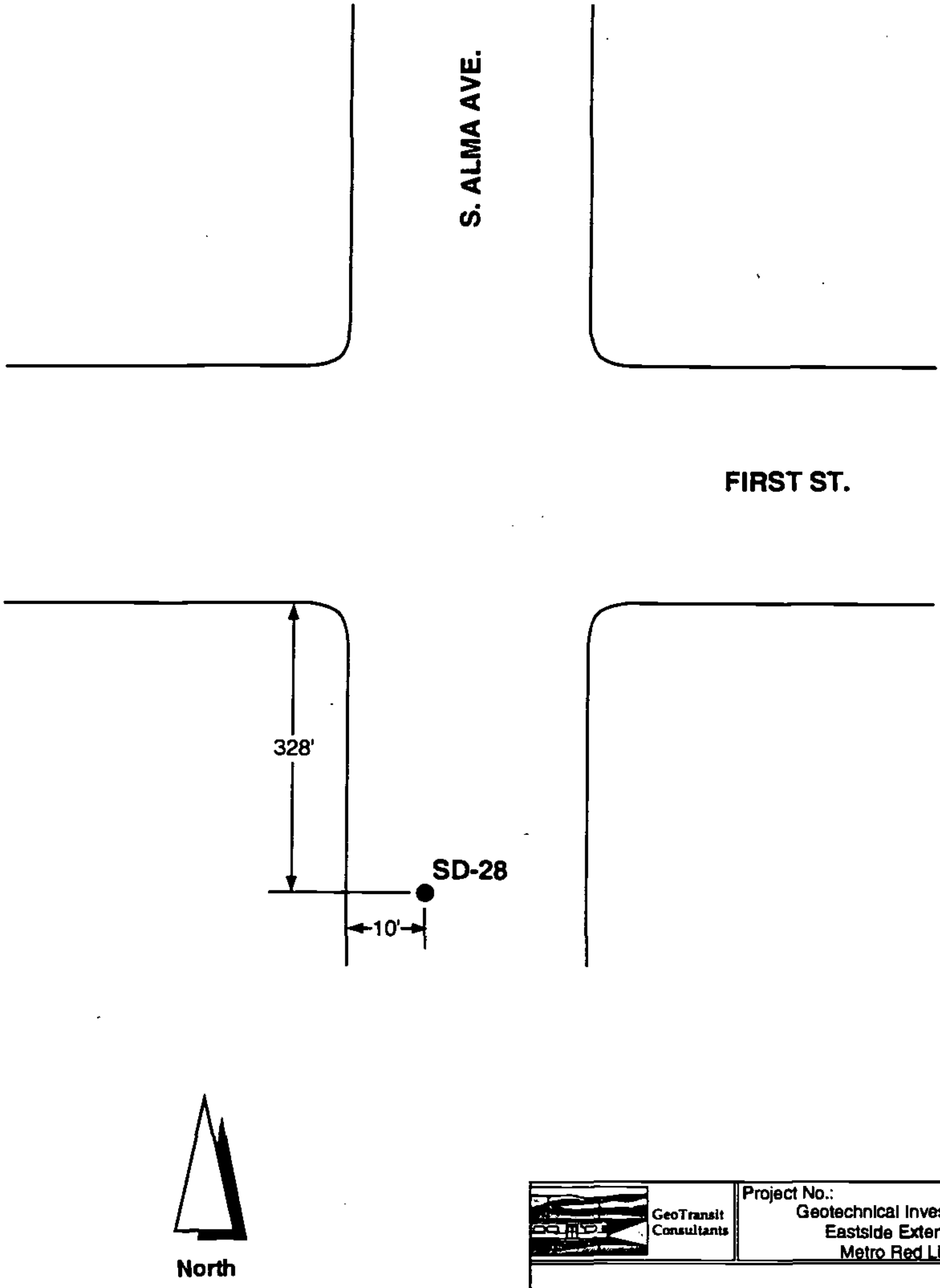
Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SILTY SAND; yellowish brown, moist, dense, fine-grained sand with silt	SM	Qa								
					18	S	29 53 55	13/18	5.5 [5.2]			10:05
		SAND; yellowish brown, slightly moist, very dense, fine- to medium-grained sand, trace silt	SP									
85					19	D	34 46	6/12	5.3 [5.0]	103	8	10:20
		increased silt content, SANDY SILT (ML); dark yellowish brown, silt with fine-grained sand, micaceous										
					20	S	20 38 38	13/18	6.2 [5.2]			10:30
90												
		SILTY SAND; light yellowish brown, moist, dense, fine-grained sand with silt	SM		21	D	30 49	5/12	5.7 [5.0]	105	13	10:40
		SANDY SILT; light yellowish brown, moist, hard, silt with fine-grained sand	ML		22	S	19 36 31	14/18	6.7 [5.2]			11:00
95												
					23	D	25 40	5/12	6.2 [5.0]	108	19	11:17
100		3" layer of SILT (ML), micaceous, pockets of silty sand, fine- to medium-grained sand			24	S	21 28 46	15/18	7.6 [5.8]			11:30
105					25	D	23 27	8/12	5.2 [5.1]	104	24	11:45
		layers of fine- to medium-grained SAND (SP/SM); dense			26	S	17 40 39	13/18	6.8 [5.2]			11:56

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**	
Project Number: **95-8347**	Boring Number: **DD-28**	Sheet 5 of 5

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SILTY SAND; dark yellowish brown, slightly moist, dense to very dense, fine-grained sand with silt, micaceous	SM	Qa	27	D	30 43	6/12	5.7 [5.0]	102	22	12:15
115		pockets of SAND (SP), fine to medium grained			28	S	28 47 61/4"	14/16	5.0 [4.0]			12:30
120		SANDY LEAN CLAY; strong brown, moist, hard, low plasticity clay with fine-grained sand, some medium- and coarse-grained sand	CL		29	D	24 32	8/12	5.3 [5.0]	121	14	12:50
125					30	S	42 75	9/8.5	6.1 [5.1]			13:07
		CLAYEY SAND/SILTY SAND; dark brown, moist, very dense, interlayered silty sand and clayey sand, fine-grained sand with low plasticity clay and silt	SC/SM									
130		wet			31	D	40 50	6/12	5.8 [5.0]	108	22	13:28
		End of boring at 131 feet.										
135												
140												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



Not to Scale

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Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring SD-28

12-95

Figure A-9

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Project Name: METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN

Project Number: 95-8347 **Boring Number:** DD-62 **Sheet** 1 **of** 4

Boring Location: On Velasco St., 37.4' S of 1st St. **Elevation and Datum(feet):** 312.0

Health and Safety: Level D **Date Started:** 11/16/95 **Date Finished:** 11/16/95

Drilling Equipment: Mayhew 1000 **Total Depth (feet):** 100.5 **Depth to Bedrock(feet):** None

Drilling Method: Rotary Wash **Number of Samples:** 28 **Depth to Water (feet):** Unknown

Boring Diameter: 5 inches **Completion Information:** Grouted to Surface

Hammer Information:
SPT Hammer: 140-lb and 30-inch drop
Downhole Hammer: 400-lb and 18-inch drop

Logged By: Greg Renfrew **Checked By:** Gamini Weeratunga

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		ASPHALT										
		ALLUVIUM CLAYEY SAND; yellowish red, moist, medium dense, trace subangular gravel up to 3/4" in size	SC	Qa								
5		SILTY SAND/SILTY GRAVEL; yellowish brown, moist, medium dense, to dense, fine- to coarse-grained sand, and fine gravel about 1/2" in size, subangular	SM/GM		1	B			9.7 [9.2]			
10		CLAYEY SAND; dark yellowish brown, moist, loose, fine- to medium-grained sand with low plastcity clay, some silt, trace fine gravel	SC		2	D	5 5	12/12	9.8 [9.8]			08:43
15		4" thick layer of fine- to coarse-grained sand, dense			3	S	15 19 12	13/18	13.0 [9.7]			08:51

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Project Name:	**METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**		
Project Number:	**95-8347**	**Boring Number:** **DD-62**	Sheet 2 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Samples Type	Samples Blow Count (no./6 inches)	Samples Recovery (inches/inches)	Samples OVA (ppm) [Background]	Samples Dry Density (pcf)	Samples Moisture Content (%)	Drilling Time
		SILTY SAND; yellowish brown, moist, medium dense, silt with fine-grained sand, trace medium- to coarse-grained sand gradationasl contact	SM SP-SM	Qa	4	D	12 16	10/12	13.0 [9.7]			08:57
		SAND with SILT; yellowish brown, moist, medium dense, fine-grained sand, some silt, trace coarse-grained sand										
25		LEAN CLAY; dark yellowish brown, moist, very stiff, trace fine- to medium-grained sand	CL		5	S	5 11 12	18/18	13.0 [12.0]			09:06
30					6	D	11 16	12/12	14.0 [13.0]			09:09
		SANDY LEAN CLAY/SANDY SILT; dark yellowish brown, moist, hard, low plasticity clay and silt with fine-grained sand, trace medium-grained sand, very dark stains	CL/ML									
35		LEAN CLAY; yellowish brown, moist, very stiff, low to medium plasticity clay, some silt, trace fine-grained sand, corbonous	CL		7	S	7 11 13	18/18	14.0 [13.0]			09:12
		SILTY CLAYEY SAND; dark yellowish brown, moist, medium dense to dense, fine-grained sand with low plasticity clay and silt, some medium-grained sand, black specks (carbonous deposits)	SC-SM		8	D	12 24	12/12	15.0 [14.0]			09:18
40					9	S	10 22 23	18/18	14.0 [14.0]			09:20
		SILTY CLAY; dark yellowish brown, moist, hard, silt and clay with fine-grained sand	CL-ML		10	D	12 18	12/12	14.0 [13.0]			09:31
45												
		SILTY SAND; dark olive brown, moist, very dense, fine-grained sand with silt, some coarse-grained sand, trace subrounded gravel up to 1" in size	SM		11	S	25 100	11/18	15.0 [14.0]			09:38

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: **METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN**

Project Number: **95-8347** Boring Number: **DD-62** Sheet 3 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
		SILTY SAND; see description above	SM	Qa	12	D	25 40	6/12				09:42
		SAND; light yellowish brown, moist, very dense, fine- to medium-grained sand, trace silt	SP									
		SAND with SILT and SILTY GRAVEL; yellowish brown, moist, very dense, interbedded fine- to coarse-grained sand and fine to coarse gravel up to 1-1/2" in size, subangular	SP-SM /GM		13	S	65 50/1"	6/7	16.0 [14.0]			10:02
55												
					14	D	60 100/4"	9/10	17.0 [14.0]			10:12
					15	S	69 82	11/12	15.0 [14.0]			10:26
60		gravel up to 2" in size and medium-grained sand interbeds										
					16	D	100	6/6	15.0 [14.0]			10:36
65												
					17	S	58 62/3"	9/9	14.0 [14.0]			10:46
					18	D	100/5"	4/5	14.0 [14.0]			10:58
		iron-oxide stain										
70		gravel up to 2-1/2" in size (quartz and metamorphic clasts)										
		SILT; olive brown, moist, hard, micaceous, some fine-grained sand, some clay	ML		19	S	12 38 62	18/18	15.0 [14.0]			11:06
		SILTY SAND; olive brown, moist, dense to very dense, fine-grained sand with silt, a layer of gravel, subrounded	SM		20	D	25 30	10/12	14.0 [14.0]			11:15
75												
		increased silt content, SANDY SILT (ML), hard			21	S	23 46 100/4"	11/16	14.0 [14.0]			11:28

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contects indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name:	METRO RED LINE- EASTSIDE EXTENSION, DETAILED DESIGN		
Project Number:	95-8347	Boring Number: DD-62	Sheet 4 of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples							Drilling Time
					Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	
		SILTY SAND; yellowish brown, moist, dense to very dense, fine-grained sand with silt, subrounded, thin layers of sand (SP), thin layers of gravel up to 3/4" in size, subrounded, some iron-oxide stains	SM	Qa	22	D	30 50	10/12	14.0 [14.0]			11:38
					23	S	44 55/3"	9/9	15.0 [14.0]			11:48
85		SAND with SILT and GRAVEL; olive brown, moist, very dense, fine- to coarse-grained sand, some silt, gravel up to 3/4" in size, interlayered	SP-SM		24	D	100	5/6	14.0 [13.0]			12:00
					25	S	100/3"	1/3	15.0 [14.0]			12:14
90		gravel up to 1" in size										
					26	D	100/5"	3/5	14.0 [14.0]			12:27
95		gravel up to 1" in size			27	S	66 40/2"	7/8	14.0 [14.0]			12:42
100		gravel up to 2" in size			28	D	100/5"	3/5	14.0 [14.0]			12:55
		End of Boring at 100.5 feet										
105												

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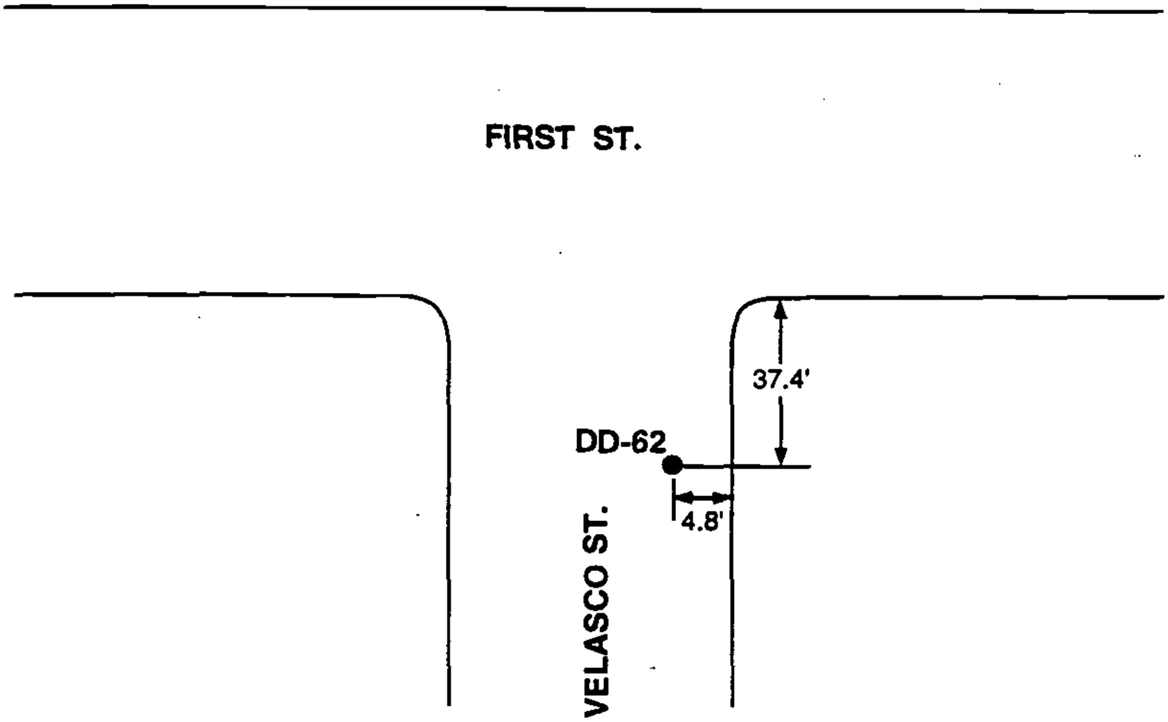

North

Not to Scale

GeoTransit Consultants

Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
DD-62

12-95

Figure A-10

Project Name:	Metro Rail: Eastside Extension - Metro Red Line				
Project Number:	94-1100	Boring Number:	PE-13	Sheet 1 of 4	
Boring Location:	First/Lorena		Elevation and Datum(feet): 292 MSL		
Health and Safety:	Level D	Date Started:	11/17/93	Date Finished:	11/17/93
Drilling Equipment:	Mayhew 1000	Total Depth (feet):	90.5	Depth to Bedrock(feet):	88.5
Drilling Method:	Rotary Wash	Number of Samples:	15	Depth to Water (feet):	Unknown
Boring Diameter:	5 inches	Completion Information:	Installed Monitoring Well		
Hammer Information: SPT Hammer: 140-lb and 30-inch drop Downhole Hammer: 400-lb and 18-inch drop		Logged By: Greg Renfrew		Checked By: Grant Miller	

Depth (feet)	Lithology	Description	USCS Classi fication	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content(%)	Drilling Time
	■	FILL Asphaltic concrete to 8", concrete to 12" and base to 30"		af								
	▨	ALLUVIUM CLAY with SAND; reddish yellow, moist to wet, medium stiff, medium plasticity, fine- to medium-grained sand, trace coarse-grained, trace subangular gravel to 1/4" in size	CL	Qa								
5	▨				1	S	2 3 3	8/18	8.5 [8.0]			09:25
10	▨	SANDY CLAY, yellowish red, low to medium plasticity, fine- to coarse-grained sand, trace angular to subangular gravel to 3/4" in size	CL		2	D	6 7	0/12	8.6 [8.0]		27	09:35
15	▨	dark reddish brown, moist			3	S	1 2 3	12/18	8.1 [8.0]			09:50

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Project Name: Metro Rail: Eastside Extension - Metro Red Line

Project Number: 94-1100 | **Boring Number:** PE-13 | **Sheet** 2 **of** 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples Number	Type	Blow Count (no./6 inches)	Recovery (inches/inches)	OVA (ppm) [Background]	Dry Density (pcf)	Moisture Content (%)	Drilling Time
		CLAYEY SAND; reddish yellow, moist, medium dense, fine- to coarse-grained, medium plasticity clay, trace gravel to 2" in size	SC	Qa	4	D	9 10	0/12	8.5 [8.0]		16	10:00
25		SILTY SAND/SANDY SILT; reddish yellow, moist, medium dense/very stiff, fine-grained sand, nonplastic silt	SM/ML		5	S	6 7 17	<1/18	8.0 [8.0]			10:20
30		SILTY SAND (SM); reddish yellow, moist, very dense, fine- to coarse-grained, with fine-gravel to 3/4" in size	SM		6	D	60	2/6	12 [7.4]	114	16	10:40
35					7	S	38 45 50	8/18	8.2 [8.0]			10:50
		fine-grained, trace gravel to 1/2" in size, micaceous										
40		dense			8	D	50	2/6	14 [7.5]	94	19	11:00
		[strong rig chatter from 44' to 46']										
45		GRAVEL with SILT and SAND/SAND with SILT and GRAVEL; reddish yellow to brownish yellow, moist, very dense, fine- to coarse-grained sand, subangular gravel to 3/4" in size, nonplastic silt	GP-GM / SP-SM		9	S	50	4/6	8.0 [8.0]			11:20

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

Project Name: Metro Rail: Eastside Extension - Metro Red Line

Project Number: 94-1100 | **Boring Number:** PE-13 | **Sheet** 3 **of** 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Samples: Type	Samples: Blow Count (no./6 inches)	Samples: Recovery (inches/inches)	Samples: OVA (ppm) [Background]	Samples: Dry Density (pcf)	Samples: Moisture Content (%)	Drilling Time
		GRAVEL with SILT and SAND/SAND with SILT and GRAVEL; brownish yellow, moist to wet, very dense, fine- to coarse-grained sand, angular gravel to 1" in size, nonplastic silt	GP-GM / SP-SM	Qa								
55					10	D	130	2/3	11 [7.3]	119	13	11:45
60		gravel to 3/4" in size										
65		gravel to 1" in size			11	S	50/3"	2/3	8.5 [8.0]			
70												12:15
75		interval of SILT (ML) interval of SAND with SILT (SP-SM), fine- to medium-grained, trace coarse-grained			12	D	80/ 4.5"	<1/ 4.5	8.1 [8.0]		21	
		[strong chatter from 76.5' to 80'] decreased silt content, GRAVEL with SAND/SAND with GRAVEL (GW/SW), fine- to coarse-grained sand										

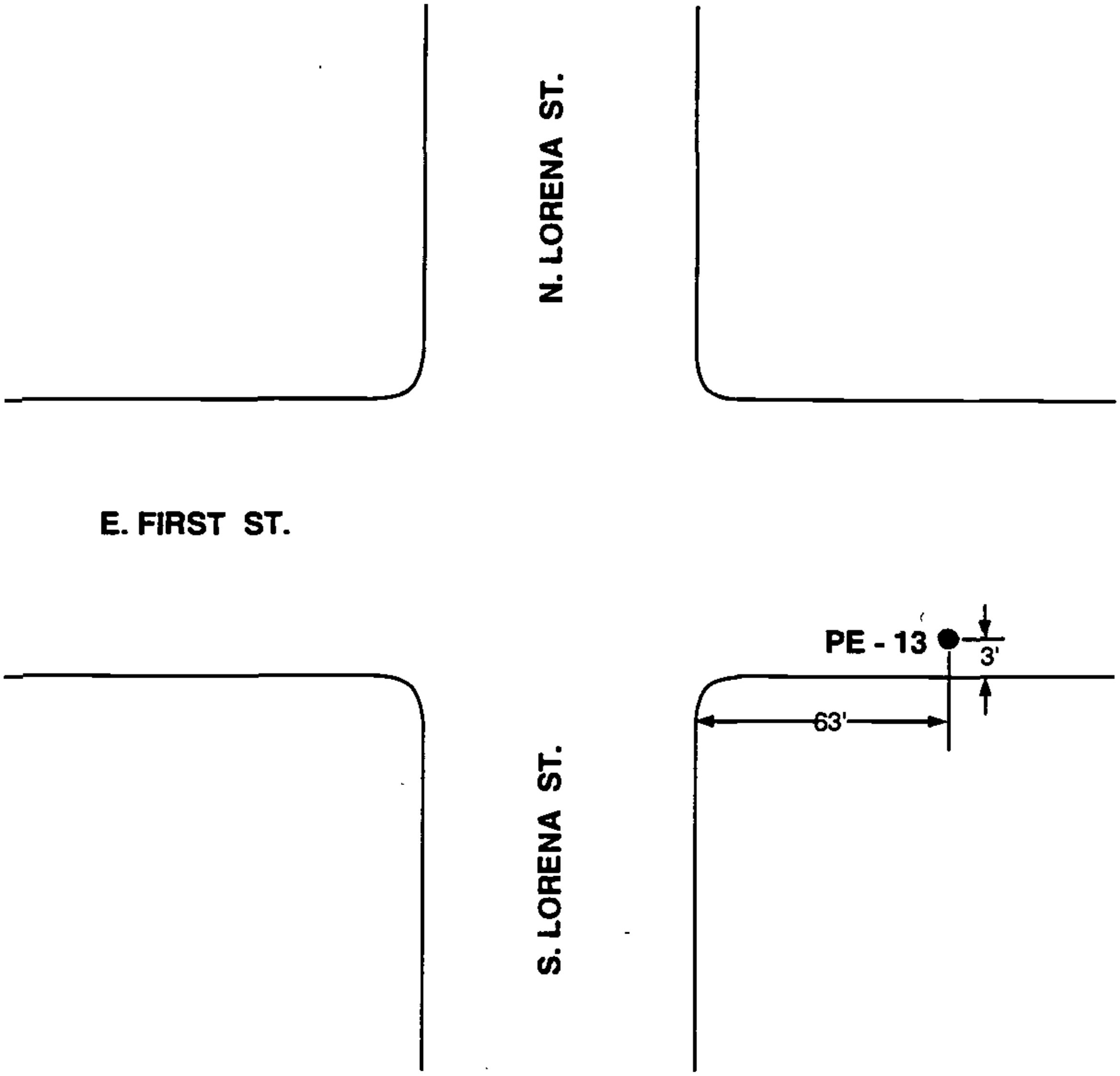
Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.

GeoTransit Consultants

Project Name:	Metro Rail: Eastside Extension - Metro Red Line				
Project Number:	94-1100	Boring Number:	PE-13	Sheet 4	of 4

Depth (feet)	Lithology	Description	USCS Classification	Geologic Unit	Samples: Number	Samples: Type	Samples: Blow Count (no./6 inches)	Samples: Recovery (inches/inches)	Samples: OVA (ppm) [Background]	Samples: Dry Density (pcf)	Samples: Moisture Content(%)	Drilling Time
	▨	CLAY; strong brown, moist to wet, hard, medium plasticity	CL	Qa	13	S	12 23 29	18/18	8.0 [8.0]			12:50
85	▨	interval of SILT/LEAN CLAY, yellowish brown, moist, low to medium plasticity, micaceous	ML/CL		14	D	60	1/6	11 [7.3]	105	22	13:15
	⁘	possible interval of SILTY GRAVEL/SILTY SAND (GM/SM)										
	×	**FERNANDO/PUENTE FORMATION** SILTSTONE; olive yellow, moist to wet, soft rock, nonplastic, fine-grained sand, highly micaceous, intensely weathered to decomposed	ML	Tf/Tp								
90	×				15	S	50	5/6	9.2 [8.0]			13:40
		Boring terminated at 90.5 feet. Monitoring well was dry on 1/11/94.										
95												
100												
105												

Note: This boring log is based on field classification and visual soil description and is further modified to include results of laboratory classification tests, where available. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of the actual conditions encountered. Lithologic patterns are generalizations and necessarily imprecise. Lithologic contacts indicated represent approximate boundaries between subsurface material types and the transitions may be gradual.



North

Not to Scale

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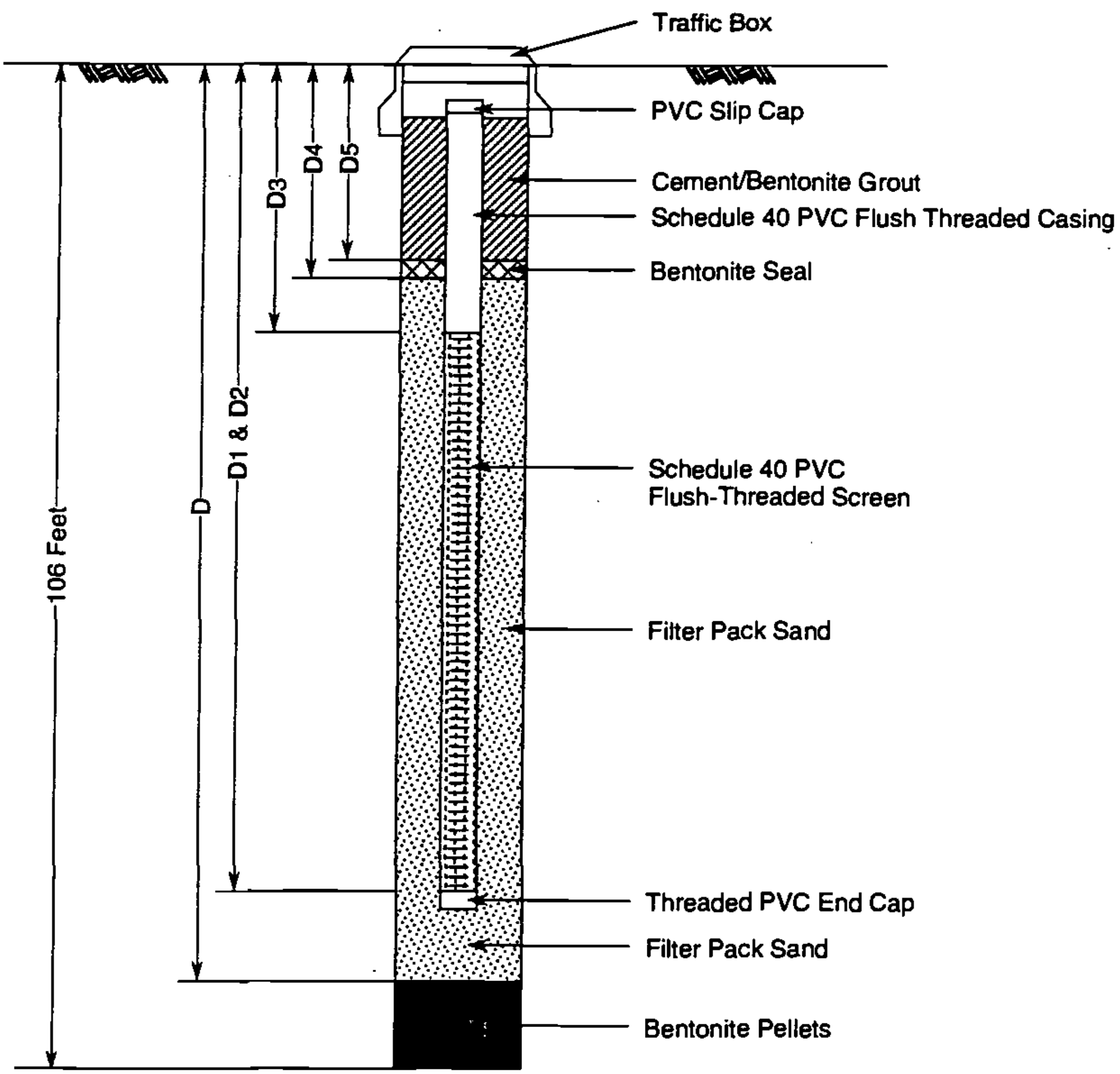
Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

Location of Boring
PE - 13

12-95

Figure A-11A



Total Depth (D)	=	67.5	Feet
Total Depth of Casing (D1)	=	65	Feet
Depth to Bottom of Well Screen (D2)	=	65	Feet
Depth to Top of Well Screen (D3)	=	35	Feet
Depth to Bottom of Top Seal (D4)	=	32.5	Feet
Depth to Top of Top Seal (D5)	=	29.5	Feet
Well Casing Diameter	=	2	Inches
Well Screen Slot Size	=	0.01	Inch
Filter Pack Sand Type	=	#2/12	Monterey
Bentonite Seal Type	=	1/2	Inch Pellets

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Project No. 95-8347
Geotechnical Investigation
Eastside Extension
Metro Red Line

Monitoring Well Installation Schematic
PE-13

12-95

Figure A-11B

APPENDIX A1

PRESSUREMETER TEST RESULTS

APPENDIX A1

PRESSUREMETER TEST RESULTS

Pressuremeter Data

Eastside Extension Metro Rail Project

Submitted to:

Geotransit Consultants

Los Angeles, California

October 1995

HUGHES INSITU ENGINEERING INC.
Suite 804, 938 Howe Street. Vancouver B.C. Canada V6Z-1N9
Phone (604) 331-4451 Fax (604) 331-4452

Metro Rail Project Eastside Extension

1.0 Introduction

This data report covers the pressuremeter testing undertaken at Hole DD27 for the Eastside extension of the Metro Rail Project. This pressuremeter study was conducted by Hughes Insitu Engineering Inc., under subcontract to TDL Engineers Inc., Los Angeles. The field work was directed by Mr. Bill Lu of TDL Engineers Inc.

From September 27 to September 29, 1995, 13 pressuremeter tests were attempted in Hole DD27 on the south-west corner of Indian Avenue and 1st Street in East Los Angeles. These pressuremeter tests were undertaken in dense natural silty sand deposits. Some gravel layers were present.

The primary purpose of this study was to determine the material properties such as stiffness and strength for the proposed tunnel. The test levels are listed in Table I.

2.0 Pressuremeter

The pressuremeter is essentially an instrumented packer which is placed in the borehole and expanded, under controlled conditions, against the borehole wall. The pressures and radial displacements which occur during the expansion are measured electronically inside the packer. The data is recorded automatically by a computer every 6 seconds. During the test, the data is displayed graphically on a computer screen. The details of this system are illustrated in Figure 1.

A high-pressure pressuremeter, with a diameter of 2.92 inches, with a pressure range of up to 3000 psi, was used for this study. This instrument was placed down a hole which was carefully drilled to a diameter as close to that of the pressuremeter as possible. The expansion of the pressuremeter was controlled by manually pumping hydraulic oil into the packer section.

The hole was advanced with a 4.5 inch bit. SPT tests and drive samples were obtained between each pressuremeter test. At the pressuremeter test location, a pocket was formed for the pressuremeter by drilling a four-foot length of hole using either a 2-3/4 inch diameter drag bit or a 2-15/16 inch diameter tricone bit. The drag bit did not prove very successful. However the tricone bit worked well, except for Test 12 at 92.5 ft. At this location, the hole was washed out, probably due to the presence of gravel.

Date	File Number	Depth	Material type [1]	Hole formation
September 13				
	Ertec1.p	28 ft	silty sand	2 -3/4" Drag bit[3,4]
	Ertec2.p	40.5 ft	sandyclay.	2 -3/4" Drag bit[3]
	Ertec3.p	47 ft	silty sand/clayey sand	2-15/16" Tricone
	Ertec4.p	53.5 ft	clay/silty sand	"
September 14				"
	Ertec5.p	60 ft	silty sand[2]	"
	Ertec6.p	66.5 ft	silty sand	"
	Ertec7.p	73 ft	silty sand/sandy silt	"
	Ertec8.p	79.5 ft	silty sand[2]	"
	Ertec9.p	86 ft	silty sand	"
September 15				
	Ertec10.p	92.5 ft	fine silty/sand	"
	Ertec11.p	99.5 ft	sandy silt/silty sand	"
	Ertec12.p	107.5 ft	sandy silt/silty sand	"[3,4]
	Ertec13.p	112.5 ft	clayey sand/silty clay	"

Notes

1) Material type as determined from adjacent samples.
2) The form of the pressuremeter curves indicate a predominantly cohesive material.
3) Oversized hole
4) Oversized hole, no useful data obtained

Table I. Test depth, Material type and Hole Formation

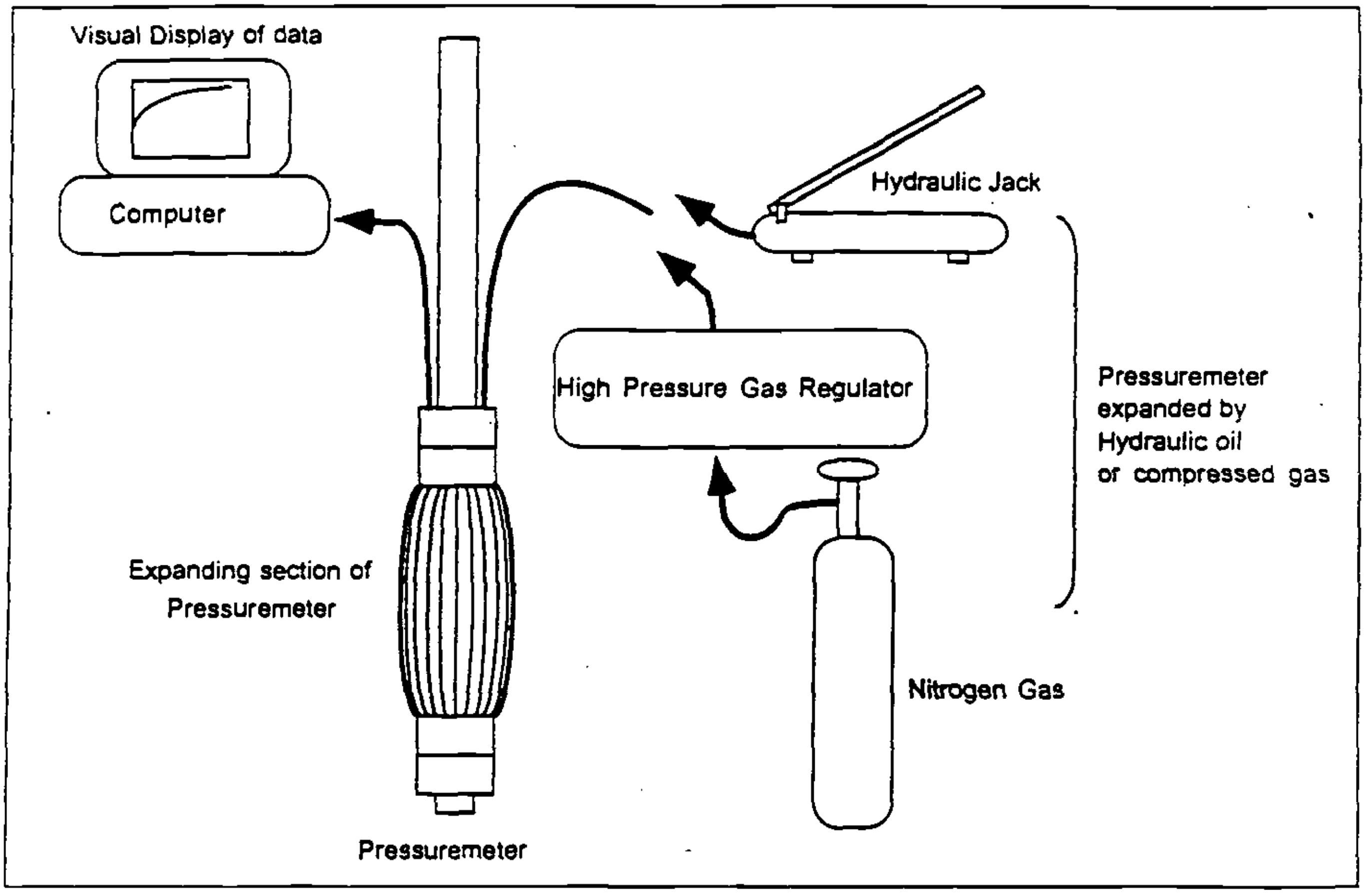


Figure 1. Details of Pressuremeter System

3.0 Test procedure

The general procedure was conducted according to the ASTM standard D4719-87 as illustrated by the pressuremeter Test 11 (Figure 2). After the pressuremeter is placed at the test location, the packer section is expanded by manually pumping hydraulic oil into it from the surface. If there is a gap around the instrument, as shown in Figure 2, the expansion occurs rapidly, until the membrane comes in contact with the borehole wall (i.e. at approximately 5% radial displacement). The pressuremeter is then expanded in increments of approximately 50 psi.

The pressure increment of 50 psi was selected to provide about ten loading increments before the strain reached a maximum on any one of the displacement sensors. In heterogeneous soils, it is difficult to estimate the magnitude of the appropriate pressure increment required to achieve ten loading increments. As a result, the number of pressure increments varied from test to test.

After each pressure increment is reached, the pressure is held constant for 60 seconds. The pressure is then increased again a further increment, and held for a further 60 seconds. In the test illustrated, there are eight stages at which the pressure is held constant. After the end of some of the holding phases, the pressure is reduced and then increased again (as, for example, after the fourth and the sixth increment in Test 11). The pressure is reduced to usually less than half or one-third of the maximum pressure at the start of the pressure deduction Line G-H.

The purpose of limiting the size of the pressure deduction cycle is to keep the strains within the elastic range. If the pressure reduction cycle is too large, the soil surrounding the pressuremeter will fail inwards, and large plastic strains will result, as can be seen on the final unloading curve. A measure of the small strain elastic shear modulus (G) can be obtained from the slope of the pressure expansion loops, A-B and C-D.

4.0 Termination of the Test

The test was terminated when one of the following criteria was satisfied:-

1) One of the three displacement sensors exceeded 18% strain.
2) If there was no increase in the strain, as measured by the displacement sensors, as oil continued to be pumped into the pressuremeter. In this situation, the membrane is probably expanding into a cavity remote from the displacement sensors located in the center quarter of the membrane. Further expansion in this situation will only rupture the membrane. (Note: if the membrane is expanded by using compressed gas, the above termination criterion is not available.)

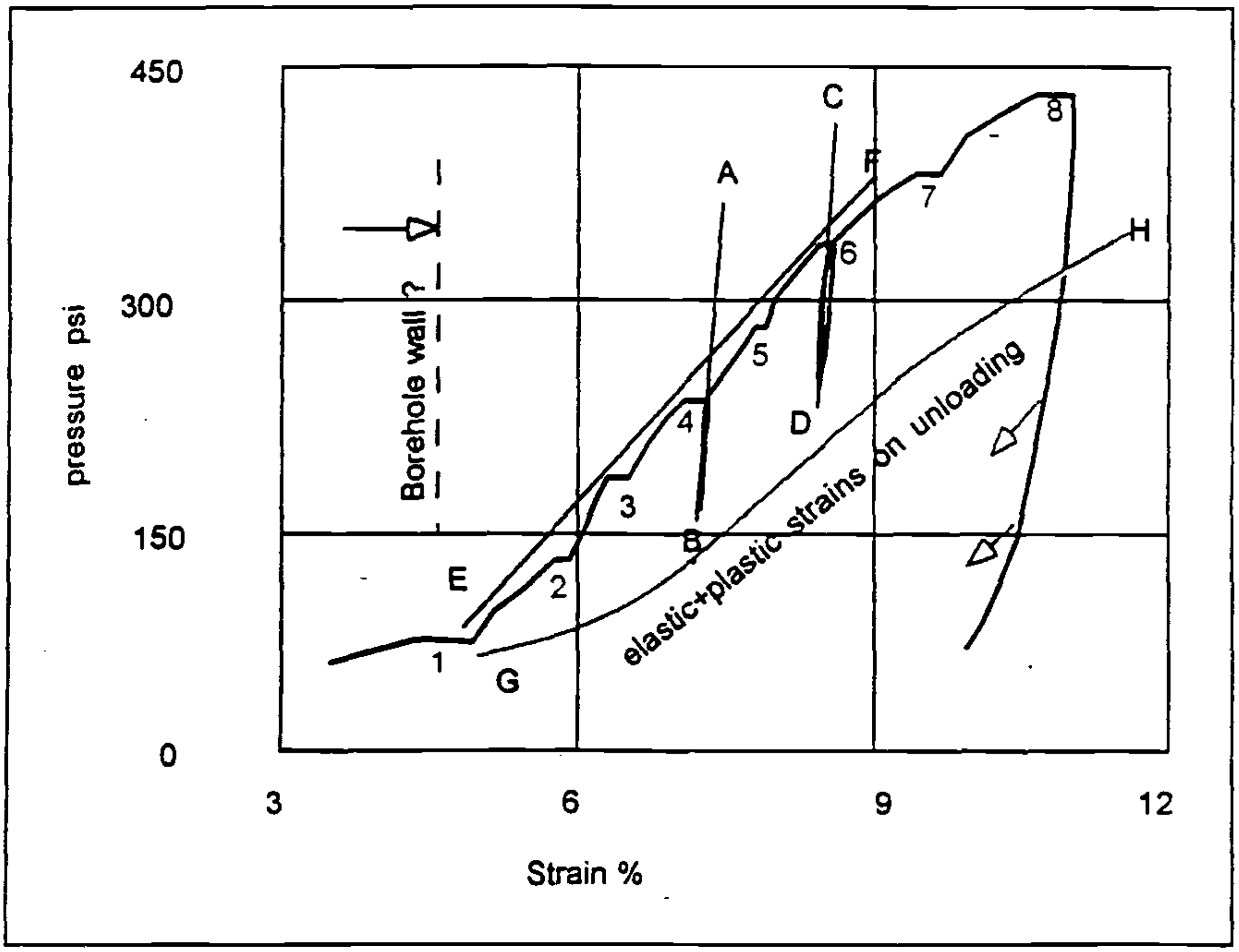


Figure 2. Pressuremeter Test 11 at 99.5 ft depth

5.0 Data Presentation

The pressure and strain sensors inside the pressuremeter are on the inside of the packer membrane. Hence, they do not directly measure the pressure or displacement on the borehole wall. Corrections have to be made to adjust for the compressibility and stiffness of the packer.

Two corrections are made:-

1) The probe is expanded in air to determine the stiffness of the membrane
2) The probe is expanded inside a thick-walled steel tube to determine the compressibility of the membrane

All of the data presented in Appendix I have been adjusted to account for these effects.

Also included in these plots are the creep strains, which occur while the pressure is held constant. The creep strains over the 60 seconds that the pressure is held constant are presented to the left of the pressure expansion curve. The intended purpose of this information is to determine the creep limit (P_ℓ), the pressure at which substantial creep movement initiates. For Test 11 (Figure 2), the creep pressure is in the region of the seventh holding phase at 370 psi. In Test 2 and Test 13, which were oversized holes (particularly Test 2), creep readings were not taken.

6.0 Basic Analysis of the Pressuremeter Data

The pressure expansion curves have been analyzed in the conventional manner to determine the Limit Pressure (P_L), the "Menard" modulus (E_M), and the equivalent undrained cohesion (S_u). The slope of the load/unload loops is used to determine the shear modulus (G_{ur}) The creep strain curves are used to assess the creep pressure (P_ℓ), the pressure at which substantial creep movement initiates.

6.1 Limit Pressure (P_L)

The limit pressure is conventionally defined as the pressure at which the probe has expanded by twice its original volume. As the pressuremeter is never expanded to this value, the limit pressure is determined by extrapolation.

In cohesive material, the total pressure plotted against the radial strain often tends toward a straight line. This line is then used to determine the limit pressure by extrapolation to the radial strain corresponding to the strain at twice the volume expansion. Adjustments have to be made to the strain origin to allow for the initial diameter of the cavity before expansion.

An example of the determination of the limit pressure for Test 11 is given in Figure 3. The origin for Test 11 has been shifted by 5% to allow for the initial diameter of the cavity before the expansion. The limit pressure determined in this manner is presented in Table II.

6.2 Shear Strength

If the material is assumed to deform as a linear elastic-plastic material, then the slope of the pressure against the log of the strain will tend to a straight line (as shown in Figure 3) and be proportional to the shear strength where:

Shear strength= (change in pressure) / (natural log of the change in strain)

The shear strength determined by this method is presented in Table II. In view of the assumption concerning the material behavior, this strength can be considered only as an <u>indicator</u> of the material behavior.

6.3 "Menard" Modulus (E_M)

The Pressuremeter modulus, E_M, (a form of Young's Modulus) is determined from the slope of the linear portion of the pressure-expansion curve (line EF in Figure 2). The Menard modulus is given by:-

E_M=(1+μ)* slope of the pressure / radial expansion strain
(where μ, the Poisson's ratio, is assumed to have a value of 0.3)

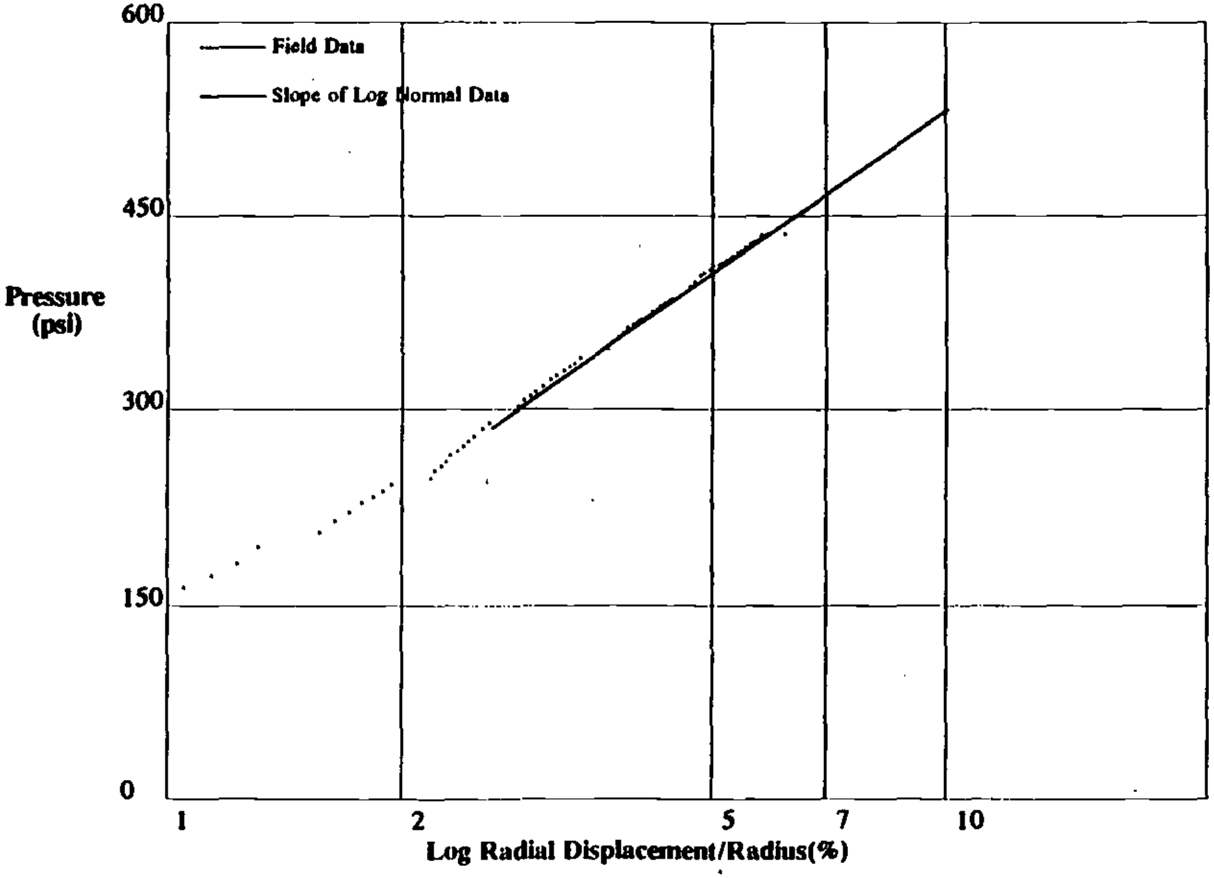
6.4 Load/Unload Modulus (G_{ur})

The low strain shear modulus G, as determined from the slope of the load/unload loops, is given by:-

G = 0.5 * slope of the pressure / radial expansion strain

The equivalent Young's Modulus, E, can be calculated from the shear modulus from the following expression: $E = 2G(1 + \mu)$. However, a value of the Poisson's ratio has to be assumed. For small deformations, a value of 0.3 for the Poisson's ratio is probably a reasonable estimate. The load unload shear modulus G is presented in Table III.

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 15 1995**
Hole No DD27 Depth 99.5 ft	D:\DATA\LOSANG\ERTEC11.P



Shear Strength	**180 psi**
Limit Pressure	**760 psi**

Figure 3. Pressure/log Displacement for Test 11

shift 5

HUGHES

File name	Depth (feet)	Limit Pressure (P_L) (psi)	"Cohesion" (psi)	Menard Modulus (E_M) (psi)	Creep Pressure (P_f) (psi)
Ertec1.p	28 ft			***	***
Ertec2.p	40.5 ft			***	***
Ertec3c.p	47 ft	540	70	21,600	320
Ertec4c.p	53.5 ft	860	200	13,460	340
Ertec5.p	60 ft	530	30	14,940?	400
Ertec6.p	66.5 ft	570	110	9,900	370
Ertec7.p	73 ft	720	140	11,060	370
Ertec8.p	79.5 ft	1,240	310	16,600	>450
Ertec9.p	86 ft	690	140	9,420	380
Ertec10.pli	92.5 ft	570	120	5,680	300?
Ertec11.pli	99.5 ft	760	180	9,220	370
Ertec12.pli	107.5 ft	***	***	***	***
Ertec13.pli	112.5 ft	860	180	12,500	***(1)

Note (1) Creep measurements not taken

Table II. Standard Pressuremeter Interpretation P_L, E_M and P_f

7.0 Alternative Method of Analysis

The procedure used in this section was developed to handle data from tests in which the material has been disturbed during the insertion of the probe. The disturbance comes as a result of material collapsing into the hole, stress relief, or the formation of a poor hole caused by some obstruction, such as stones and gravel in the hole. In this procedure, the field curve is used as a basis for developing an idealized pressuremeter curve for the particular material. The better the definition of the field curve, the better the definition of the ideal curve. The mathematical process briefly described above is known as an inversion process.

In this inversion method, the material is assumed to deform according to a mathematical model. The model is then used to predict the field curve. The parameters used in the model are adjusted to give the most appropriate fit, using an interactive computer program.

The simplest model for cohesive materials is to assume the soil deforms elastically until the shear stresses are such that the soil fails at a constant shear stress. This model is based on the procedure proposed by Menard for the analysis of pressuremeter tests in clay.

This model requires three parameters:

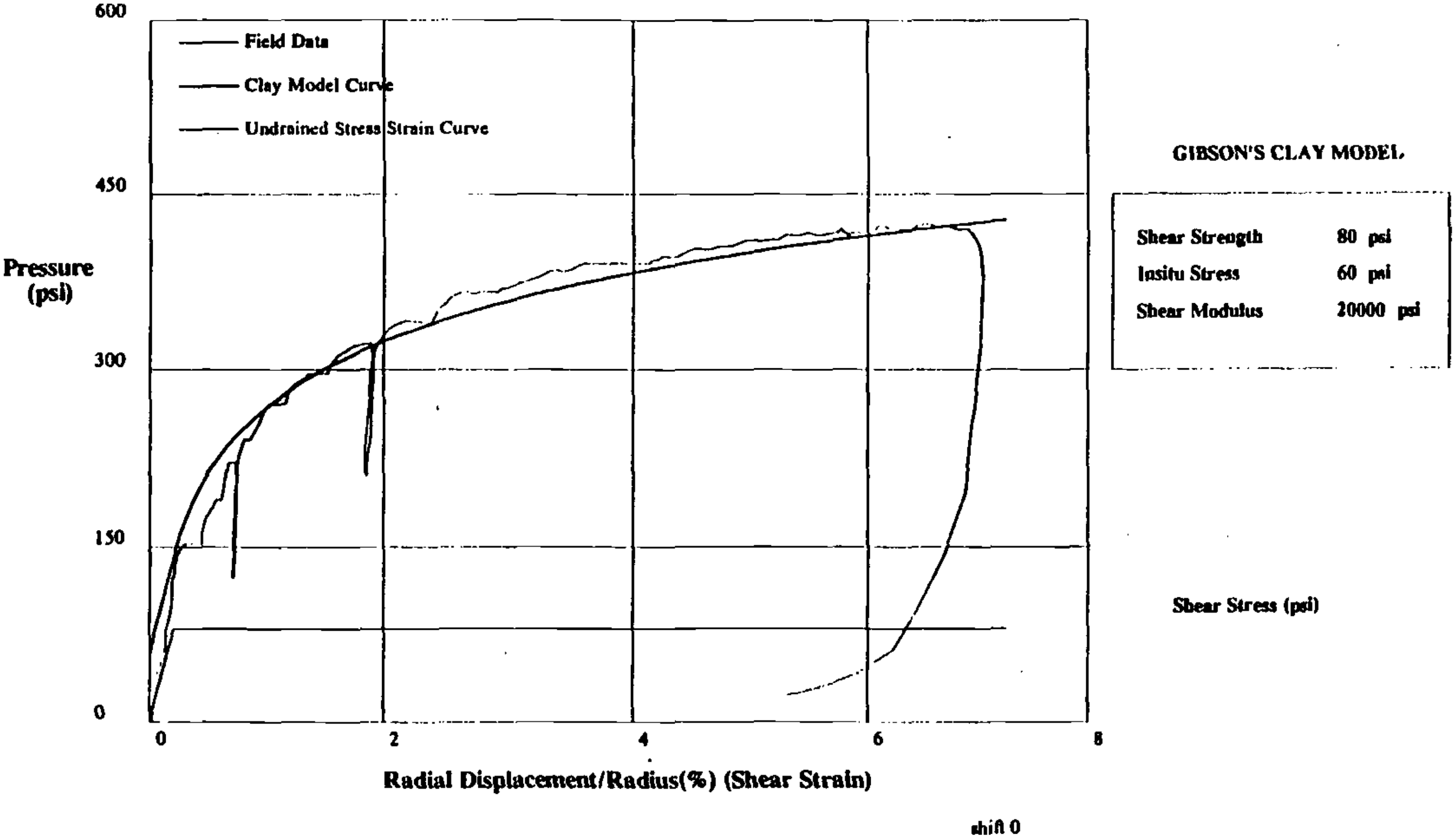
1) Initial shear modulus (i.e. the secant shear modulus from zero strain to the initiation of failure)
2) Total insitu lateral stress
3) Shear strength

An example of this technique as applied to Test 3, in a clay/silty sand material, is given in Figure 4. The assumed linear-elastic stress strain curve for the undrained behavior of the argillite is the lower graph in the figure (green line). The ideal pressure expansion curve, derived from the assumed undrained stress strain curve, is given by the smooth continuous line through the field data (blue line). Adjustments have to be made to the strain origin to allow for the initial size to the cavity.

The results of this analysis are presented in Table III and in Appendix III. The result of this alternative analysis is not to indicate that the material under test is actually purely cohesive, but that the parameters selected give some match to the field data.

It is important to emphasize that it is the <u>set of parameters</u> that match the field data. No one parameter should be considered in isolation and used with parameters obtained from other tests. If some of the parameters are obtained by alternative means, then the remainder of the parameters may have to be adjusted to make the model match the field pressuremeter data.

Pressuremeter Data		**Geotransit Consultants**
Eastside Extension, Metro Rail Project		**September 13 1995**
Hole No. DD27	**Depth 47 ft**	**File D:\DATA\LOSANG\ERTEC3.P**



GIBSON'S CLAY MODEL

Shear Strength	80 psi
Insitu Stress	60 psi
Shear Modulus	20000 psi

shift 0

HUGHES

Figure 4. Analysis of Pressuremeter Test 3 using a Cohesive Model

File Name	**Depth**	**Secant Modulus (psi)**	**Cohesion (psi)**	**Total Lateral Stress (psi)**	**Cyclic Shear Modulus G (psi)**
Ertec1.p	28 ft	***	***	***	***
Ertec2.p	40.5 ft	***	***	***	>26,000
Ertec3c.p	47 ft	20,000	80	60	72,220
Ertec4c.p	53.5 ft	8,000	170	40	55,000
Ertec5.p	60 ft	50,000	70	100	122,500
Ertec6.p	66.5 ft	8,000	110	50	32,780
Ertec7.p	73 ft	8,500	145	50	42,000
Ertec8.p	79.5 ft	8,500	240	60	48,680
Ertec9.p	86 ft	8,400	140	50	29,050
Ertec10.pli	92.5 ft	4,000	130	70	31,540
Ertec11.pli	99.5 ft	6,000	155	70	31,540
Ertec12.pli	107.5 ft				***
Ertec13.pli	112.5 ft	7,000	180	80	32,670

Table III. Alternative Analysis of the Data assuming a Cohesive Model

In general, a reasonable match can be found for all of the tests, with the exception of Test 5, at 60 ft depth. In this test, a crude fit can be found only by moving the origin to the right. This would indicate that the pilot hole into which the instrument was placed was undersize, and that the instrument was forced into place. The second unusual feature is the very flat pressure expansion curve above 450 psi. If the soil is again assumed to be cohesive, and deform at constant volume but not at constant shear stress, the model proposed by Arnold (1981) gives a better fit to this data. The results of this analysis, presented in Appendix III, indicate that the material at this location is perhaps brittle, and the material surrounding the cavity is cracking as it expands.

The tests have also been analyzed on the assumption that the soil behaves as a frictional material having no cohesion. This model assumes that the soil deforms at a constant friction ratio during failure. Further, during the failure process, the material either contracts or expands, depending on the density and stress state of the granular material.

This model requires four parameters:

1) Initial shear modulus (the secant modulus from zero strain up to the failure stress ratio)
2) Effective lateral stress
3) Friction angle
4) Critical state frictional angle

The results of this model analysis are presented in Table IV. As noted above, the result of this alternative analysis is not to indicate that the material under test is actually a purely frictional material, but that the set of parameters selected give some match to the field data.

8.0 Tunnelling problem

If the pressuremeter could be installed in the ground without any disturbance, and then made to contract inwards, the pressuremeter would then be able to measure the convergence behavior of the soil as the pressures were reduced. In this situation, the pressuremeter would model the movements in the soil adjacent to a vertical shaft, as illustrated in Figure 5.

In the dense materials at the above site, this type of installation would be very difficult. However, some indication of the convergence behavior of the soil surrounding a vertical shaft can be determined by considering the final unloading curve. As the pressure is unloaded at the end of the test, from point A onwards, the radial stresses in the soil are initially in excess of the circumferential stresses. However, as unloading continues, the radial stresses reduce, and the circumferential stresses rise. Part-way down a point is reached (B), at which the stresses are equal, and hence there is no shear on the soil. It is this section of the curve, from B to C, that is of interest. This section of the curve, with the appropriate scaling, can be used to give an indication of the behavior of the convergence of the soil from the insitu stress conditions B'-C'.

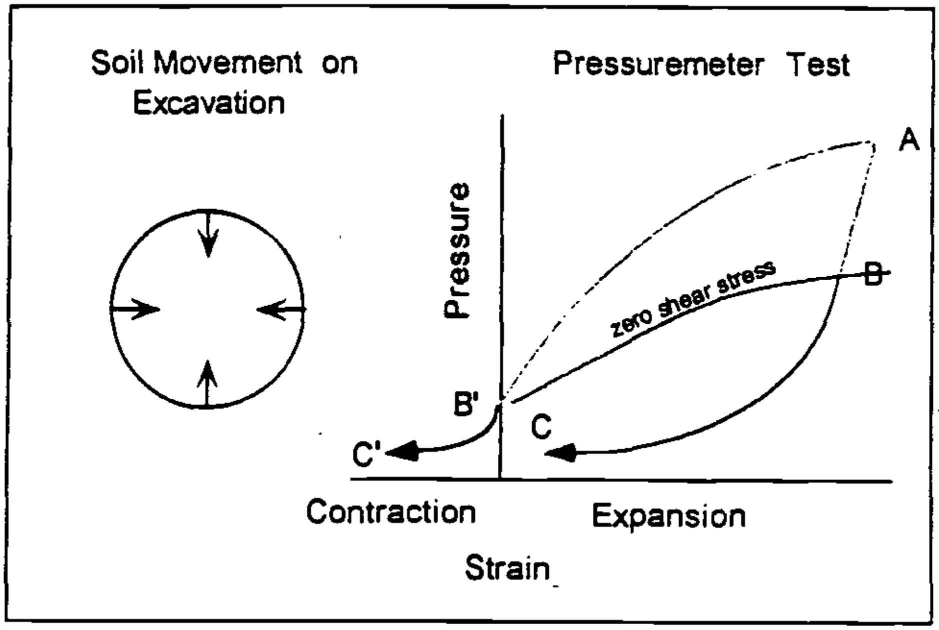


Figure 5. Convergence of Soil surrounding a Shaft

File name	Depth	Friction Angle	Secant Modulus (psi)	Effective Lateral Stress (psi)	Cyclic Shear Modulus (psi)
Ertec3c.p	47 ft	(1)			72,220
Ertec4c.p	53.5 ft	44	12,600	40	55,000
Ertec5.p	60 ft	(1)			122,500
Ertec6.p	66.5 ft	38	8,000	55	32,780
Ertec7.p	73 ft	40	9,660	60	42,000
Ertec8.p	79.5 ft	(1)			48,680
Ertec9.p	86 ft	40	7,500	65	29,050
Ertec10.p	92.5 ft	40	6,800	45	31,540
Ertec11.p	99.5 ft	41	7,250	60	31,540
Ertec13.p	112.5 ft	(1)			32,670

Notes 1) These test results indicate that the material is behaving in a predominantly cohesive manner. Therefore it not appropriate to analyze this data with a purely frictional model.

2) The water table is assumed to be at 97 ft.

3) The critical state friction angle has been assumed to be 32 degrees.

Table IV. Alternative analysis of the Data assuming a Frictional Model

Alternatively, the material parameters derived from the expansion phase of the pressuremeter test can be used to calculate the convergence of the cavity. It should be noted, however, that if gravity forces are significant, use of the unloading pressuremeter curve to determine the convergence effects could be misleading.

9.0 References

The following text covers recent research on Pressuremeter Testing and Analysis:-

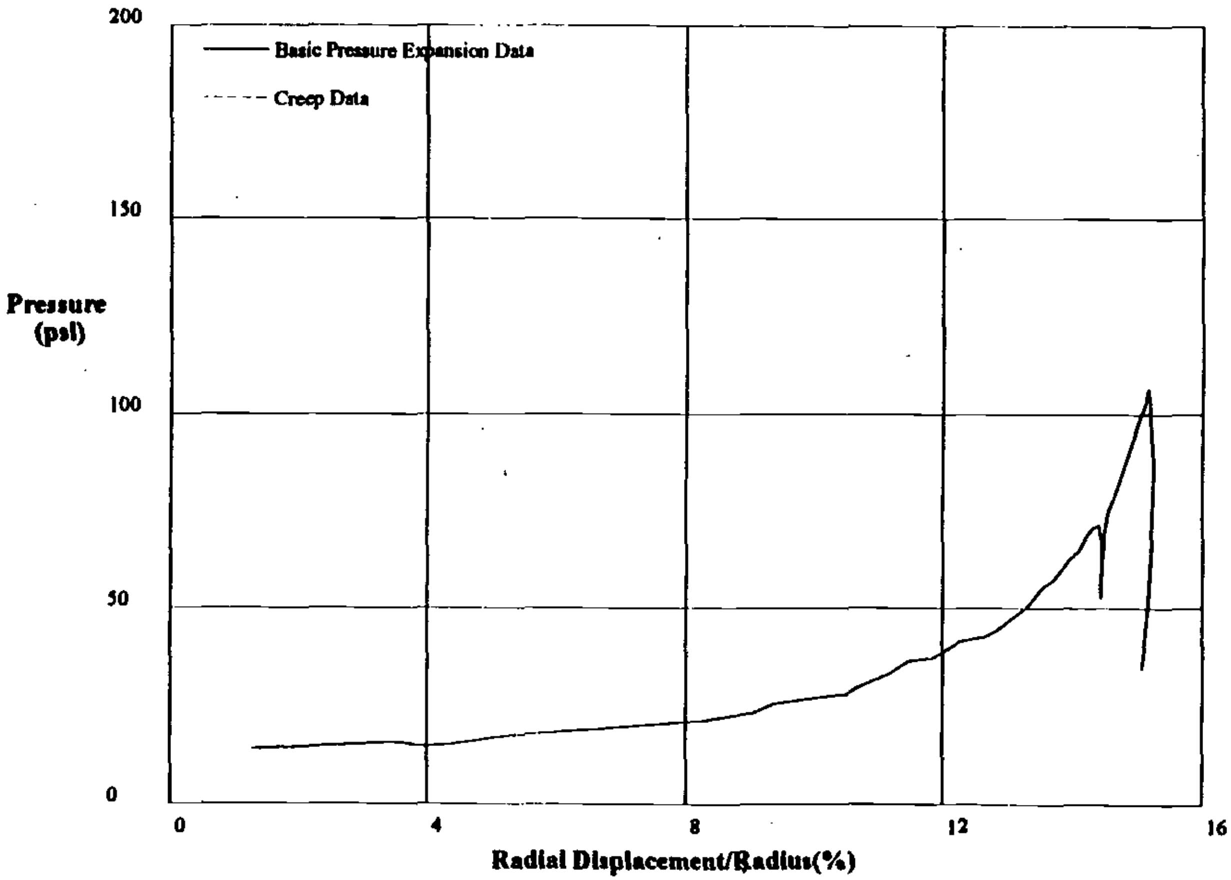
Mair, R.J., and Wood, D.M. 1987. Pressuremeter Testing : Methods and Interpretation. CIRIA Ground Engineering Report. Butterworths, London.

The following reference is not covered in the above:-

Arnold, M. 1981. An Empirical Evaluation of Pressuremeter Test Data. Canadian Geotechnical Journal 18: 455-459.

Appendix I. Basic Pressuremeter Data

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 13 1995**
Hole No. DD27 Depth 40 ft	**Files A:\ERTEC2.P**



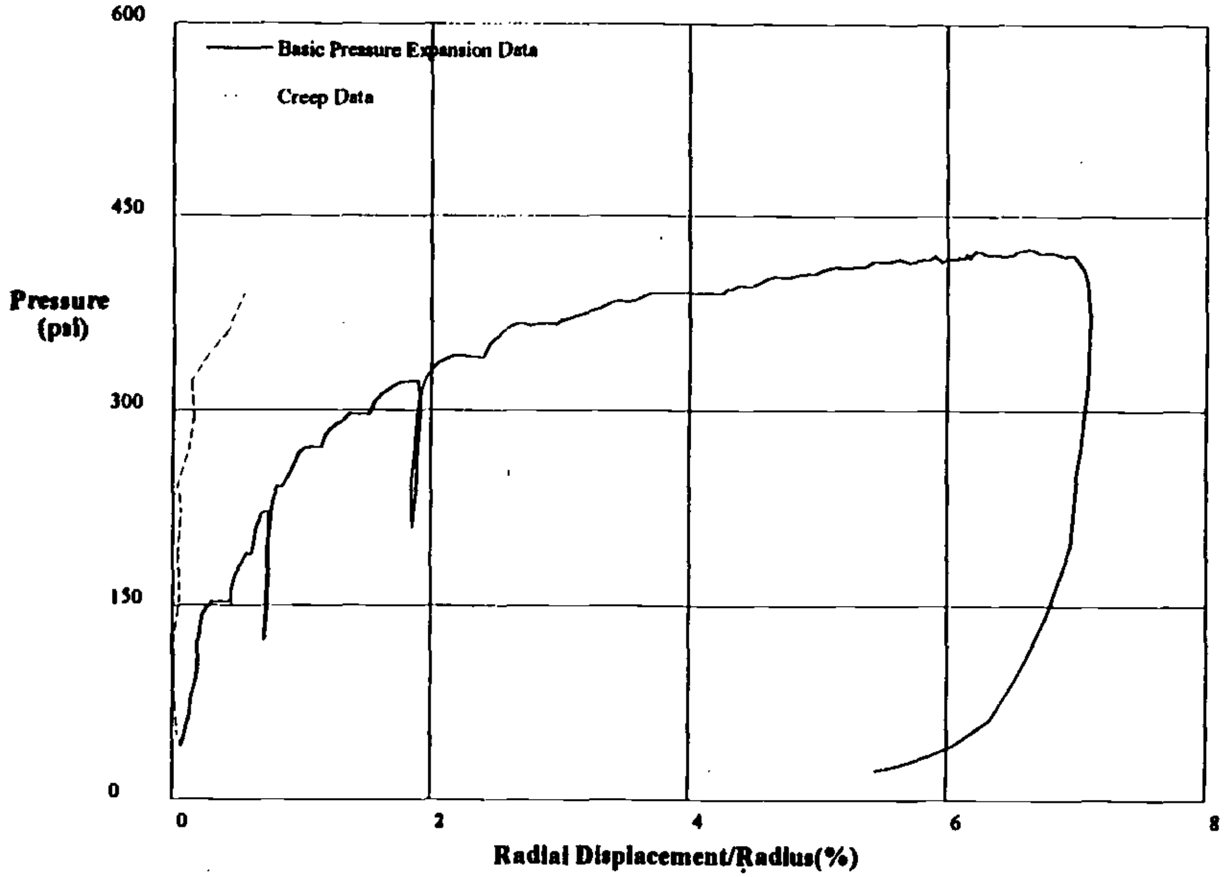
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **September 13 1995**

Hole No. DD27 **Depth 47 ft** **Files A:\ERTEC3.C, A:\ERTEC3.P**



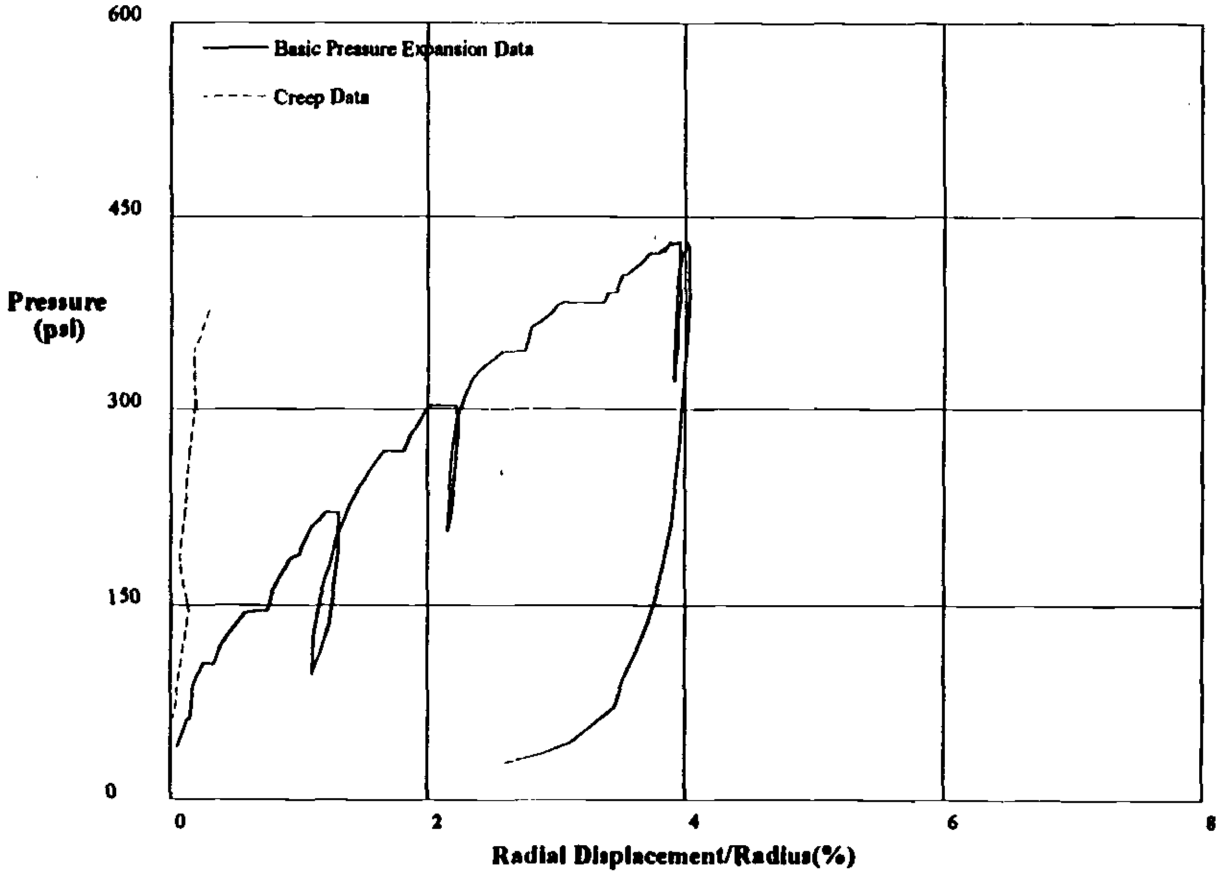
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **September 13 1995**

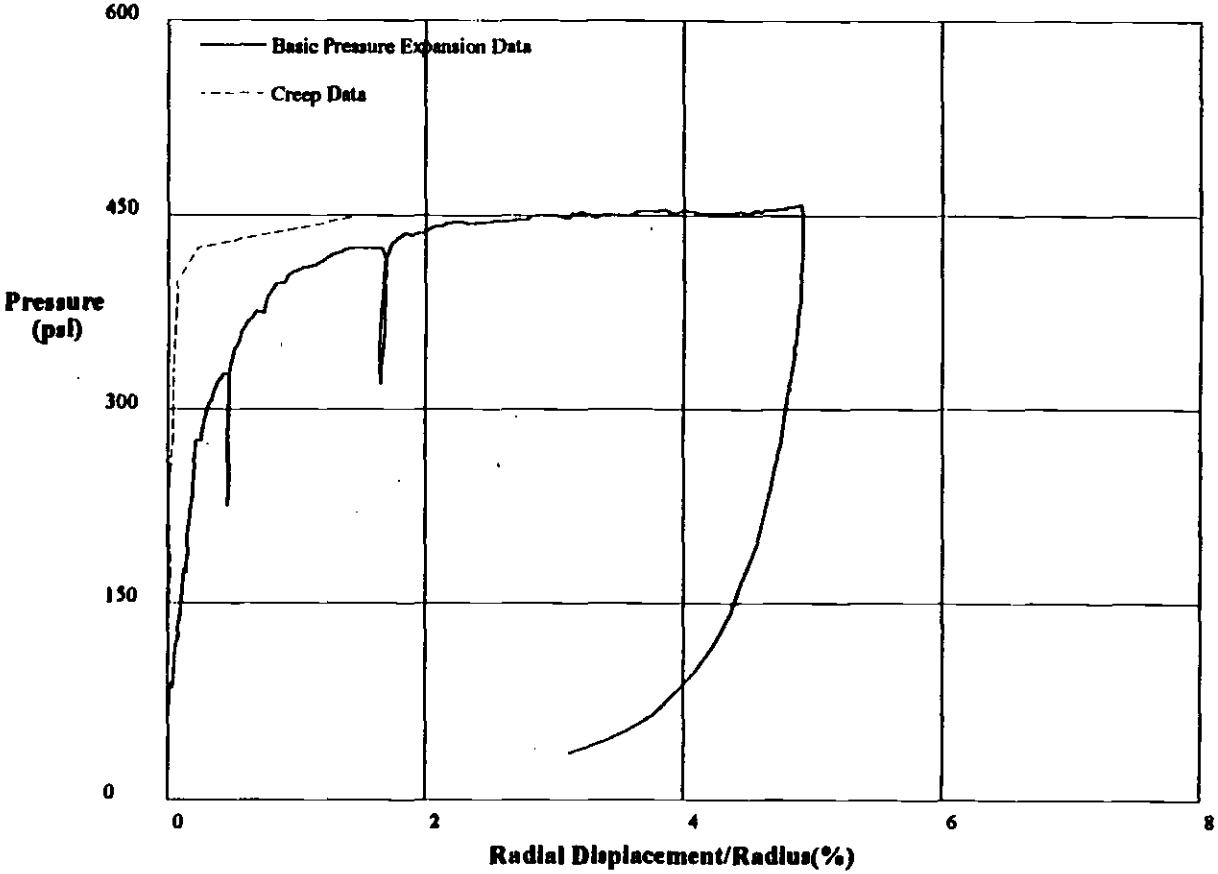
Hole No. DD27 **Depth 53.5 ft** **Files A:\ERTEC4.C, A:\ERTEC4.P**



shift 0

HUGHES

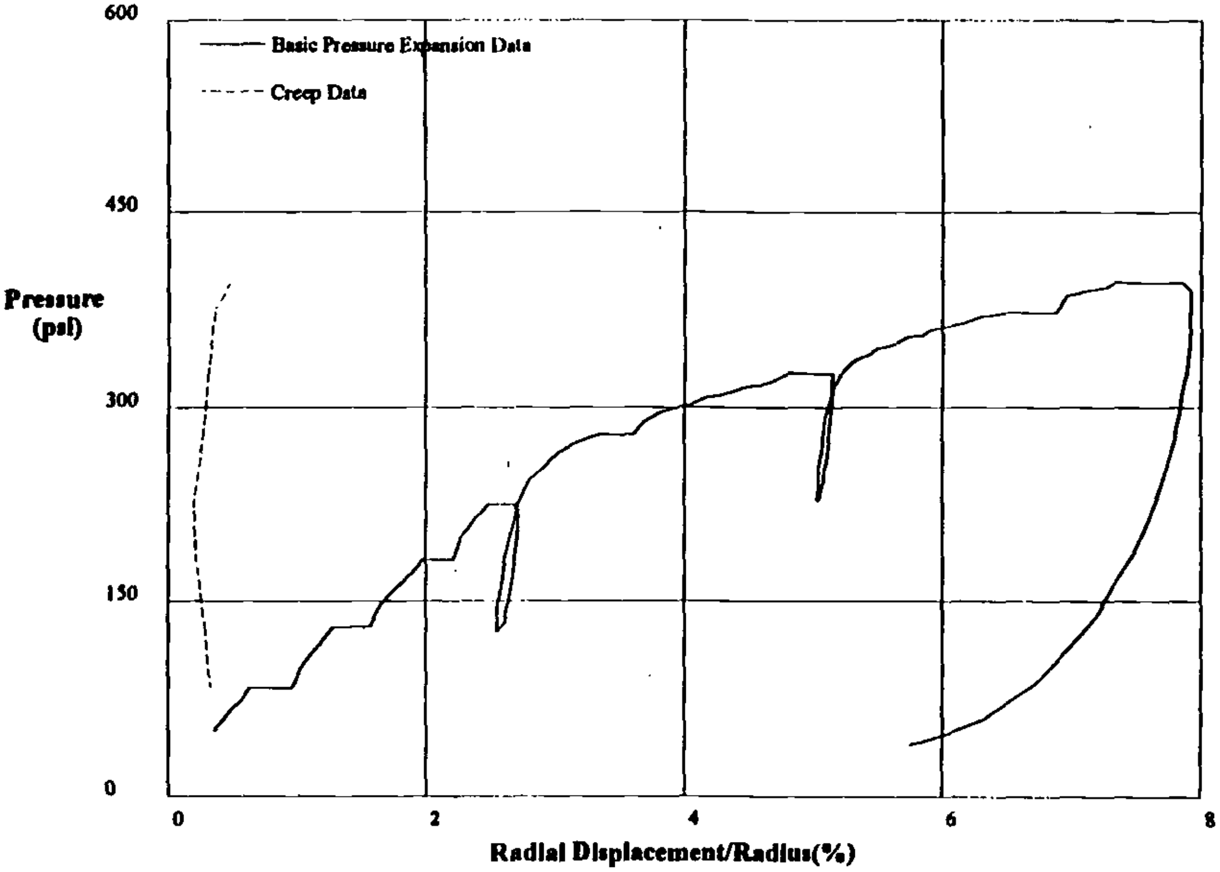
Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No. DD27 Depth 60 ft	**Files A:\ERTECS.C, A:\ERTECS.P**



shift 0

HUGHES

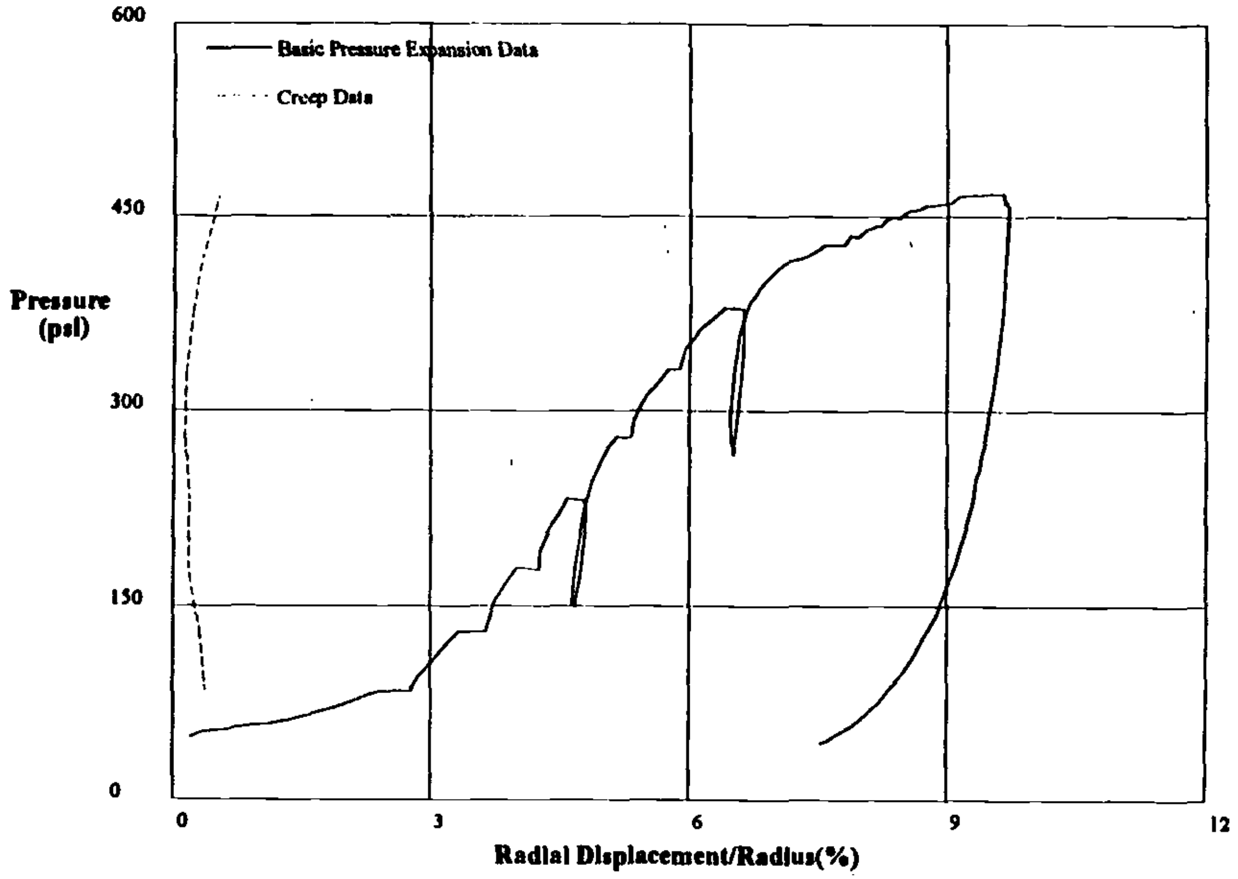
Pressuremeter Data | **Geotransit Consultants**
Eastside Extension, Metro Rail Project | **September 14 1995**
Hole No. DD27 **Depth 66.5 ft** | **Files A:\ERTEC6.C, A:\ERTEC6.P**



shift 0

HUGHES

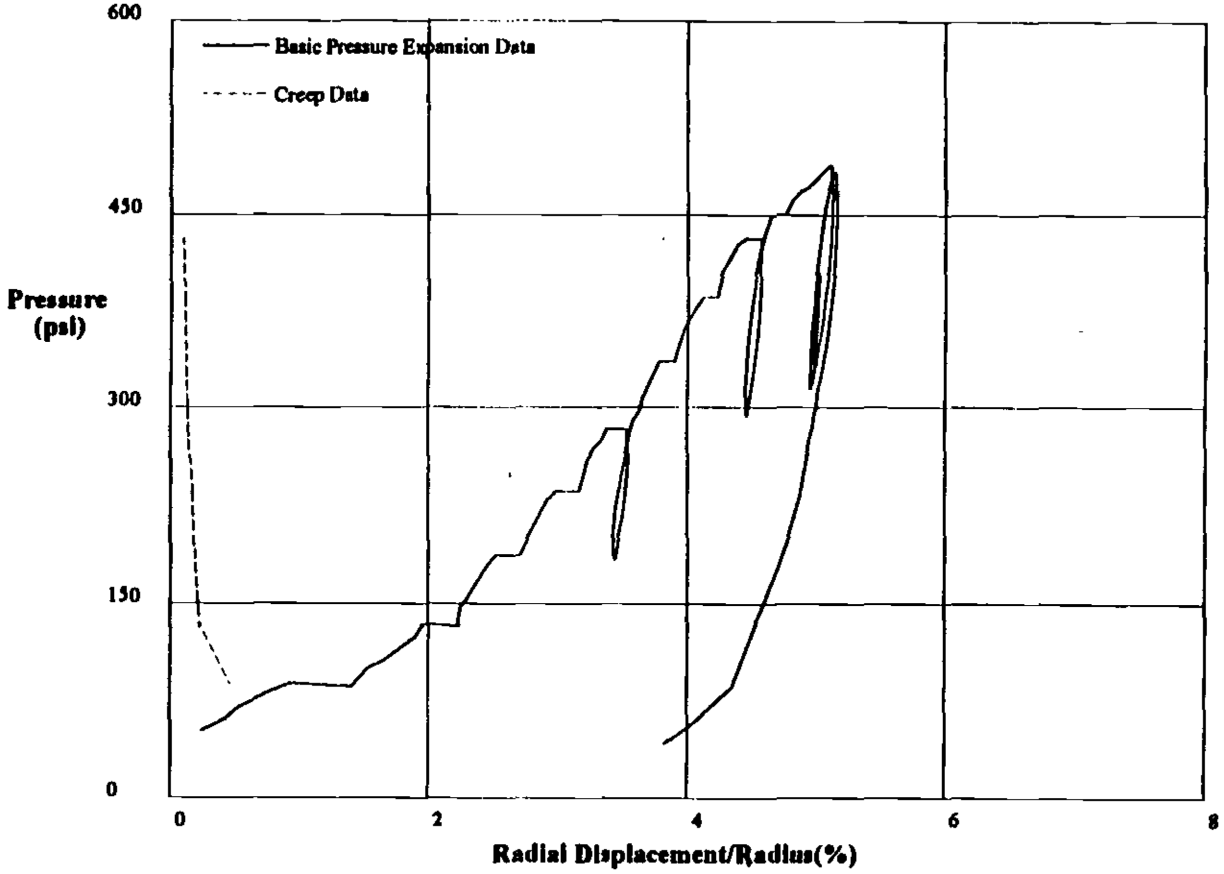
Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No. DD27 Depth 73 ft	**Files A:\ERTEC7.C, A:\ERTEC7.P**



shift 0

HUGHES

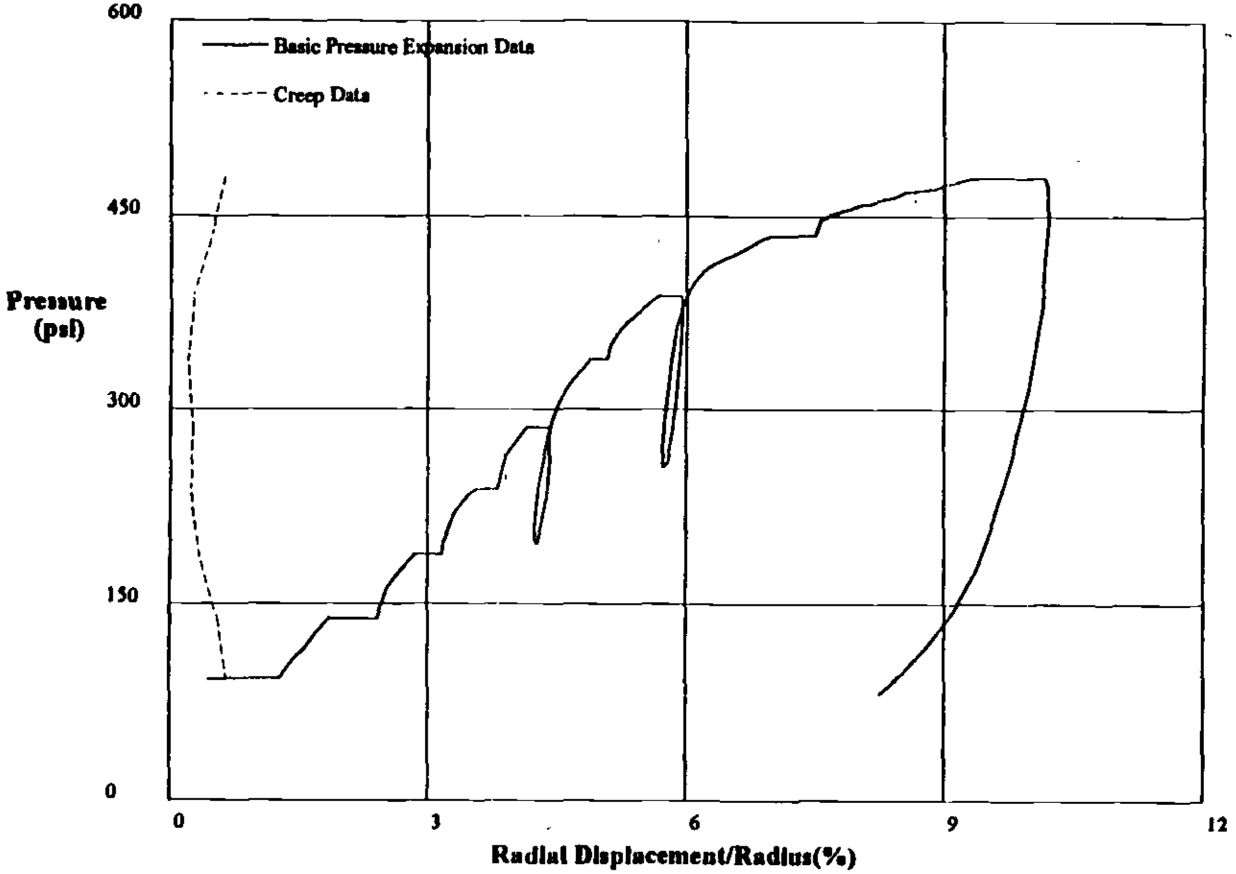
Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 14 1995**
Hole No. DD27 **Depth 79.5 ft** **Files A:\ERTEC8.C, A:\ERTEC8.P**



shift 0

HUGHES

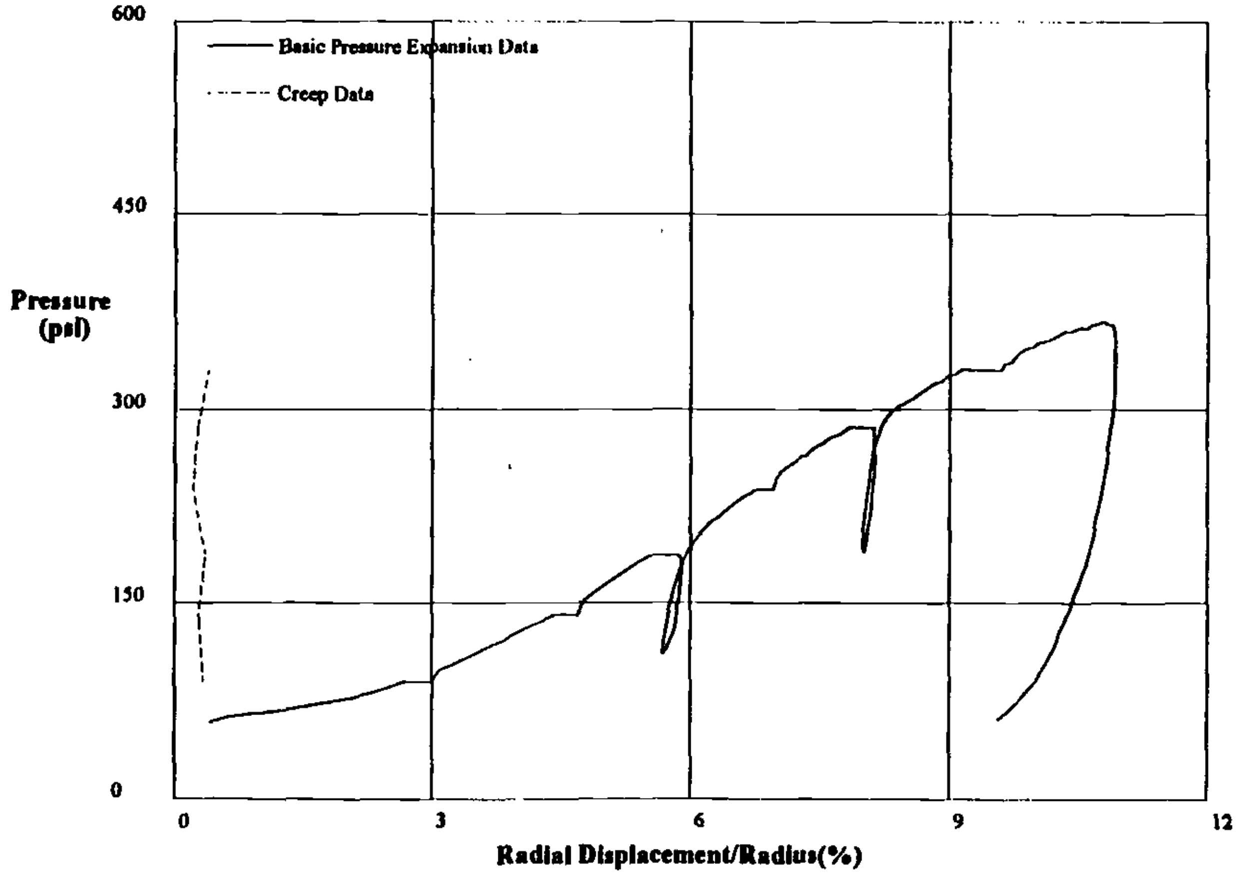
Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 14 1995**
Hole No. DD27 **Depth 86 ft** **Files A:\ERTEC9.C, A:\ERTEC9.P**



shift 0

HUGHES

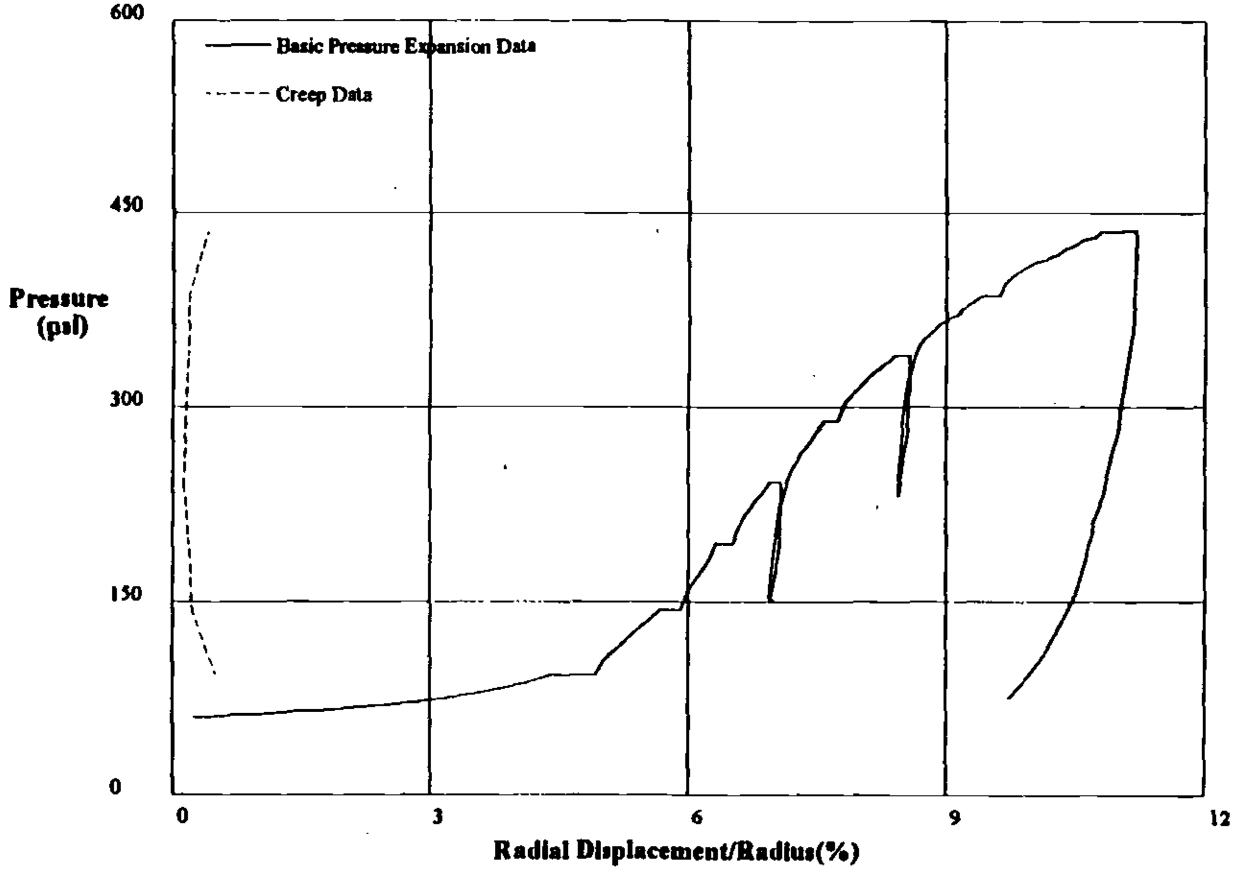
Pressuremeter Data | **Geotransit Consultants**
Eastside Extension, Metro Rail Project | **September 15 1995**
Hole No. DD27 **Depth 92.5 ft** | **Files A:\ERTEC10.C, A:\ERTEC10.P**



shift 0

HUGHES

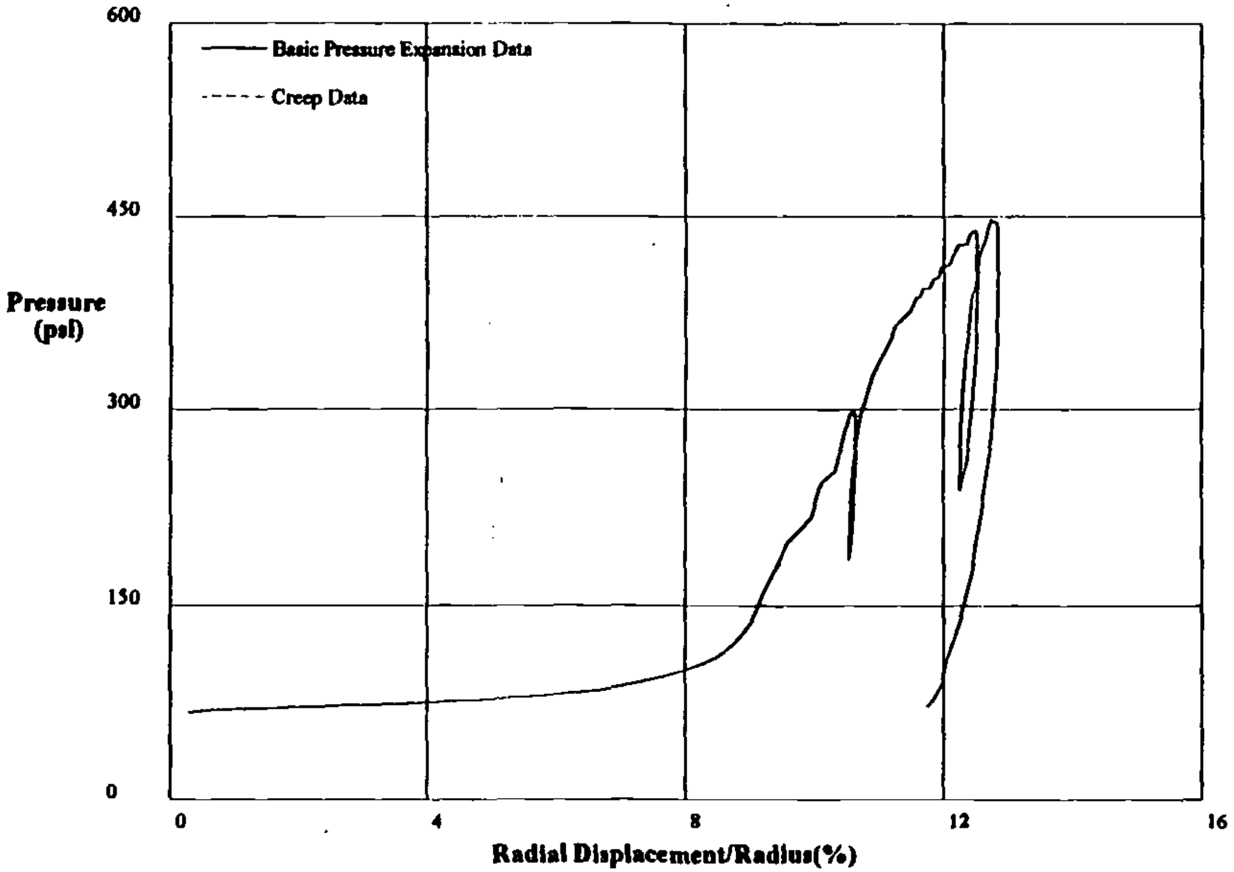
Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 15 1995**
Hole No. DD27 Depth 99.5 ft	**Files A:\ERTEC11.C, A:\ERTEC11.P**



shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 15 1995**
Hole No. DD27 **Depth 112.5 ft** **Files A:\ERTEC13.P**

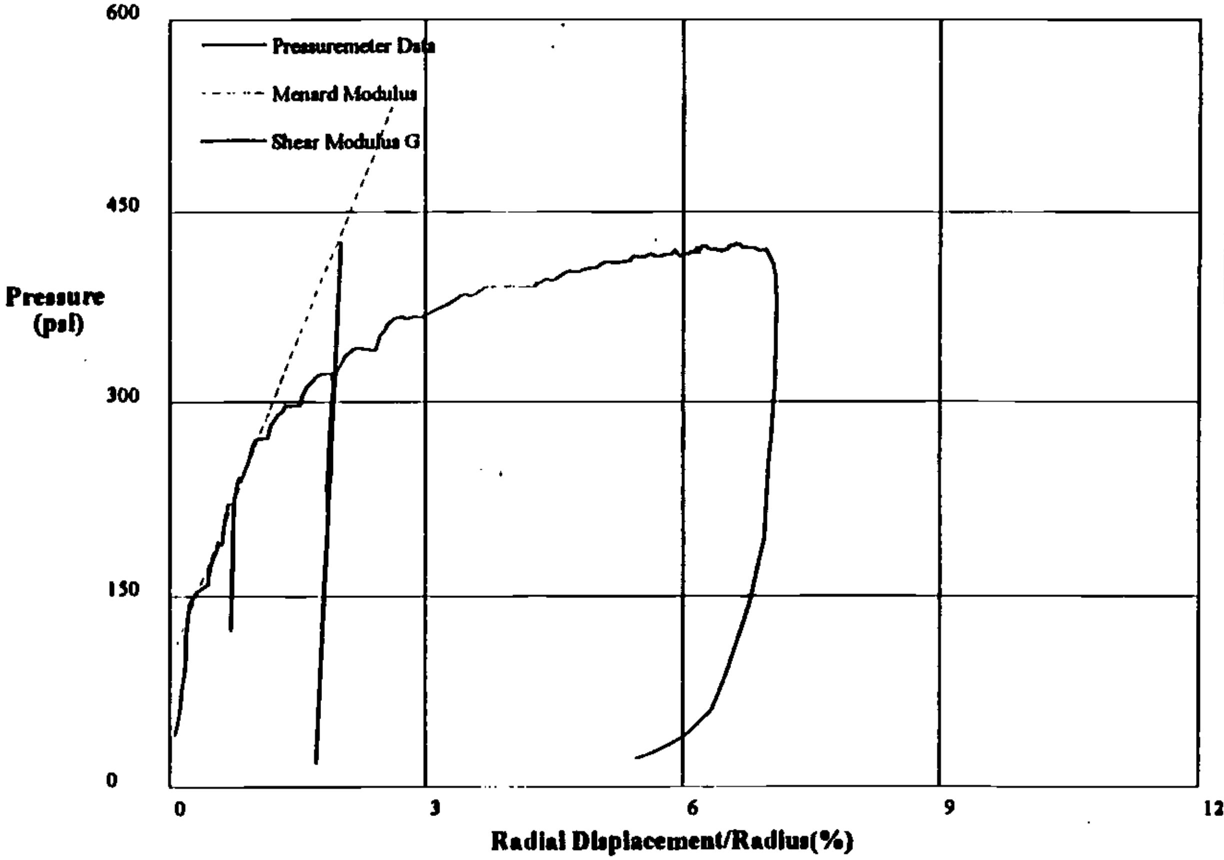


shift 0

HUGHES

Appendix II. Basic Analysis of Pressuremeter Data

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 13 1995**
Hole No. DD27 Depth 47 ft	**File A:\ERTEC3.P**



Menard Modulus Em 21590 psi

Shear Modulus G 72220 psi

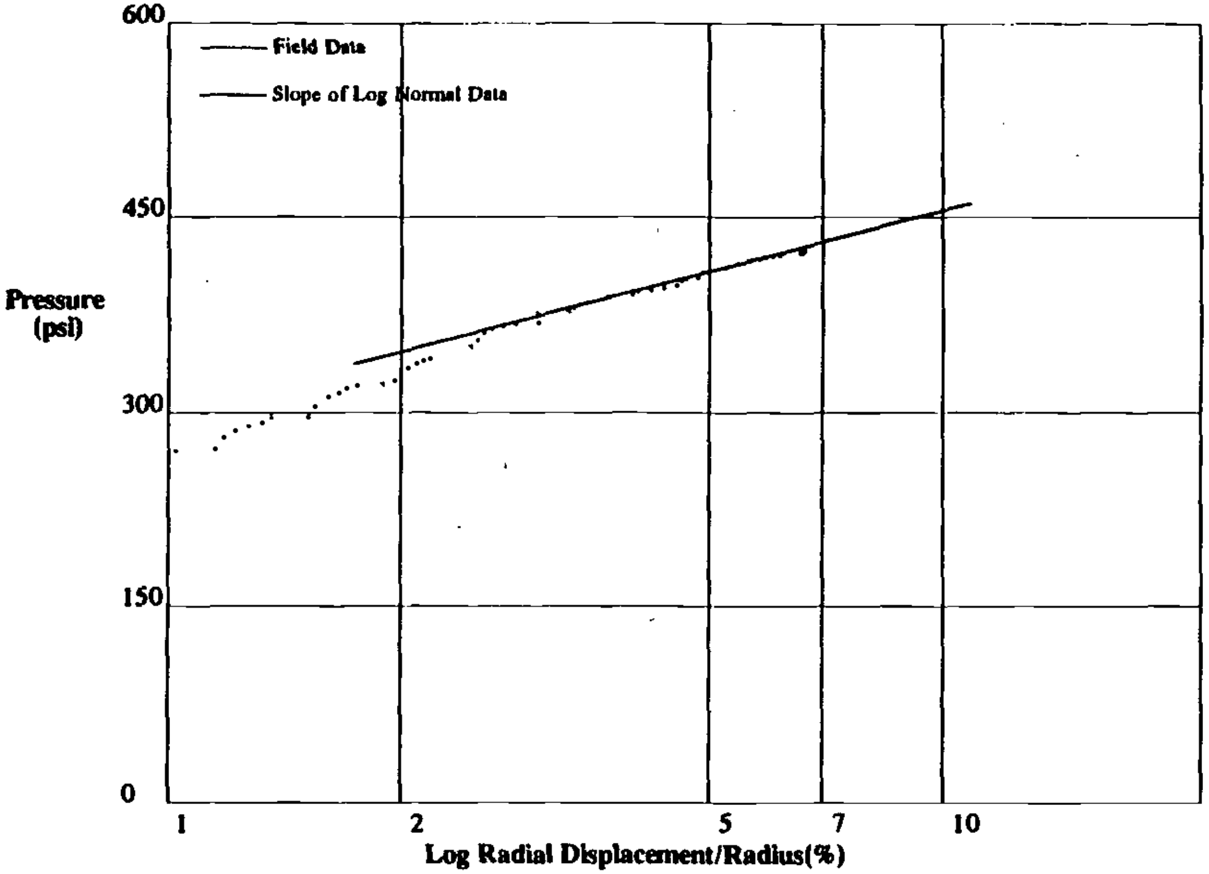
shift 0

HUGHES

Pressuremeter Data | **Geotransit Consultants**

Eastside Extension, Metro Rail Project
Hole No DD27 Depth 47 ft

September 13 1995
D:\DATA\LOSANG\ERTEC3C.P



Shear Strength	**70 psi**
Limit Pressure	**540 psi**

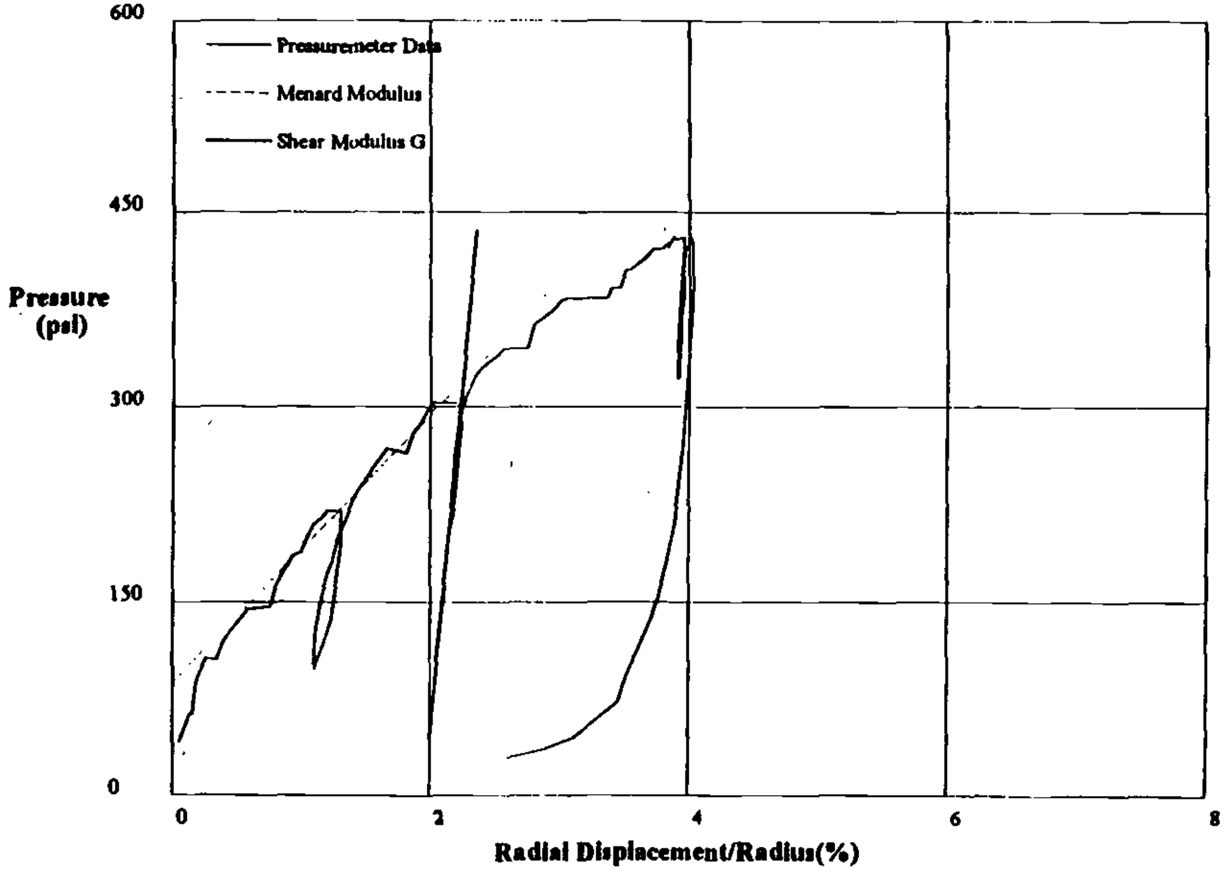
shift 0

HUGHES

Pressuremeter Data Geotransit Consultants

Eastside Extension, Metro Rail Project September 13 1995

Hole No. DD27 Depth 53.5 ft File A:\ERTEC4.P



Menard Modulus Em 13460 psi

Shear Modulus G 55000 psi

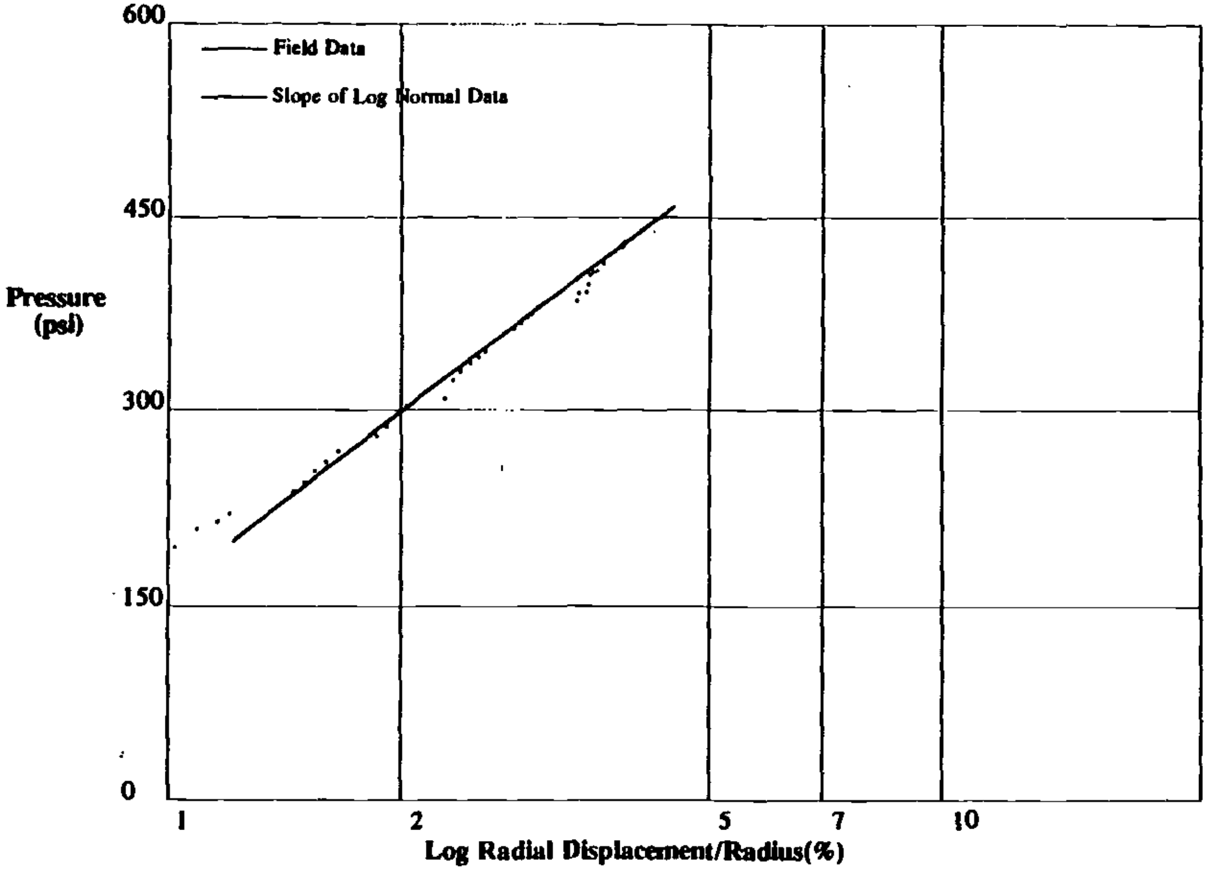
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 13 1995

Hole No DD27 Depth 53.5 ft D:\DATA\LOSANG\ERTEC4C.P



Shear Strength 200 psi
Limit Pressure 860 psi

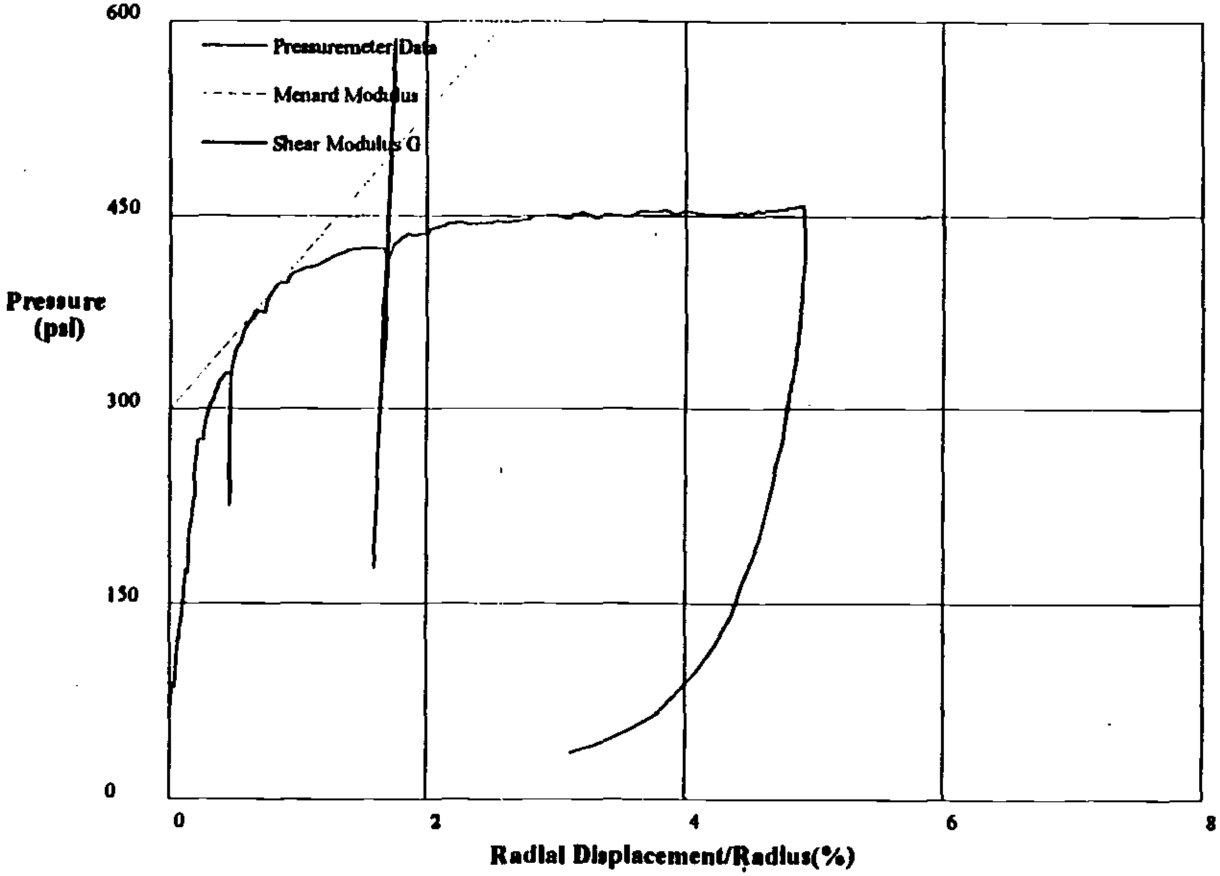
shift 0

HUGHES

Pressuremeter Data | **Geotransit Consultants**

Eastside Extension, Metro Rail Project | **September 14 1995**

Hole No. DD27 **Depth 60 ft** | **File A:\ERTECS.P**



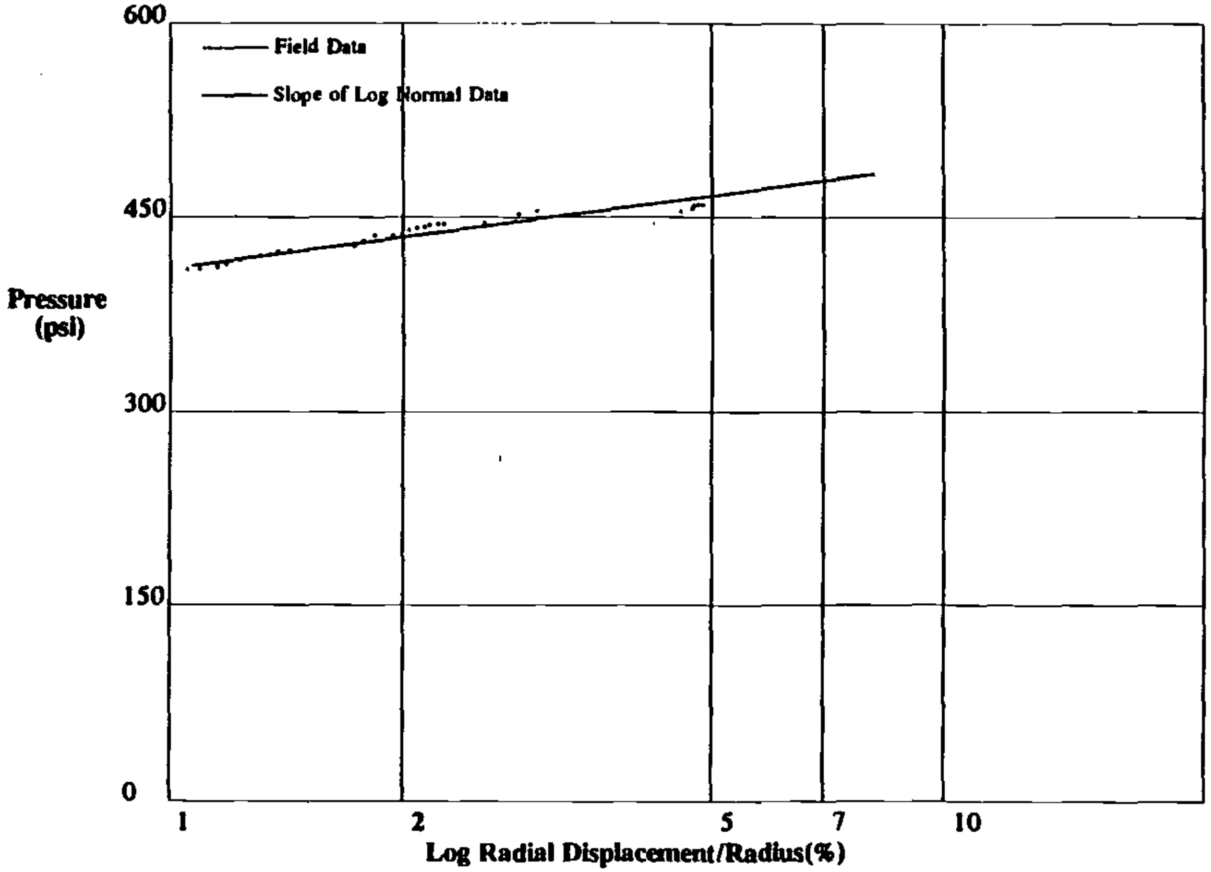
Menard Modulus Em 14940 psi

Shear Modulus G 122500 psi

shift 0

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No DD27 Depth 60 ft	D:\DATA\LOSANG\ERTEC5.P

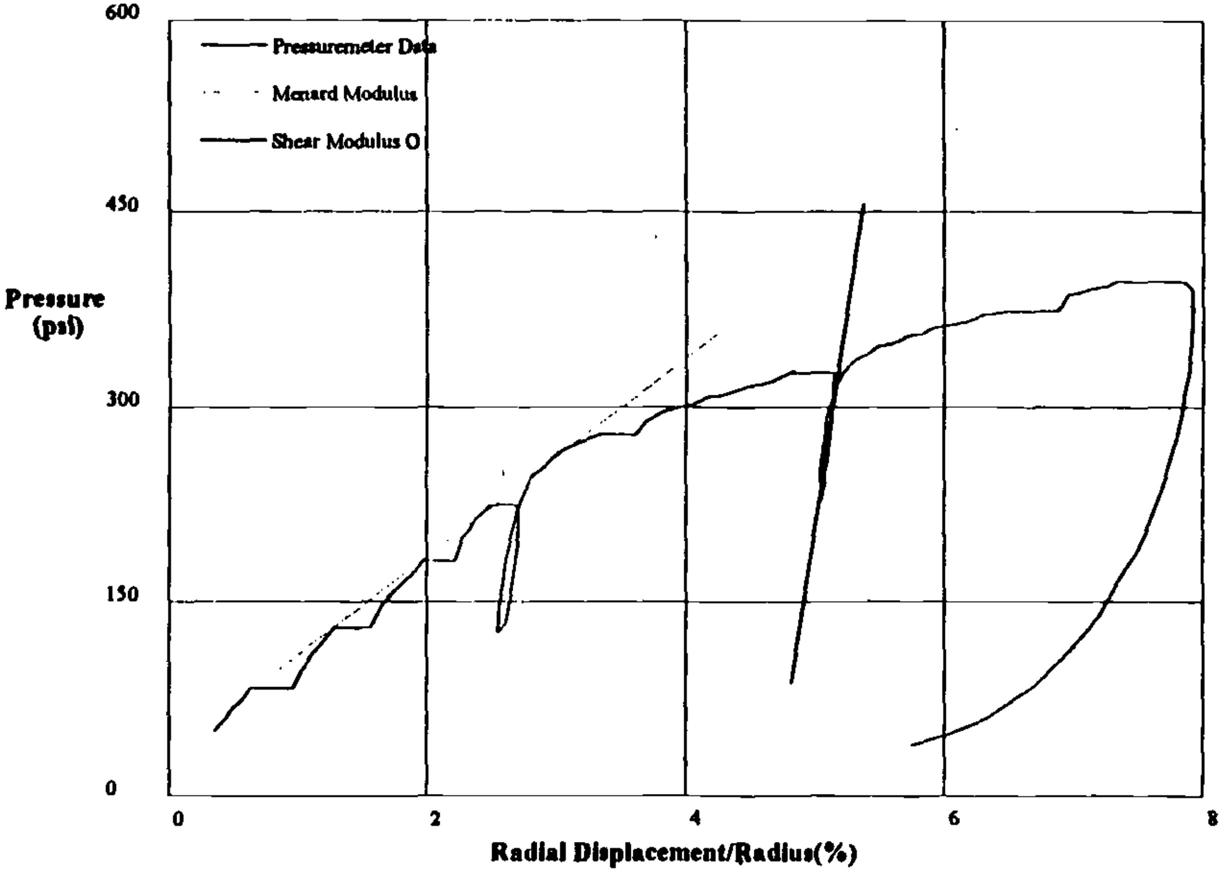


Shear Strength	30 psi
Limit Pressure	530 psi

shift 0

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No. DD27 Depth 66.5 ft	**File A:\ERTEC6.P**



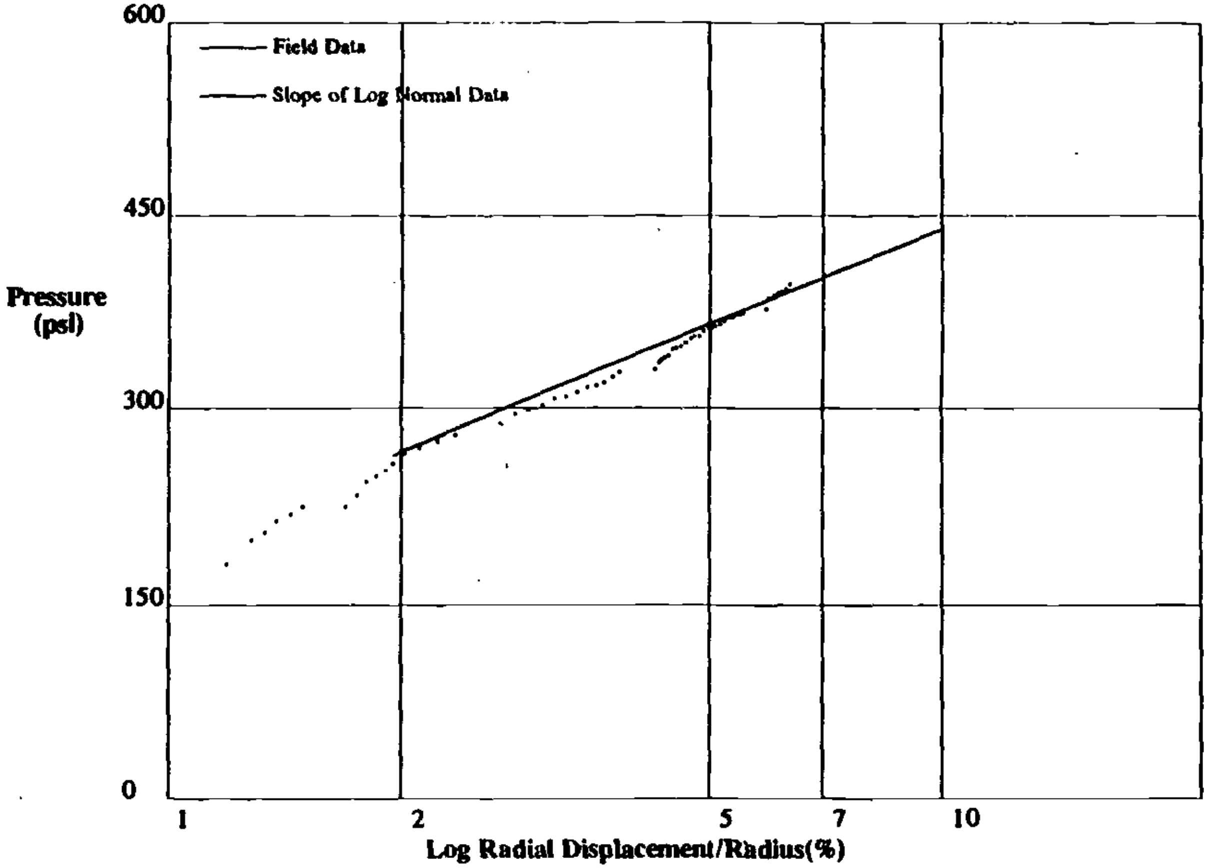
Menard Modulus Em 9900 psi

Shear Modulus G 32780 psi

shift 0

HUGHES

Pressuremeter Data | **Geotransit Consultants**
Eastside Extension, Metro Rail Project | **September 14 1995**
Hole No DD27 Depth 66.5 ft | D:\DATA\LOSANG\ERTEC6.P

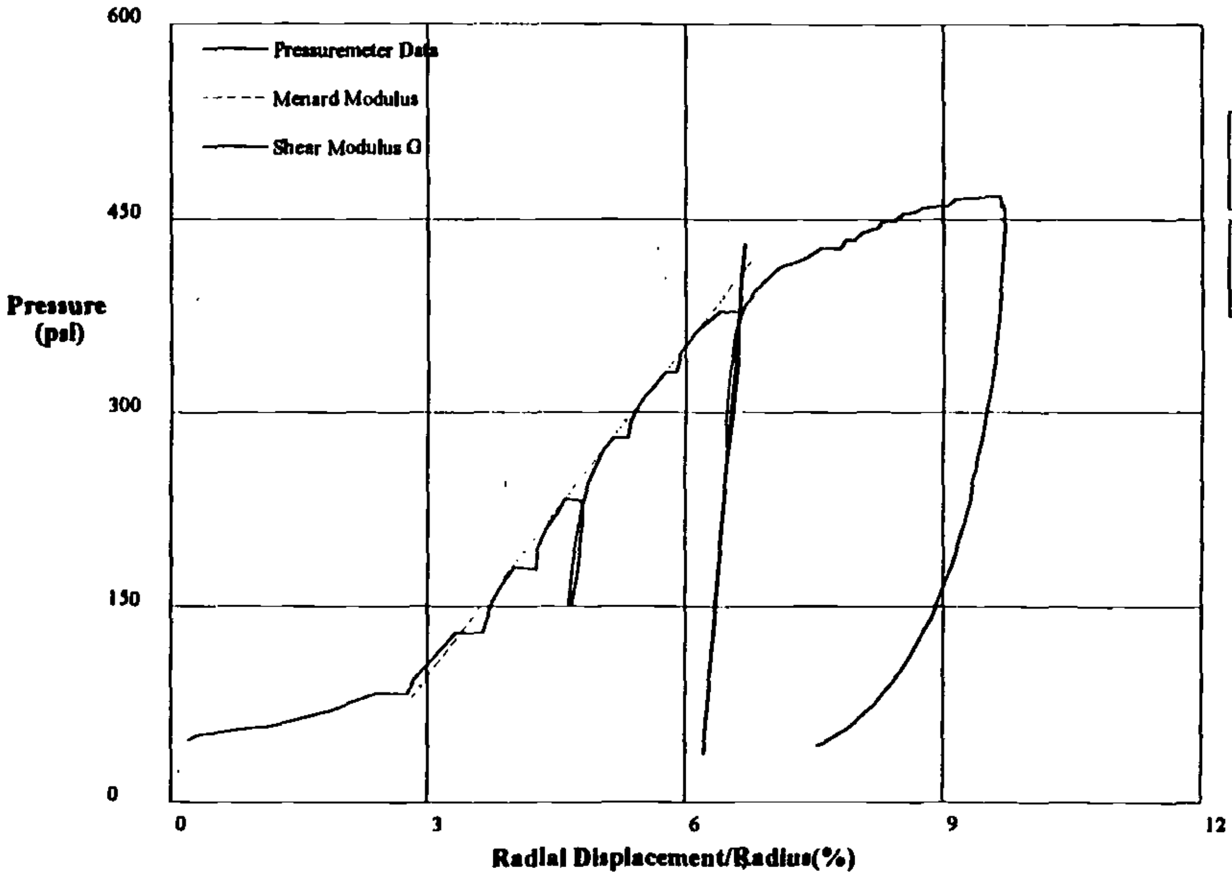


Shear Strength 110 psi
Limit Pressure 570 psi

shift 1

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No. DD27 Depth 73 ft	**File A:\ERTEC7.P**



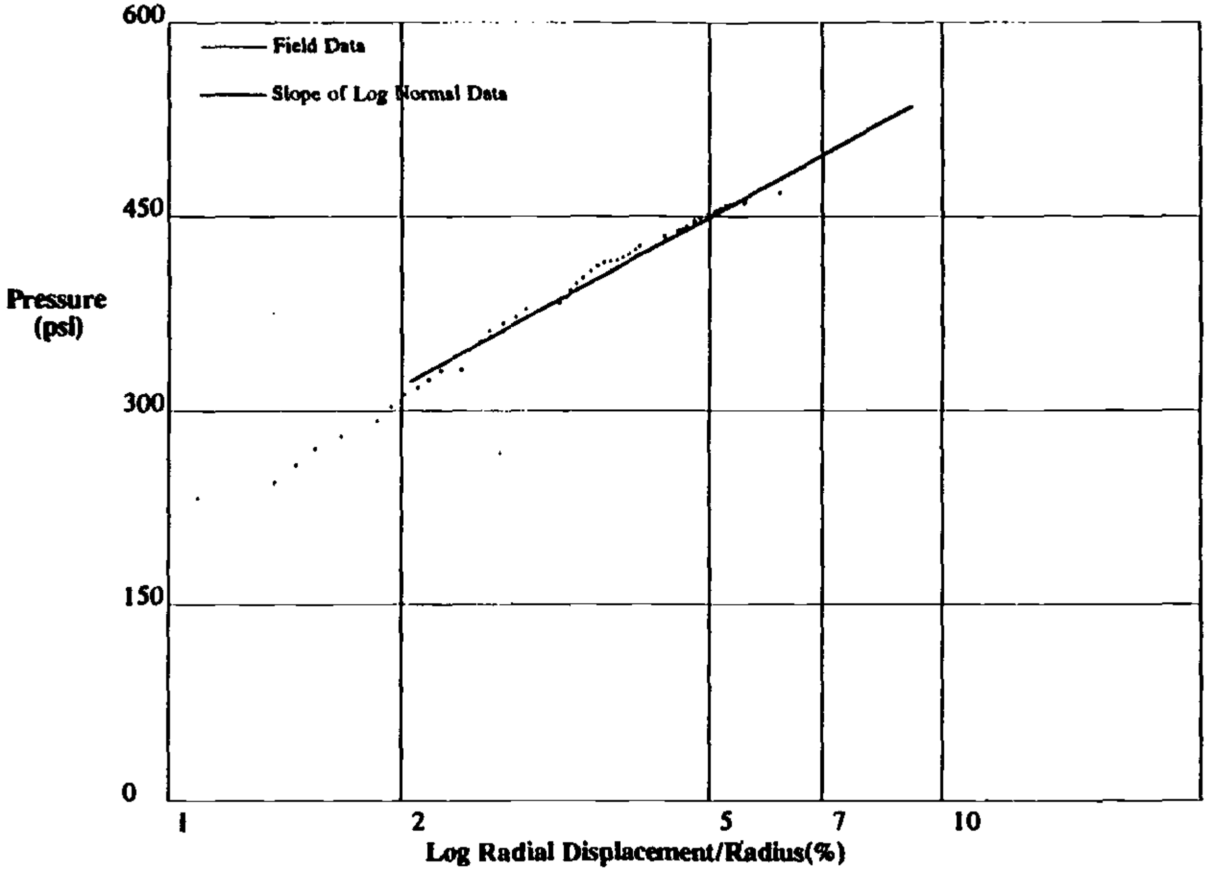
Menard Modulus Em 11060 psi

Shear Modulus G 42000 psi

shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 14 1995**
Hole No DD27 **Depth 73 ft** D:\DATA\LOSANG\ERTEC7.P

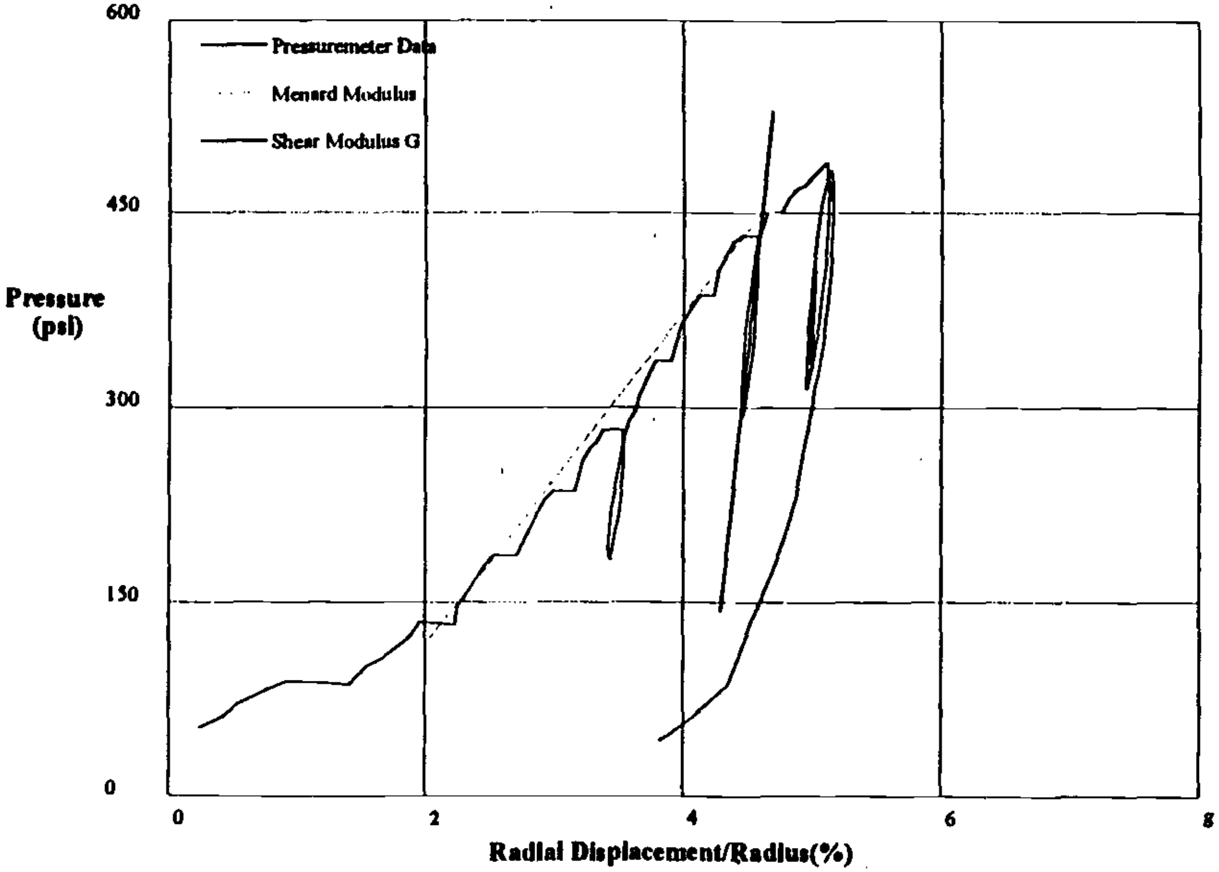


Shear Strength	140 psi
Limit Pressure	720 psi

shift 3.5

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 14 1995**
Hole No. DD27 **Depth 79.5 ft** **File A:\ERTEC8.P**



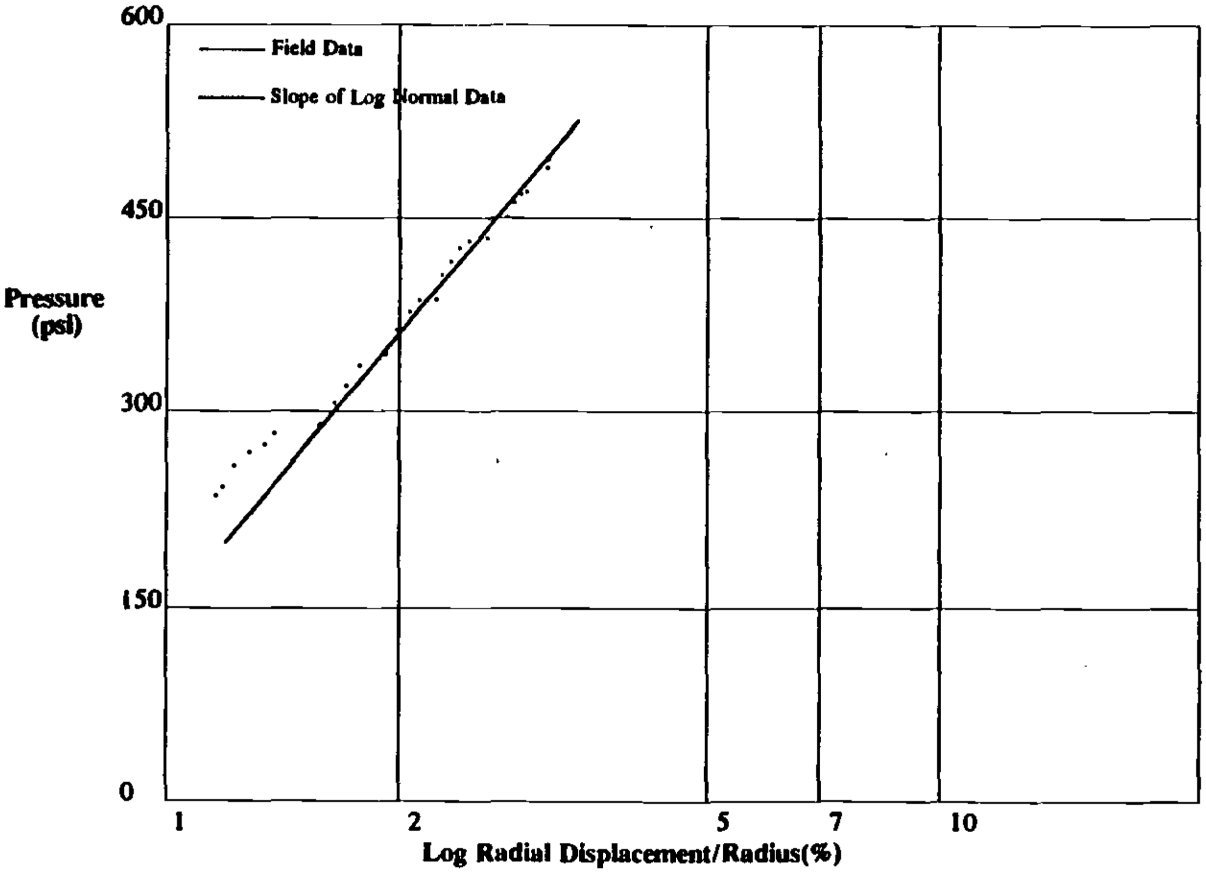
Menard Modulus Em 16600 psi

Shear Modulus G 48680 psi

shift 0

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No DD27 Depth 79.5 ft	D:\DATA\LOSANG\ERTEC8.P

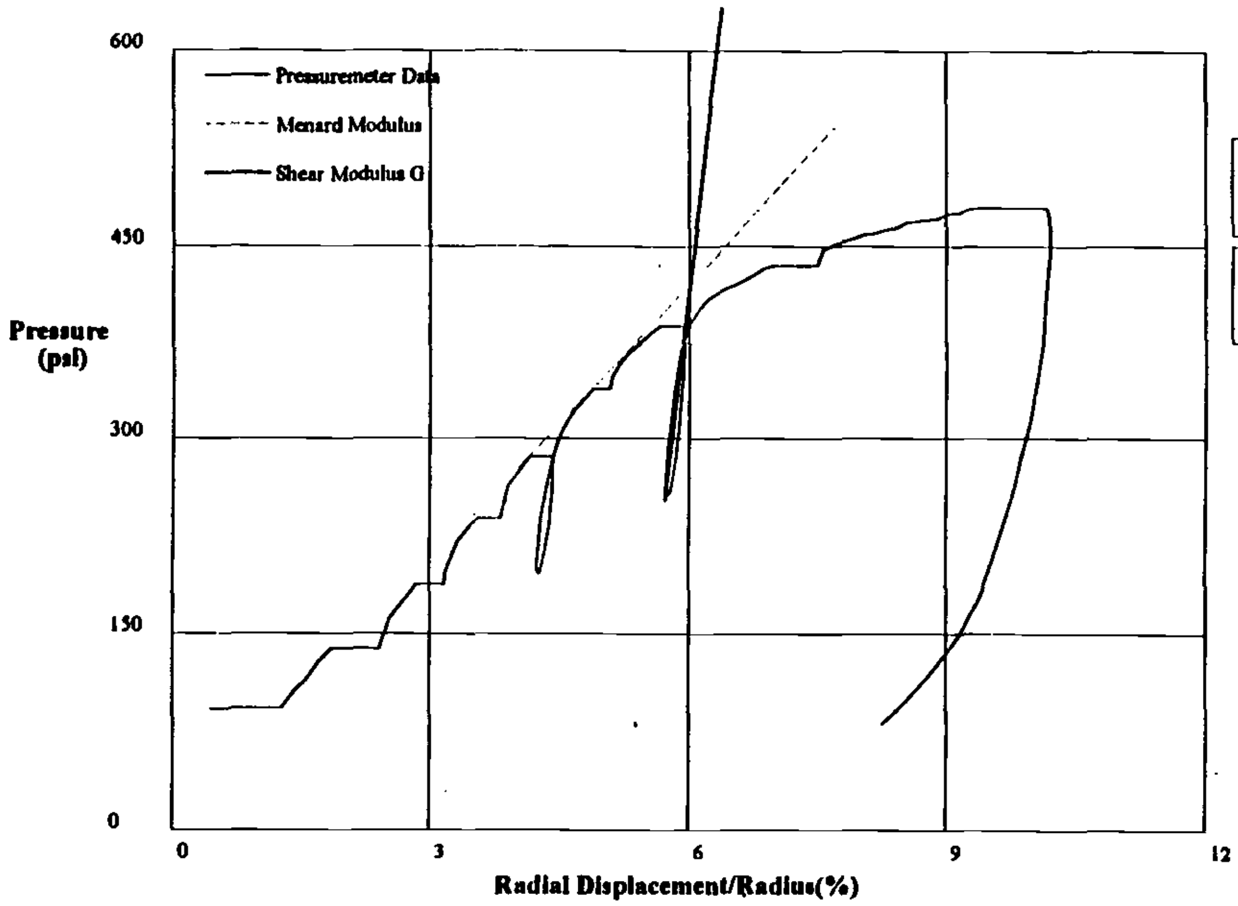


Shear Strength	**310 psi**
Limit Pressure	**1240 psi**

shift 2

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 14 1995**
Hole No. DD27 Depth 86 ft	**File A:\ERTEC9.P**



Menard Modulus Em 9350 psi

Shear Modulus G 29050 psi

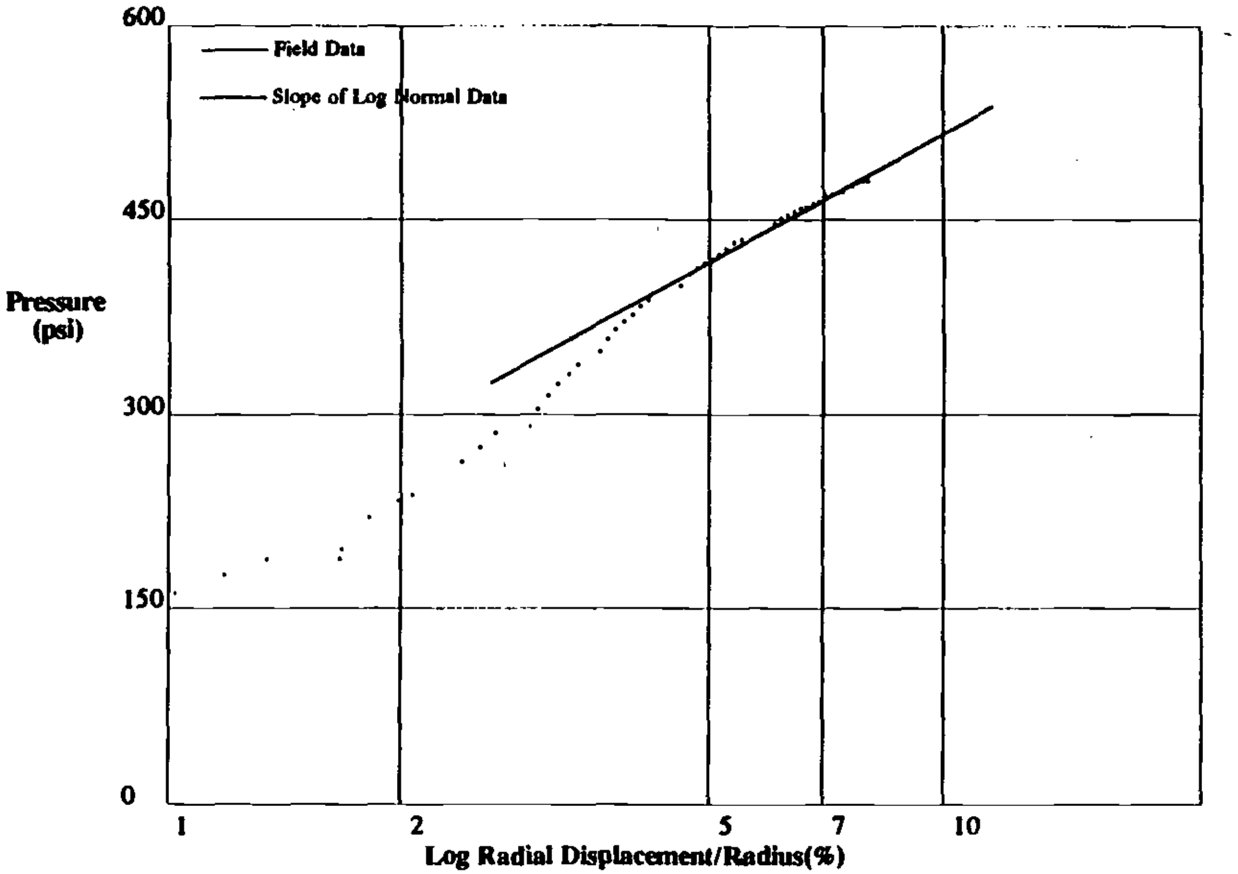
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No DD27 Depth 86 ft D:\DATA\LOSANG\ERTEC9.P

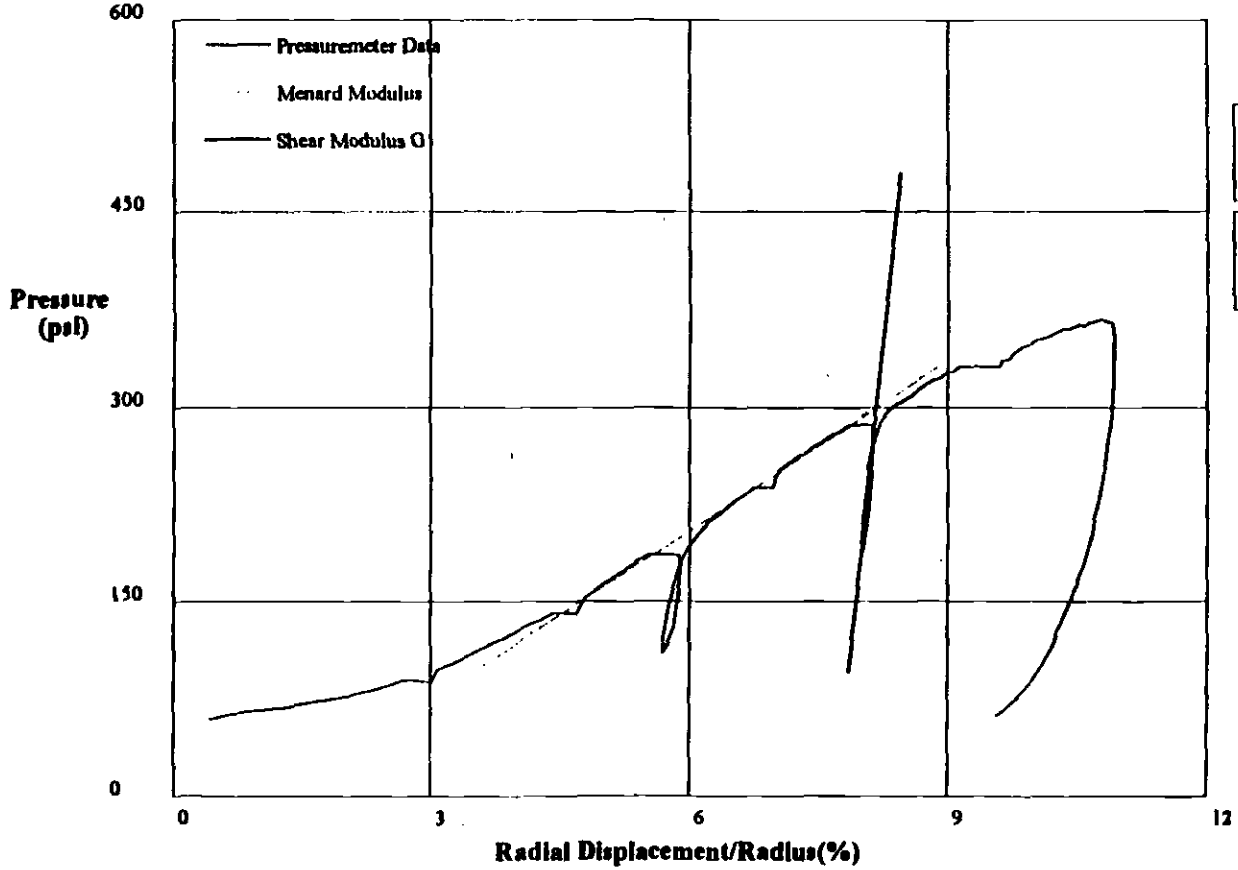


Shear Strength	140	psi
Limit Pressure	690	psi

shift 1.5

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 15 1995**
Hole No. DD27 **Depth 92.5 ft** **File A:\ERTEC10.P**



Menard Modulus Em 5680 psi

Shear Modulus G 31540 psi

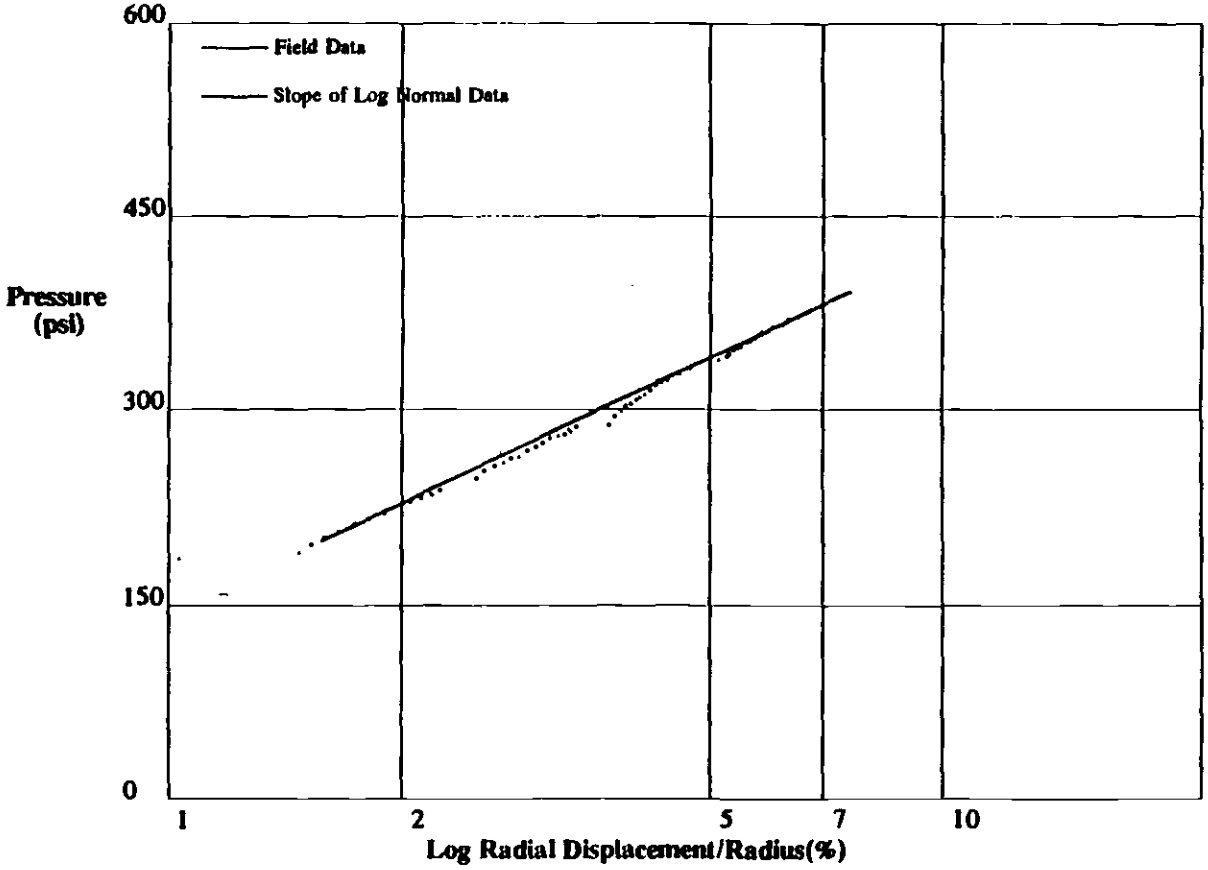
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **September 15 1995**

Hole No DD27 **Depth 92.5 ft** D:\DATA\LOSANG\ERTEC10.P



Shear Strength	**120 psi**
Limit Pressure	**570 psi**

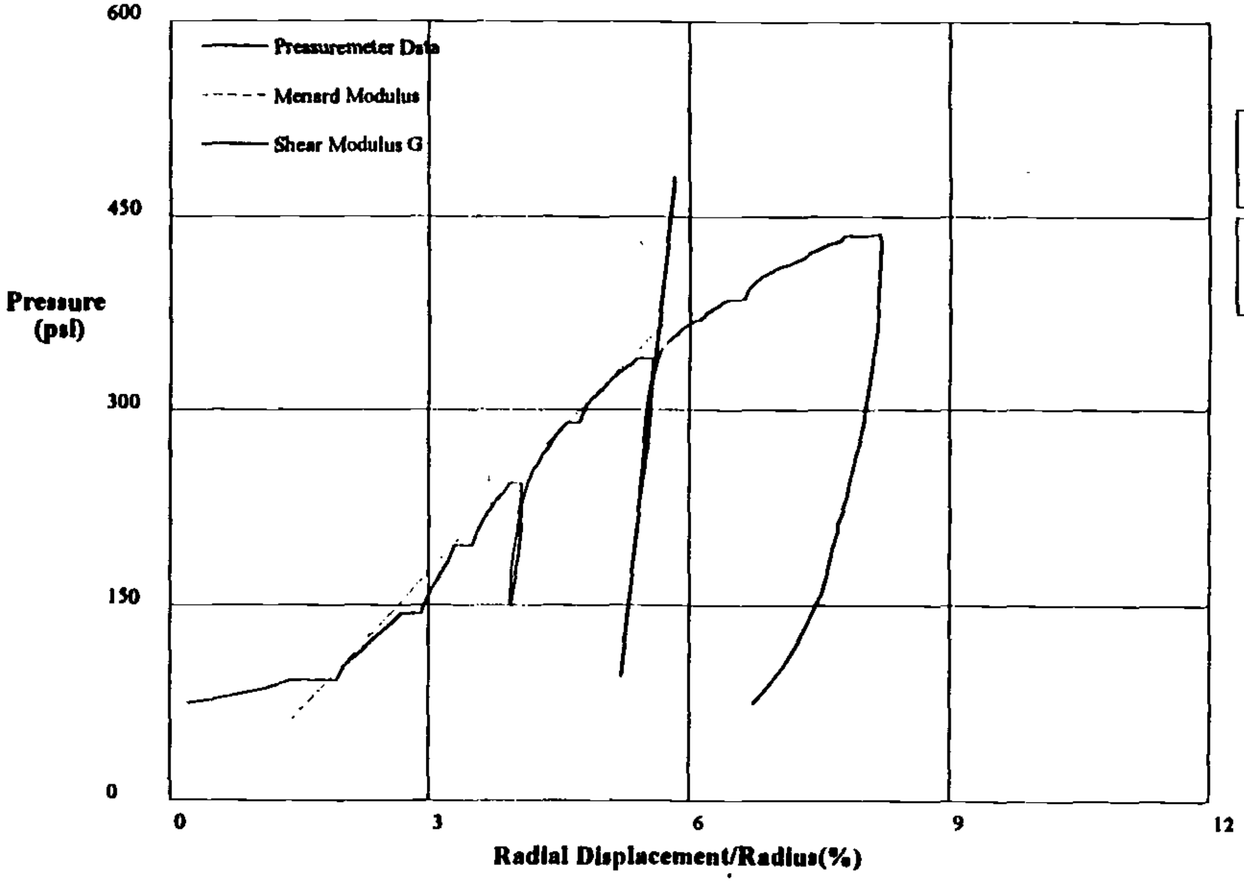
shift 4.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **September 15 1995**

Hole No. DD27 **Depth 99.5 ft** **File A:\ERTEC11.P**



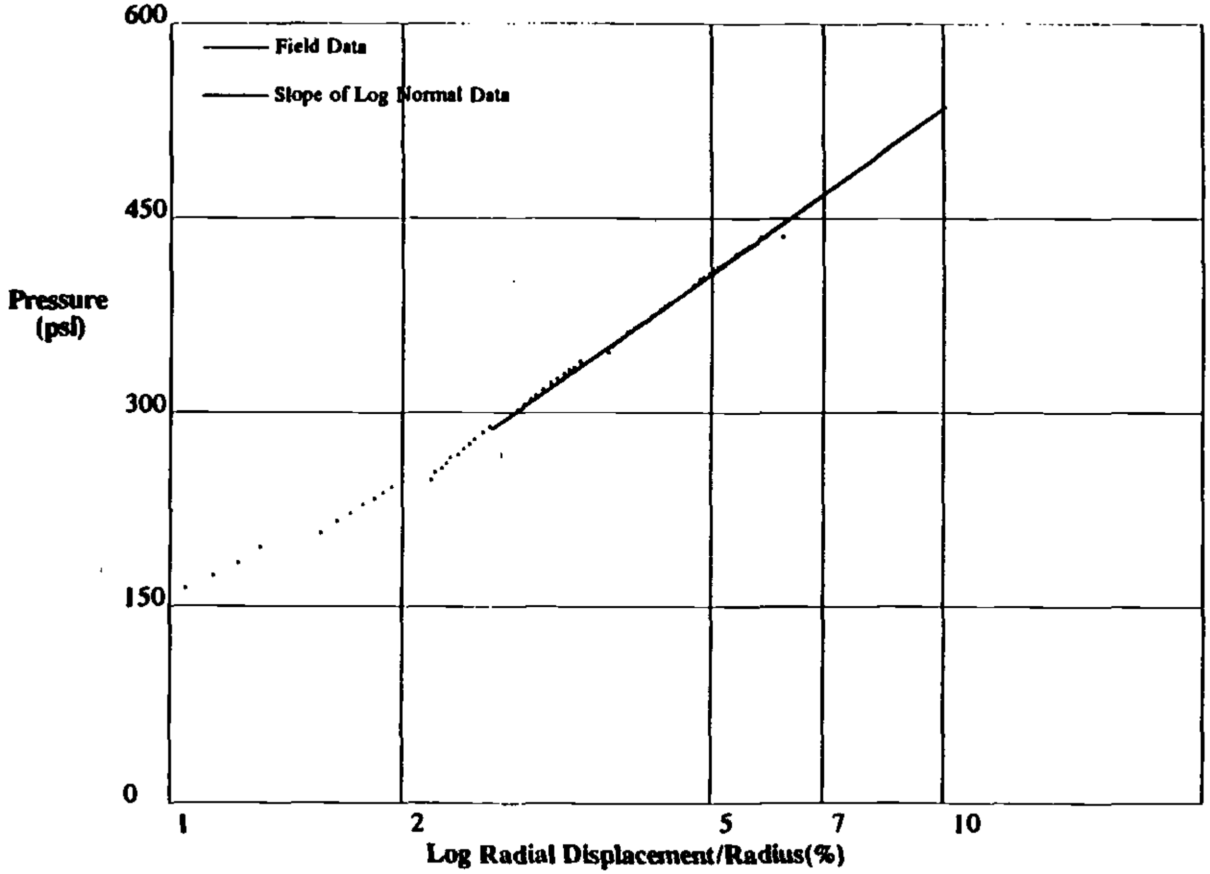
Menard Modulus Em 9220 psi

Shear Modulus G 31540 psi

shift 3

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 15 1995**
Hole No DD27 **Depth 99.5 ft** D:\DATA\LOSANG\ERTEC11.P

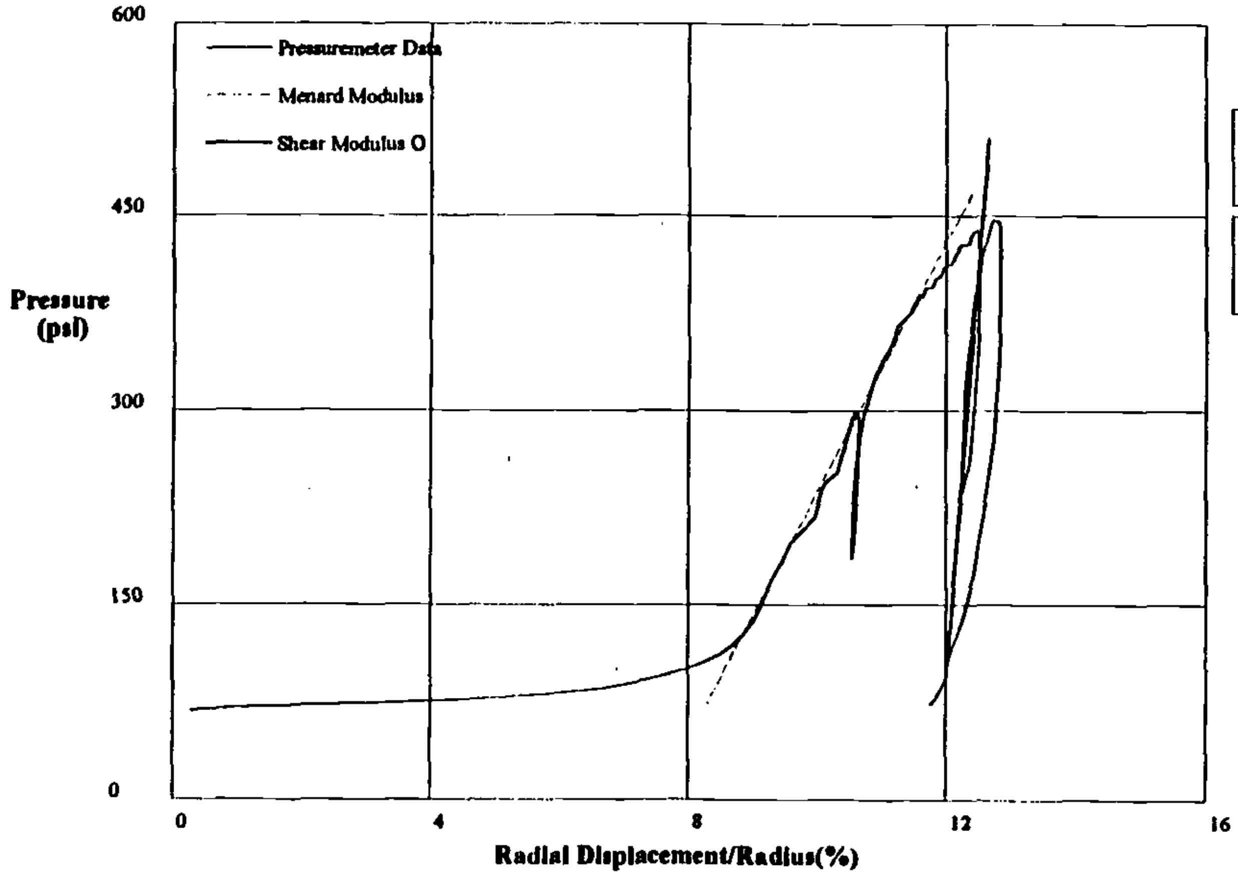


Shear Strength	**180 psi**
Limit Pressure	**760 psi**

shift 5

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	**September 15 1995**
Hole No. DD27 Depth 112.5 ft	**File A:\ERTEC13.P**



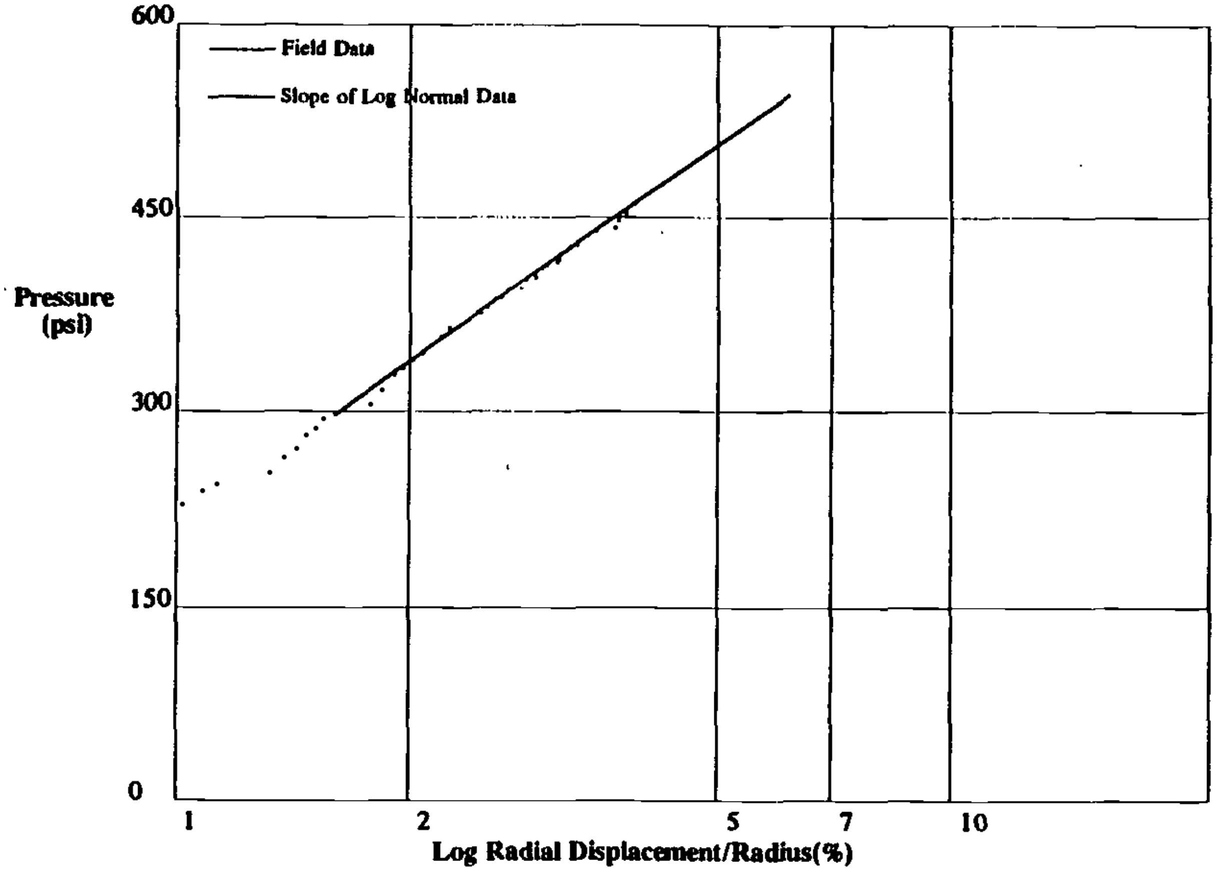
Menard Modulus Em 12500 psi

Shear Modulus G 32670 psi

shift 0

HUGHES

Pressuremeter Data Geotransit Consultants
Eastside Extension, Metro Rail Project September 15 1995
Hole No DD27 Depth 112.5 ft D:\DATA\LOSANG\ERTEC13.P



Shear Strength	180	psi
Limit Pressure	860	psi

shift 9

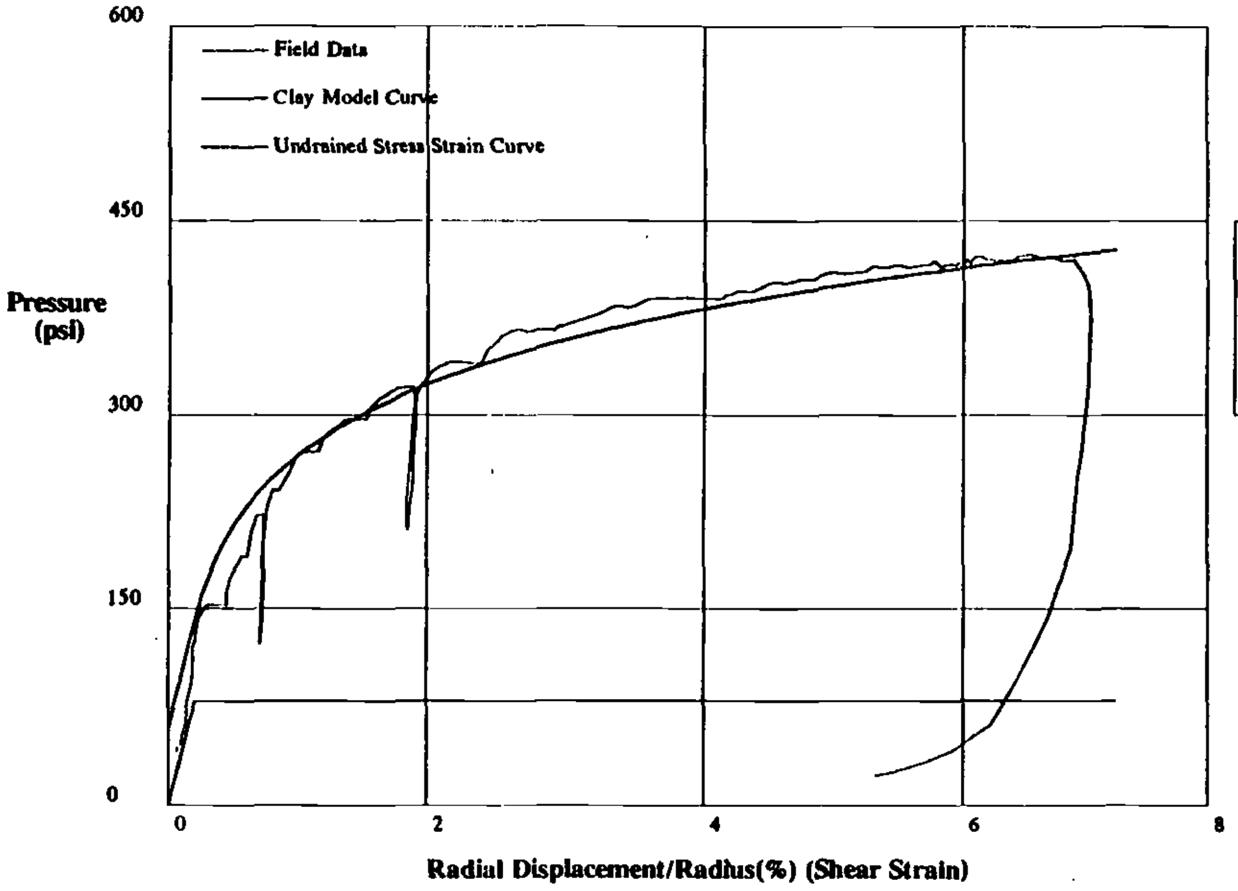
HUGHES

Appendix III. Alternative Analysis of Pressuremeter Data assuming a Cohesive Model

Pressuremeter Data

September 13 1995

Hole No. DD27 Depth 47 ft File D:\DATA\LOSANG\ERTEC3.P



GIBSON'S CLAY MODEL

Shear Strength	80 psi
Insitu Stress	60 psi
Shear Modulus	20000 psi

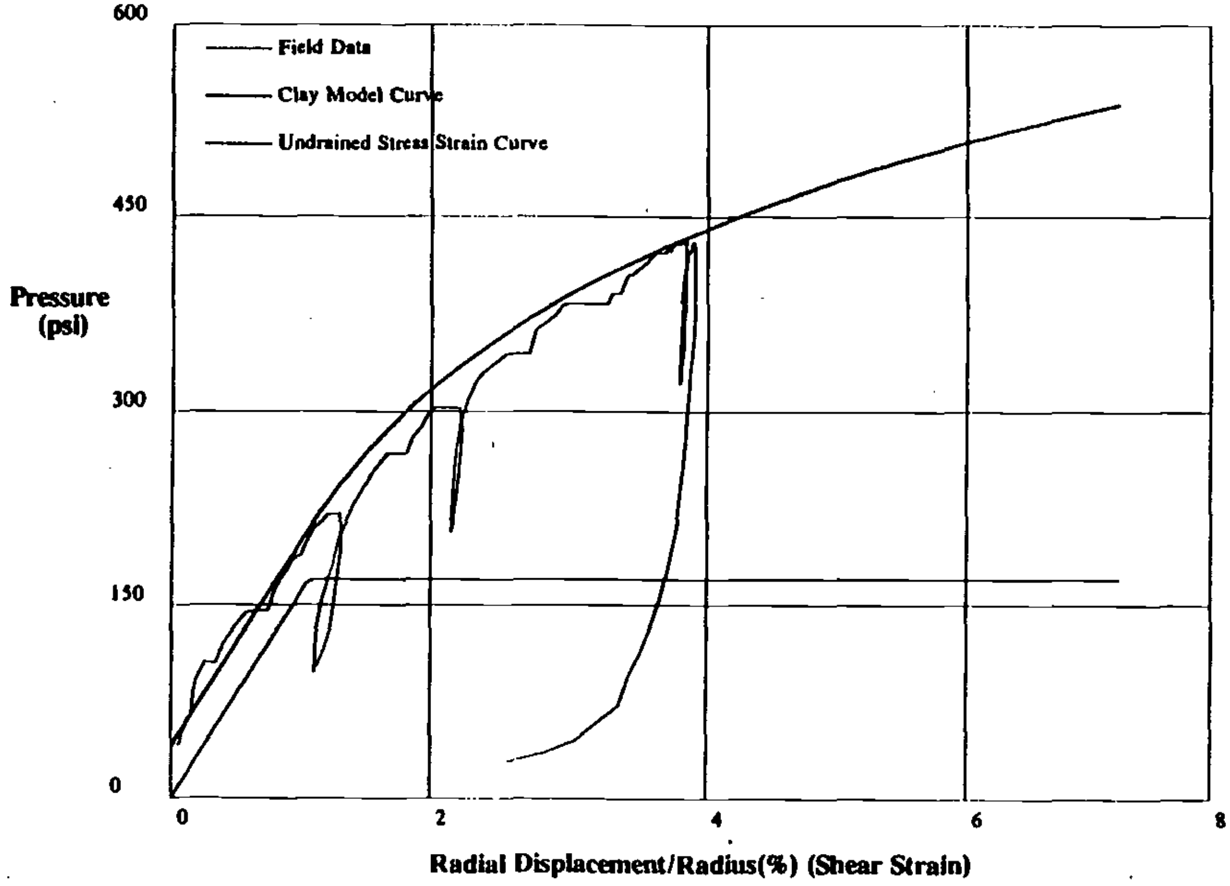
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 13 1995

Hole No. DD27 Depth 53.5 ft File D:\DATA\LOSANG\ERTEC4.P



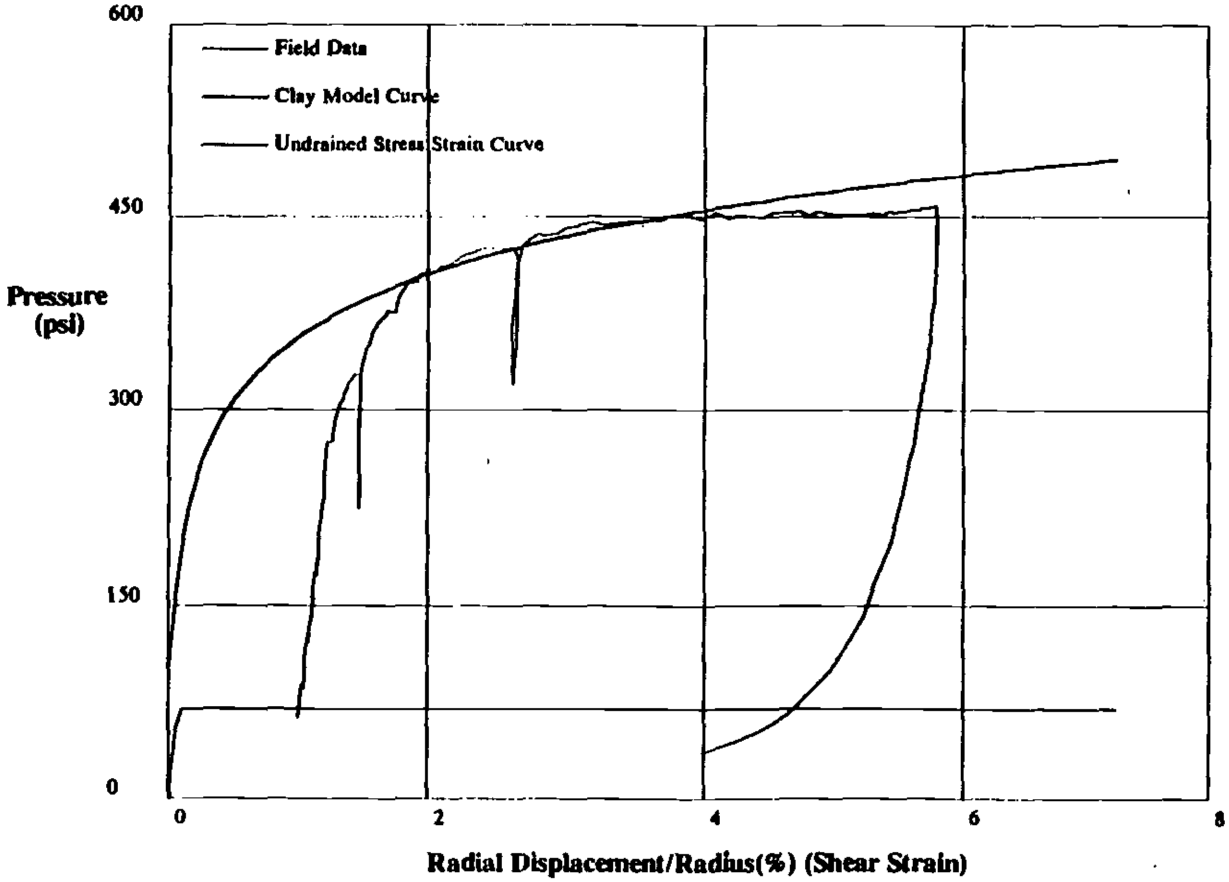
GIBSON'S CLAY MODEL

Shear Strength	170 psi
Insitu Stress	40 psi
Shear Modulus	8000 psi

shift 0

HUGHES

Pressuremeter Data	**Geotransit Consultants**
Eastside Extension, Metro Rail Project	September 14 1995
Hole No. DD27 Depth 60 ft	File D:\DATA\LOSANG\ERTEC5.P



GIBSON'S CLAY MODEL

Shear Strength	70 psi
Insitu Stress	100 psi
Shear Modulus	50000 psi

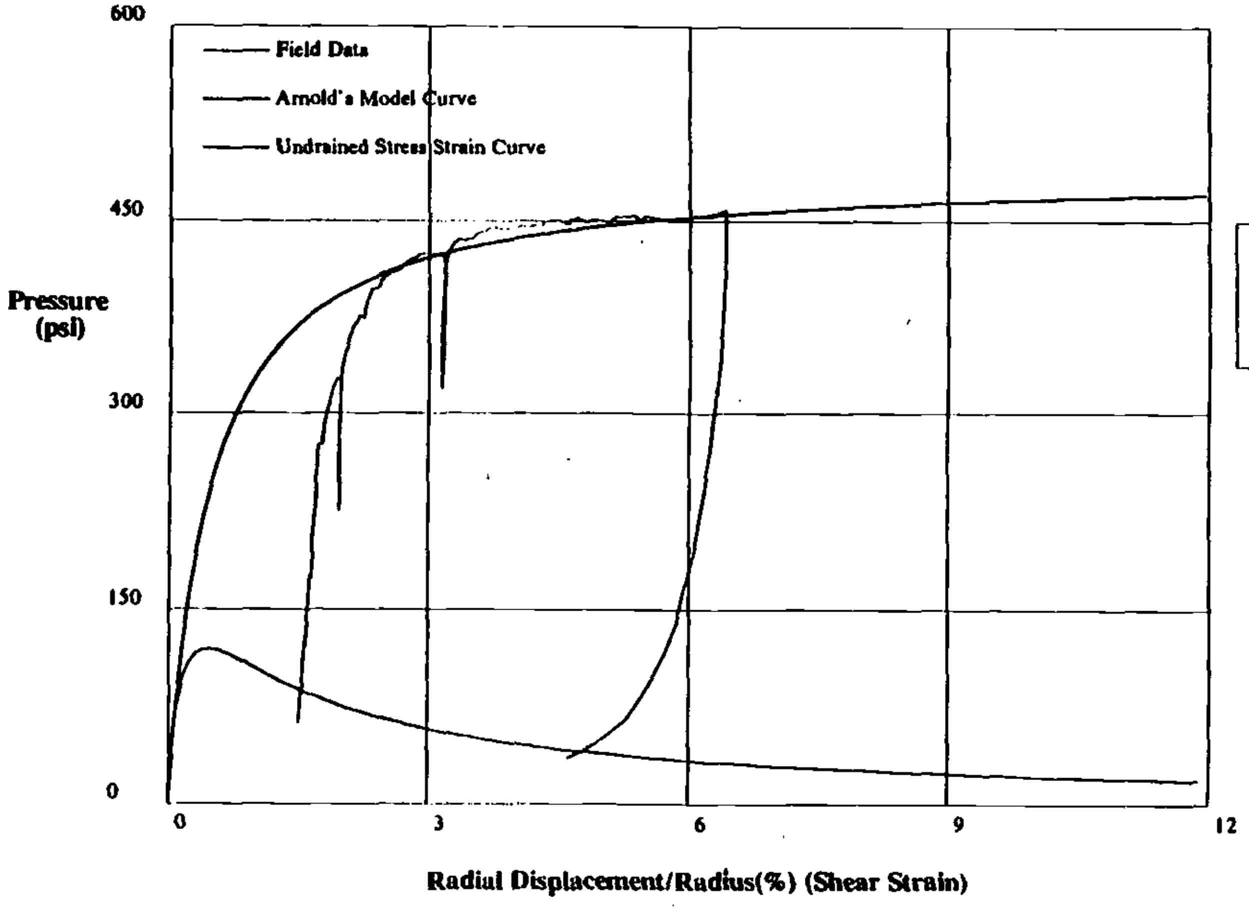
shift-f

HUGHES

Pressuremeter Data | **Geotransit Consultants**

Eastside Extension, Metro Rail Project | September 14 1995

Hole No. DD27 Depth 60 ft | File D:\DATA\LOSANG\ERTEC5.P



ARNOLD'S CLAY MODEL

Shear Strength at
10 % Shear Strain = 21 psi

Shear Stress (psi)

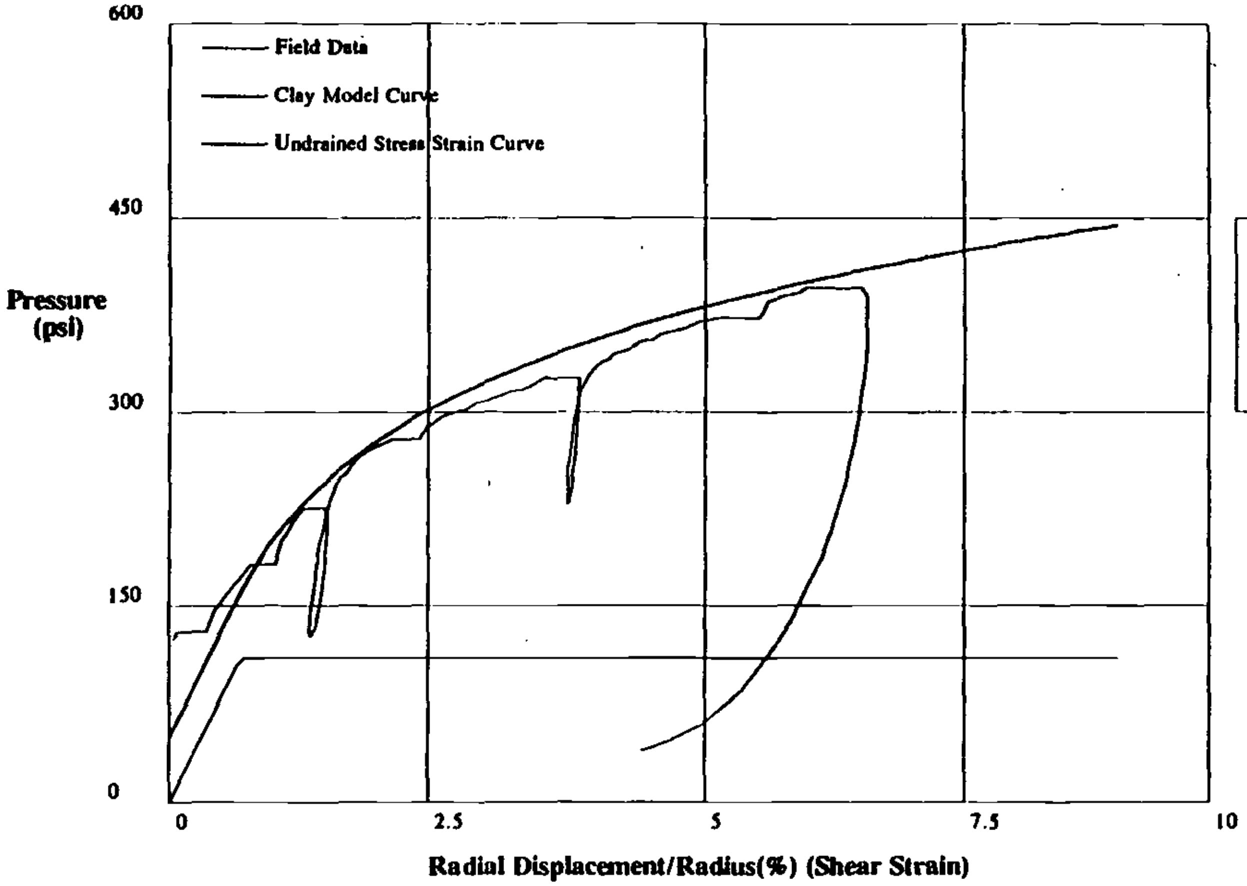
330 410 460 shift-1.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No. DD27 Depth 66.5 ft File D:\DATA\LOSANG\ERTEC6.P



GIBSON'S CLAY MODEL

Shear Strength	110 psi
Insitu Stress	50 psi
Shear Modulus	8000 psi

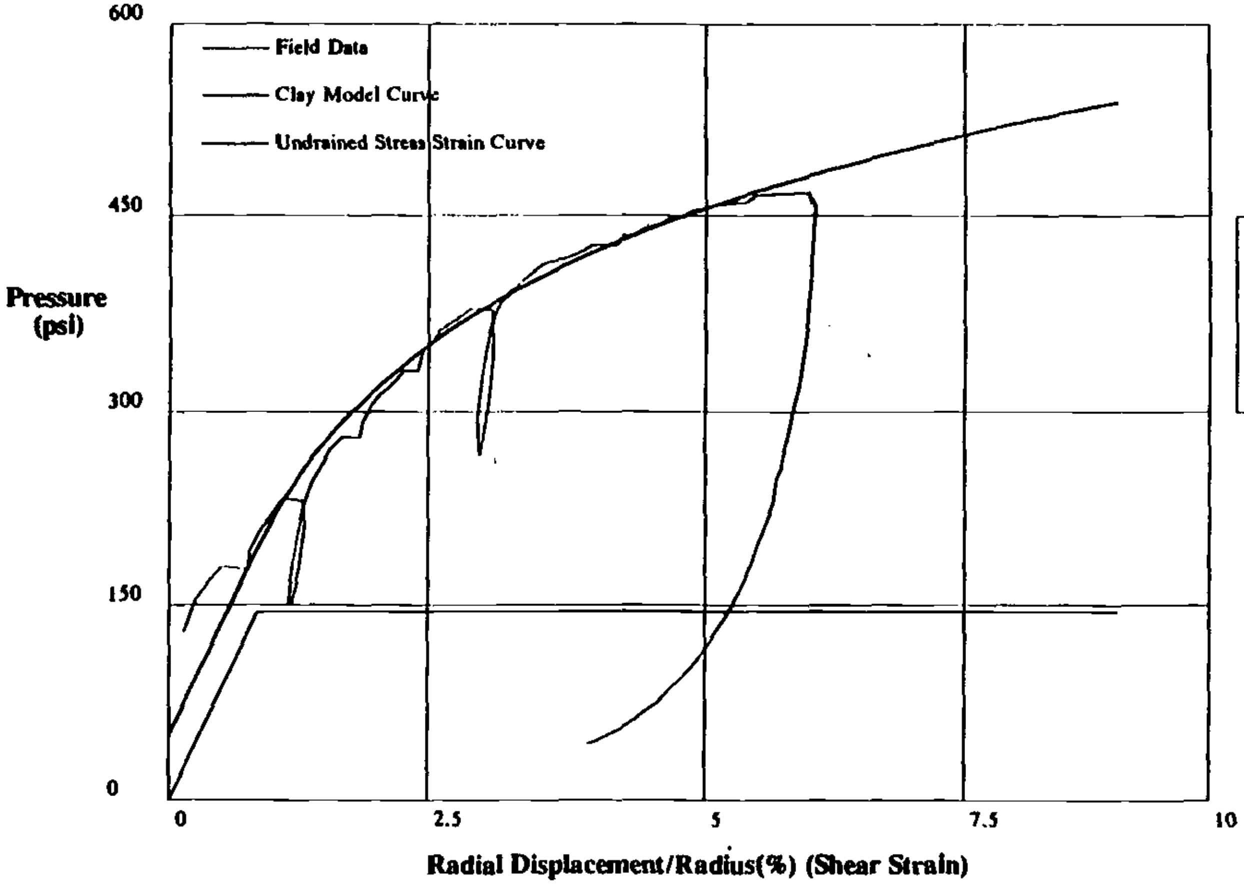
shift 1.2

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No. DD27 Depth 73 ft File D:\DATA\LOSANG\ERTEC7.P



GIBSON'S CLAY MODEL

Shear Strength	145	psi
Insitu Stress	50	psi
Shear Modulus	8500	psi

Shear Stress (psi)

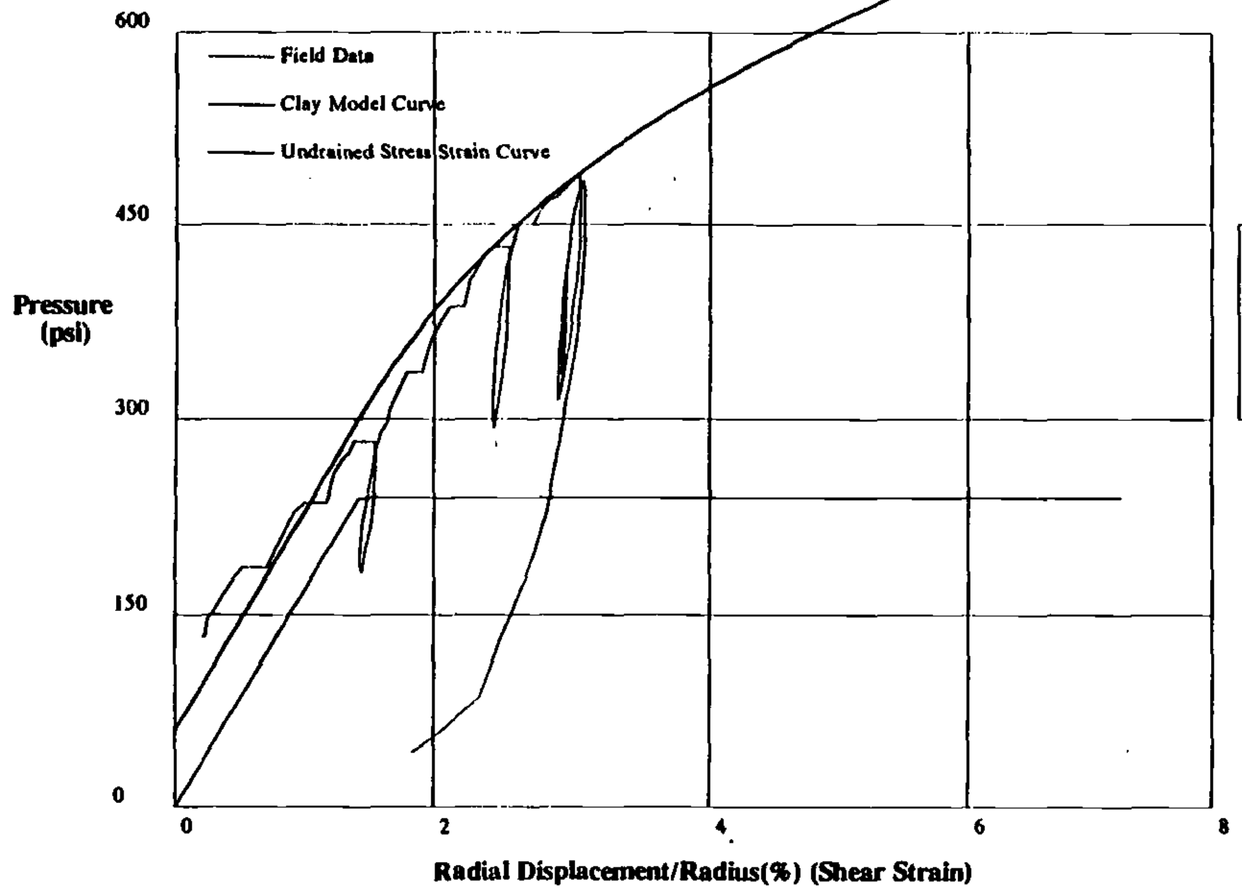
shift 3.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No. DD27 Depth 79.5 ft File D:\DATA\LOSANG\ERTEC8.P



GIBSON'S CLAY MODEL

Shear Strength	240 psi
Insitu Stress	60 psi
Shear Modulus	8500 psi

Shear Stress (psi)

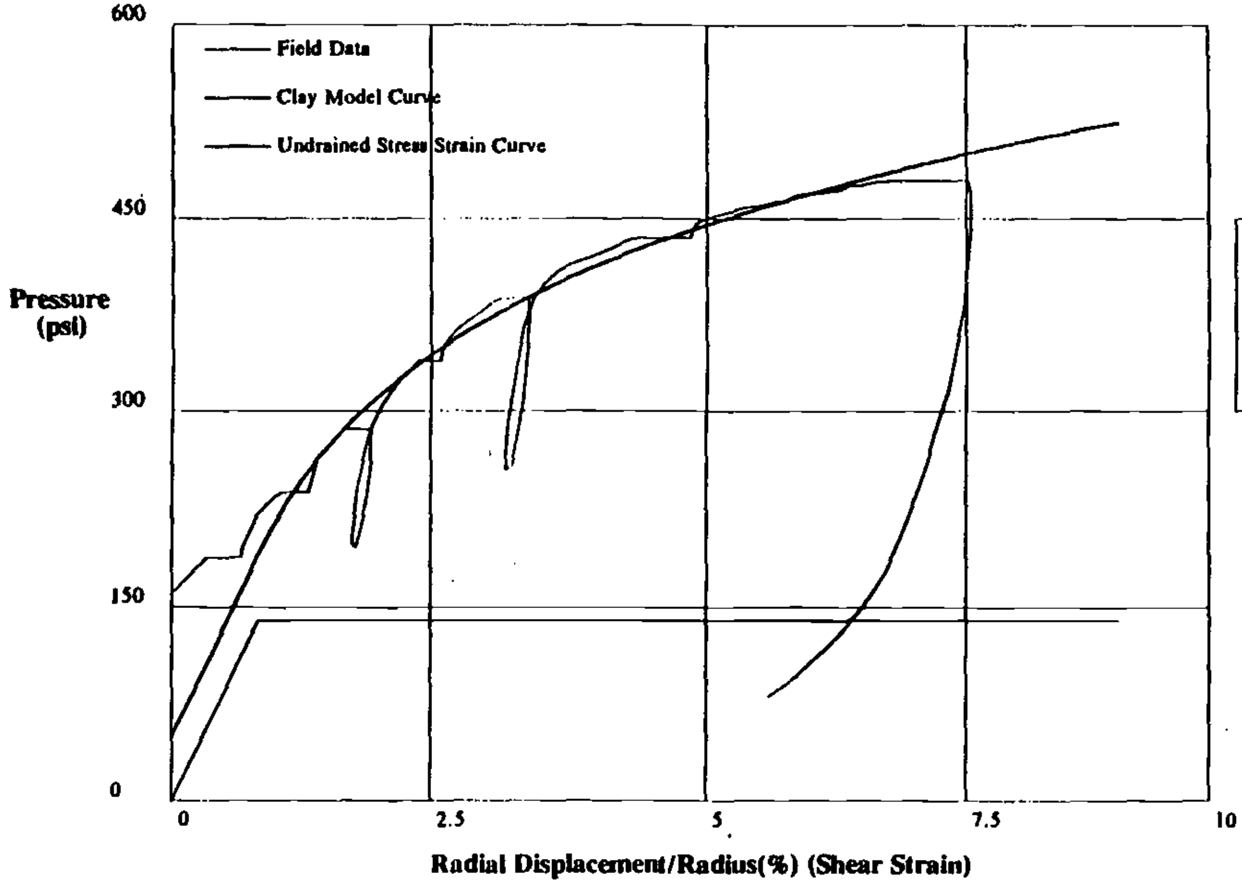
shift 2

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No. DD27 Depth 86 ft File D:\DATA\LOSANG\ERTEC9.P



GIBSON'S CLAY MODEL

Shear Strength	140 psi
Insitu Stress	50 psi
Shear Modulus	8400 psi

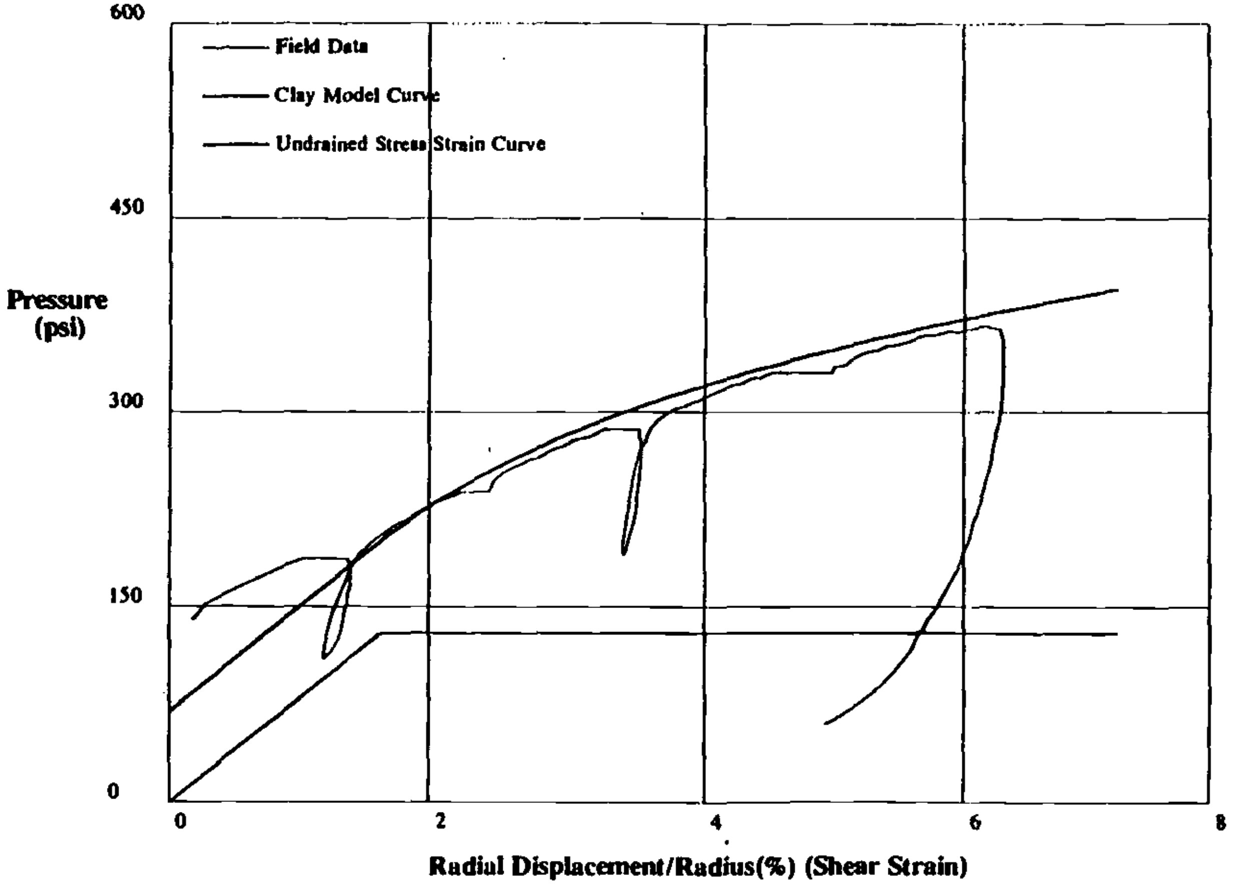
shift 2.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 15 1995

Hole No. DD27 Depth 92.5 File D:\DATA\LOSANG\ERTEC10.P



GIBSON'S CLAY MODEL

Shear Strength	130 psi
Insitu Stress	70 psi
Shear Modulus	4000 psi

Shear Stress (psi)

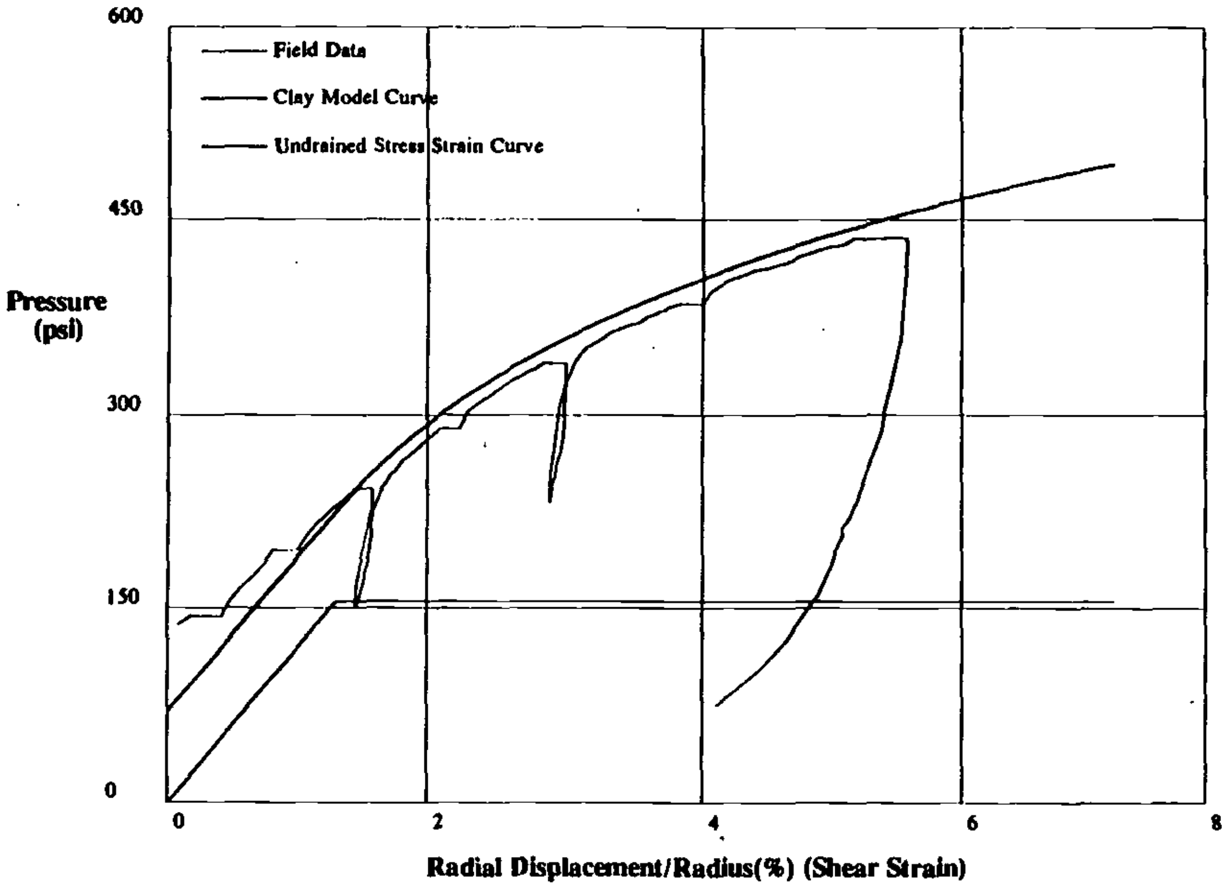
shift 4.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 15 1995

Hole No. 0D27 Depth 99.5 ft File O:\OATA\LOSANG\ERTEC11.P



GIBSON'S CLAY MODEL

Shear Strength	155 psi
Insitu Stress	70 psi
Shear Modulus	6000 psi

Shear Stress (psi)

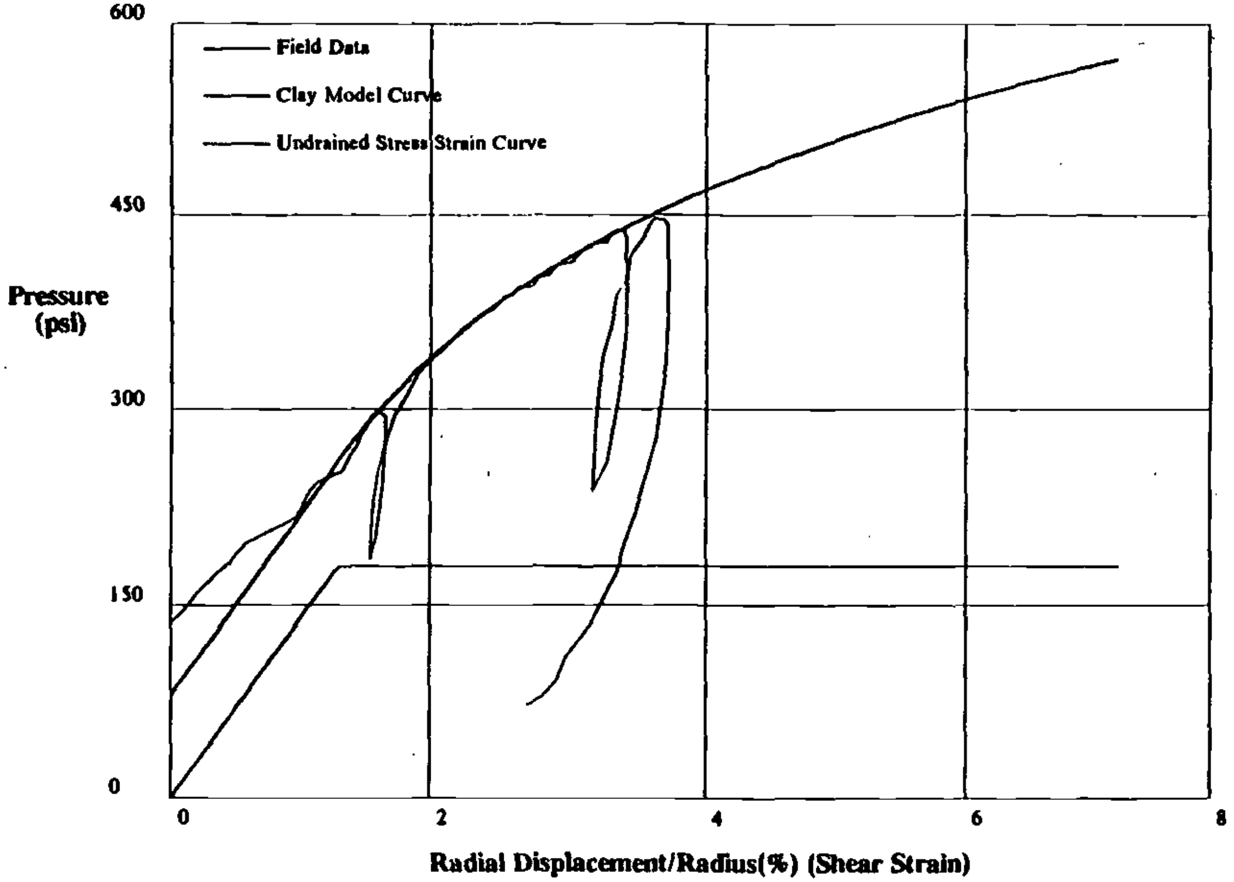
shift 5.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 15 1995

Hole No. DD27 Depth 112.5 ft File D:\DATA\LOSANG\ERTEC13.P



GIBSON'S CLAY MODEL

Shear Strength	180 psi
Insitu Stress	80 psi
Shear Modulus	7000 psi

Shear Stress (psi)

shift 9

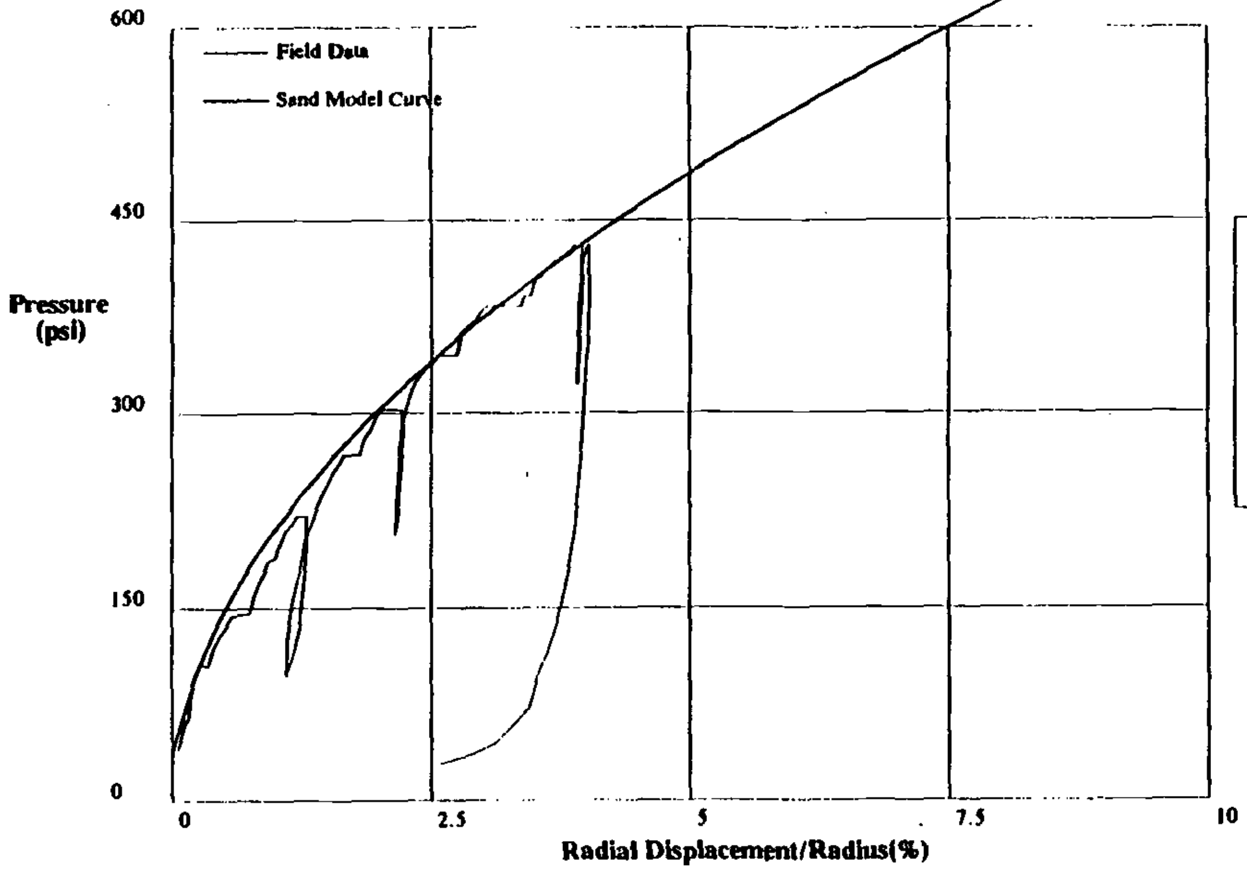
HUGHES

Appendix IV. Alternative Analysis of Pressuremeter Data assuming a Frictional Model

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **September 13 1995**

Hole No. DD27 **Depth 53.5 ft** **File D:\DATA\LOSANG\ERTEC4.P**



THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	44 deg
Critical Friction Angle	32 deg
Lateral Stress	40 psi
Shear Modulus	12600 psi

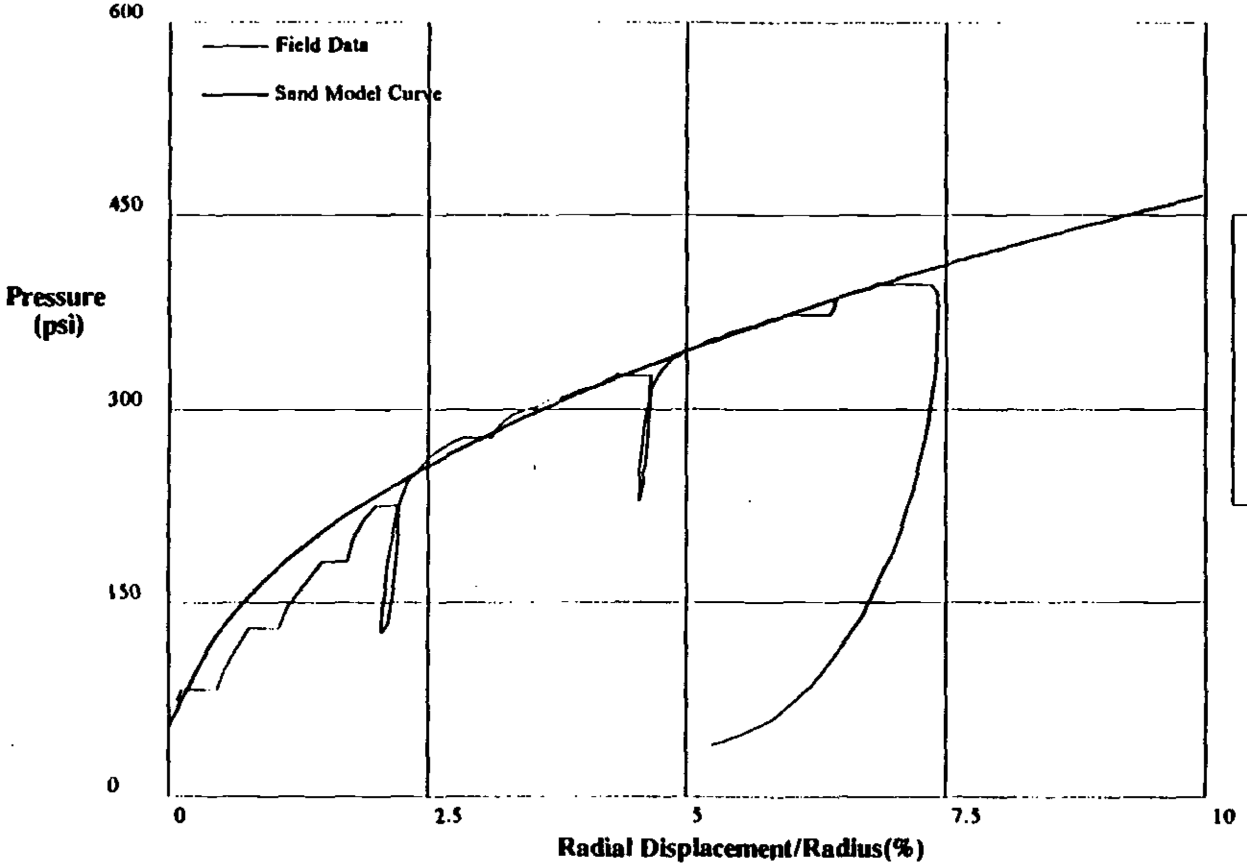
shift 0

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 14 1995

Hole No. DD27 Depth 66.5 ft File D:\DATA\LOSANG\ERTEC6.P



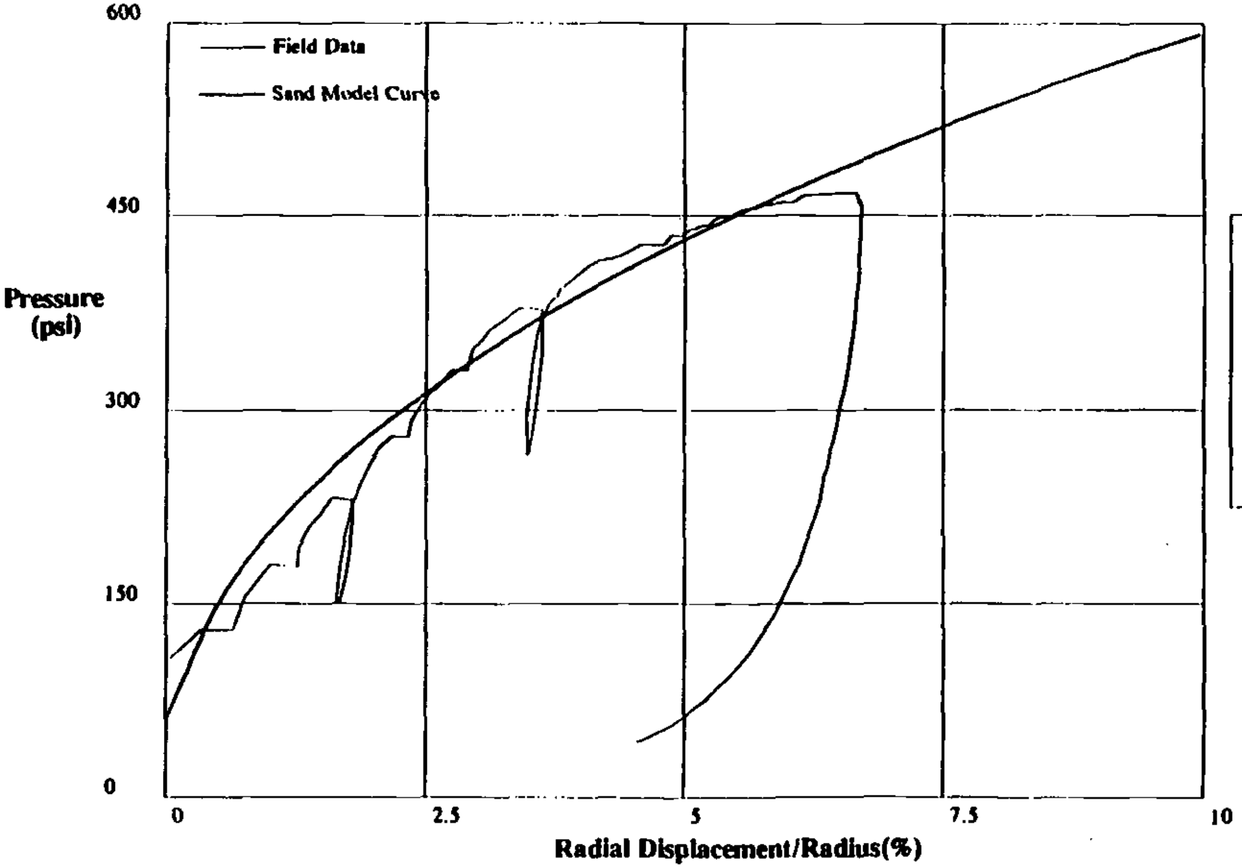
THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	38 deg
Critical Friction Angle	32 deg
Lateral Stress	55 psi
Shear Modulus	8000 psi

shift .5

HUGHES

Pressuremeter Data **Geotransit Consultants**
Eastside Extension, Metro Rail Project **September 14 1995**
Hole No. DD27 **Depth 73 ft** **File D:\DATA\LOSANG\ERTEC7.P**



THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	40 deg
Critical Friction Angle	32 deg
Lateral Stress	60 psi
Shear Modulus	9660 psi

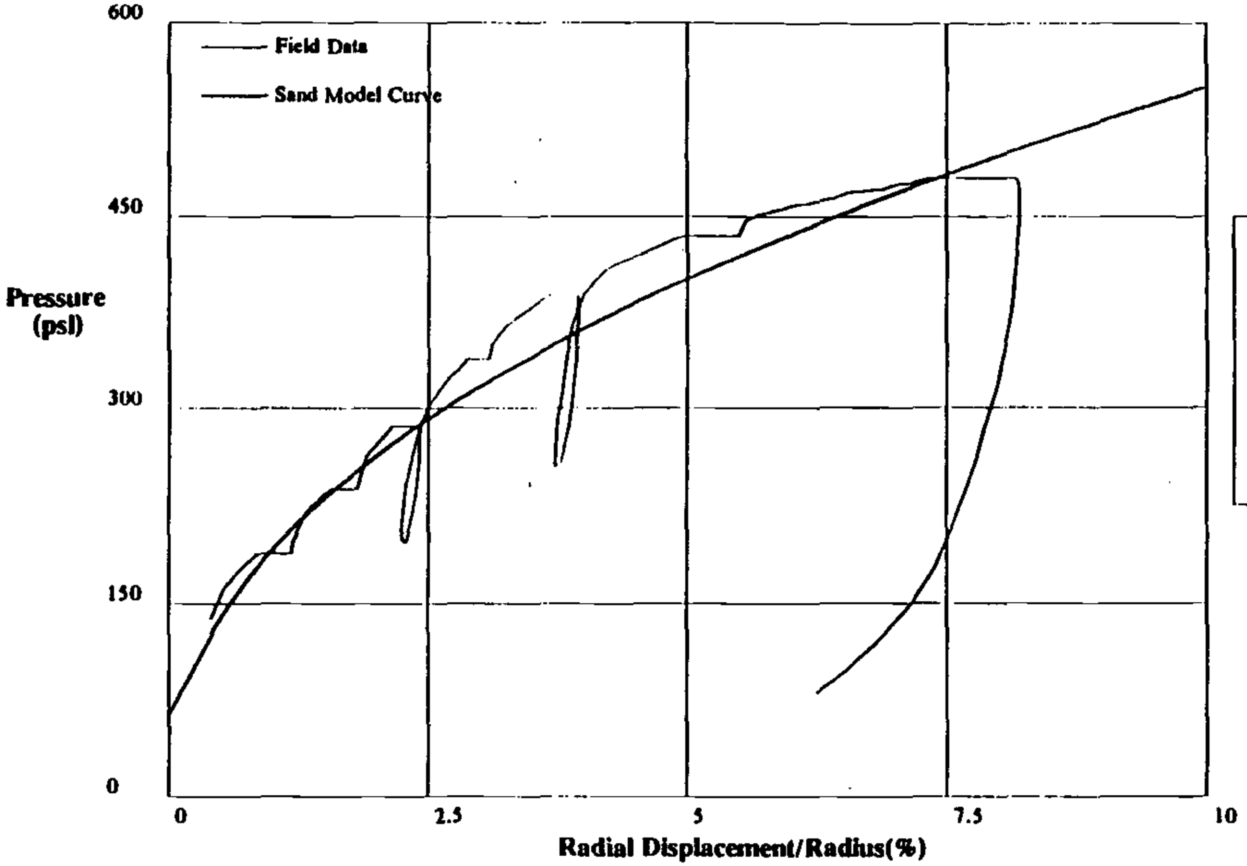
shift 3

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project 14

Hole No. dd27 Depth 86 ft File D:\DATA\LOSANG\ERTEC9.P



THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	40 deg
Critical Friction Angle	32 deg
Lateral Stress	65 psi
Shear Modulus	7500 psi

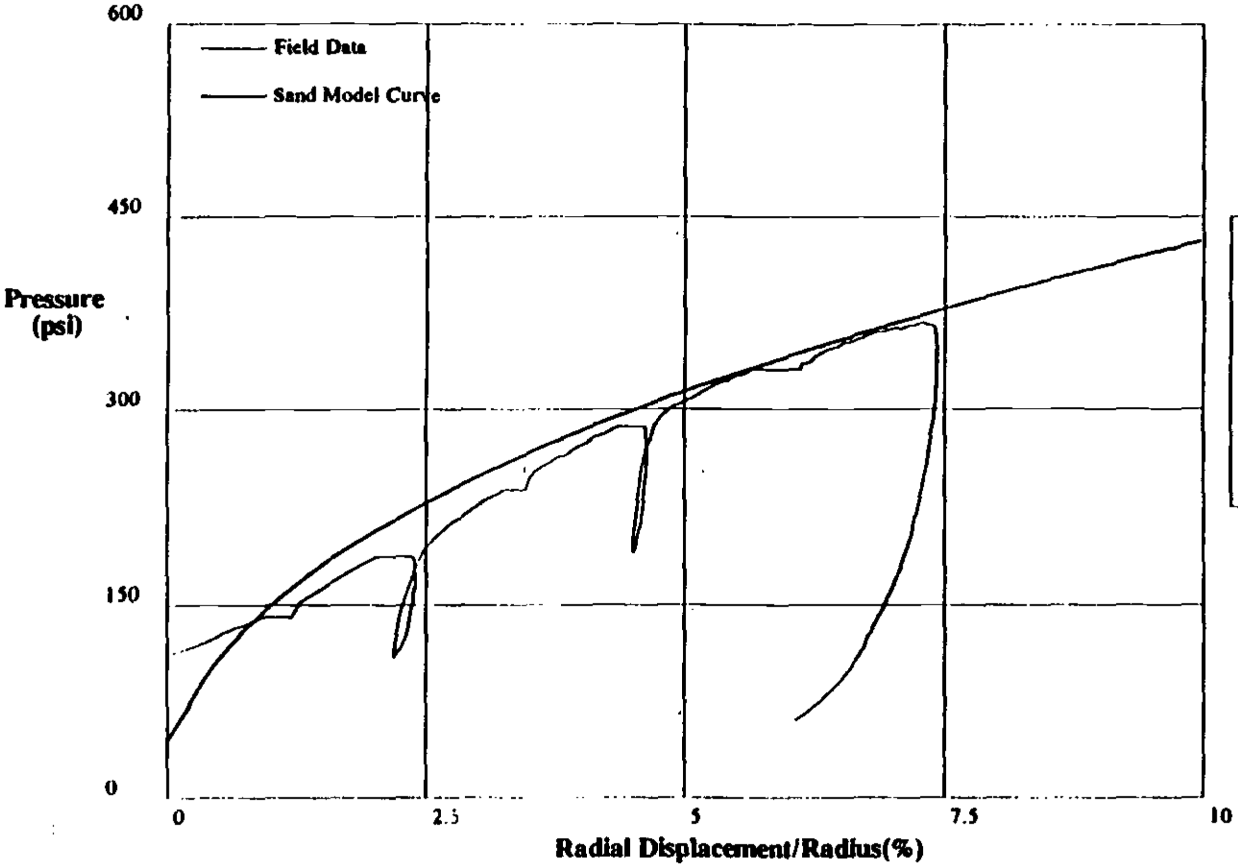
shift 2

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project **Sept ember 15 1995**

Hole No. DD27 **Depth 92.5 ft** **File D:\DATA\LOSANG\ERTEC10.P**



THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	40 deg
Critical Friction Angle	32 deg
Lateral Stress	45 psi
Shear Modulus	6800 psi

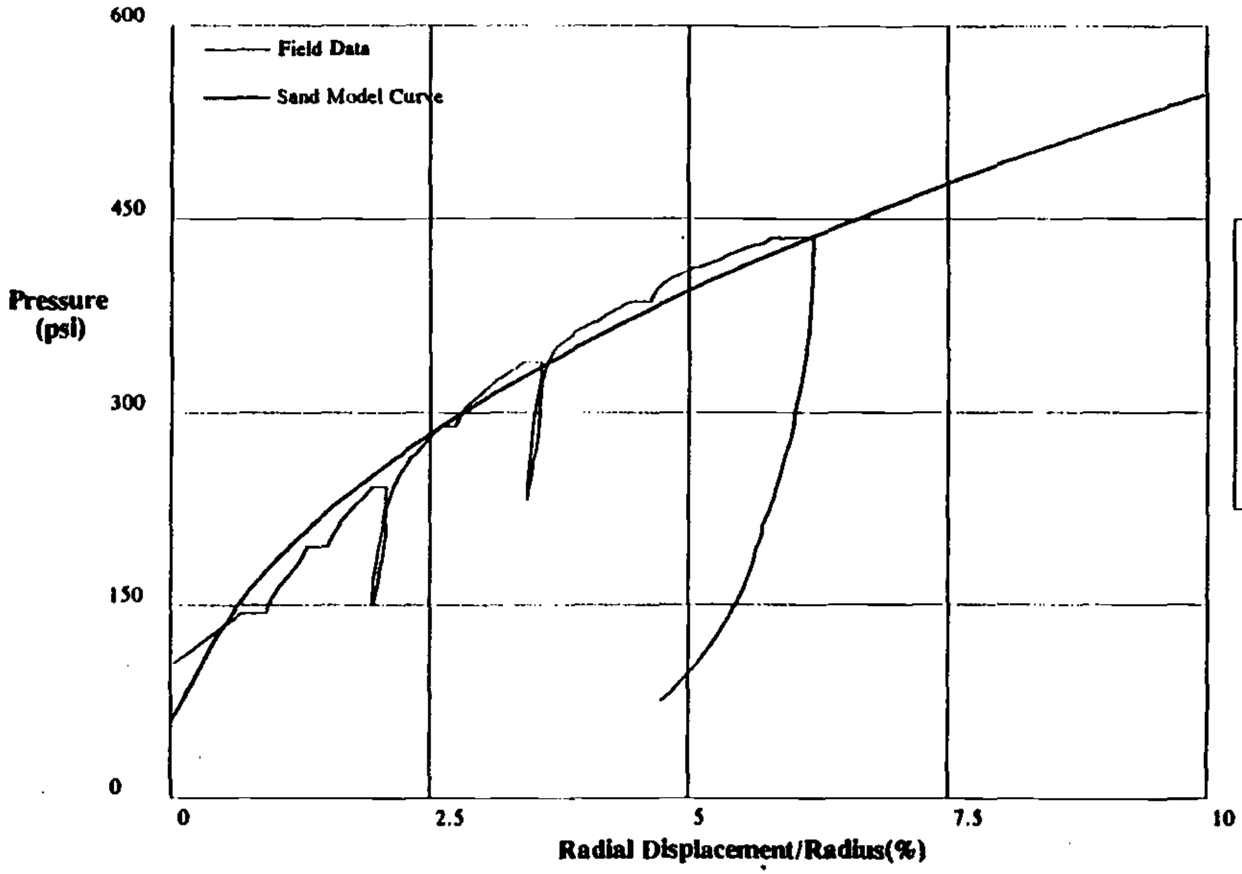
shift 3.5

HUGHES

Pressuremeter Data **Geotransit Consultants**

Eastside Extension, Metro Rail Project September 15 1995

Hole No. DD27 Depth 99.5 ft File D:\DATA\LOSANG\ERTEC11.P



THE HUGHES SAND MODEL

Water Pressure	0 psi
Friction Angle	41 deg
Critical Friction Angle	32 deg
Lateral Stress	60 psi
Shear Modulus	7254 psi

shift 5

HUGHES

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

LABORATORY TESTING

GENERAL

The results of laboratory testing performed in conjunction with this project accompany this appendix. The following laboratory tests were performed on representative samples in accordance with the latest applicable American Society for Testing Materials (ASTM) and California Department of Transportation (Caltrans) standards. Laboratory test results are summarized in Tables 3-6 and 4-3.

CLASSIFICATION

Soils were classified visually in accordance with ASTM Test Method D2488-90. Where laboratory test results were available classification was in accordance with ASTM Test Method D2487-90. The soil classifications are presented in the test data and boring logs.

MOISTURE AND IN PLACE DENSITY

The field moisture content and in place dry density determinations were performed on relatively undisturbed California Drive samples obtained from test borings. The moisture content was obtained in accordance with ASTM Test Method D2216. The in place dry density was computed using the net weight of the entire (ring) sample. The results of these tests are presented in the boring logs.

PARTICLE SIZE ANALYSIS

Grain size distribution of soil samples were obtained in accordance with ASTM Test Method D422. Laboratory test results for the grain size analysis are summarized in Table B-1. Figures B-1 through B-56 present the grain size distribution curves.

FINES CONTENT ANALYSIS

Fines content (percent passing #200 sieve) of soil samples was obtained in accordance with ASTM Test Method D1140. Laboratory test results for the fines content analysis are summarized in Table B-1.

ATTERBERG LIMITS

Tests for Atterberg Limits (Liquid Limit and Plasticity Index) were performed according to ASTM Tests Method D4318. Laboratory test results for the Atterberg Limits are summarized in Table B-2. These results are also presented along with grain size distribution curves.

SPECIFIC GRAVITY

Specific gravity tests were performed on fine-grained (passing #4 sieve) soil samples in accordance with ASTM Test Method D854.

UNCONFINED COMPRESSION TESTS

Unconfined compression tests were performed on selected relatively undisturbed soil samples in accordance with procedures outlined in ASTM Test Method D2168. The soil samples (height/diameter ratio of about 2) were subjected to a constant strain rate of 1 percent per minute. Laboratory test results for the unconfined compression tests are summarized in Table B-3 and graphically presented in Figure B-57.

DIRECT SHEAR TESTS

Direct shear tests were performed on selected relatively undisturbed soil samples which were saturated under surcharge equal to the applied normal force during testing. The apparatus used is in conformance with the requirements outlined in ASTM Test Method D3080. The test specimens, 2.4 inches in diameter and 1 inch in height, were subjected to shear along a plane mid-height after allowing time for pore pressure dissipation prior to application of shearing force.

The samples were tested under normal loads of 3, 6 and 12 ksf with a different specimen being used for each normal load. Drained and undrained direct shear tests were performed on coarse- and fine-grained soil samples, respectively. The samples were sheared at constant rates of strain ranging 0.0025 inches per minute. Shearing of each specimens was continued until the shear stress became essentially constant or until a deformation of approximately 10 percent of the original diameter had been reached. Laboratory test results for the direct shear tests are summarized in Table B-4, and are graphically presented in Figures B-58 through B-66.

CONSOLIDATION AND COLLAPSE TESTS

Consolidation tests were performed on selected relatively undisturbed soil samples in accordance with procedures outlined in ASTM Test Method D2435. The samples were placed in a consolidometer and loads were applied incrementally in geometric progression. The samples (2.4 inches in diameter and 1 inch in height) were permitted to consolidate under each load increment until the slope of the characteristic linear secondary compression portion of the thickness versus log of time plot was apparent.

The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. Hydroconsolidation (collapse) and/or expansion characteristics were also evaluated by monitoring the change in volume with the addition of water while the specimen was confined under a constant normal stress. The consolidation test results are summarized in Table B-5 and are graphically presented in Figures B-67 through B-73.

TRIAXIAL PERMEABILITY TESTS

Triaxial permeability tests were performed on selected relatively undisturbed soil samples in accordance with procedures outlined in ASTM D5084. The samples were saturated by the application of back pressure and tested under an effective confining pressure equivalent to the in situ vertical stress. Water flow between the top and bottom of the sample was measured by applying a pressure head difference of 2 psi. Laboratory test results for the permeability tests are summarized in Tables 3-5 and B-6 and are tabular presented in Tables B-8 and B-9.

pH, CHLORIDE CONTENT, SULFIDES CONTENT, SULFATE CONTENT AND ELECTRICAL *RESISTIVITY TESTS*

The concentrations of water soluble sulfate ions of selected soil samples were determined in accordance with the Caltrans Method No. 417-B. Soil pH values were determined in accordance with USEPA Method 9045. Sulfides content was determined in accordance with USEPA Method 9030. Chloride content was determined in accordance with Caltrans Method No 422. Tests to determine the electrical resistivity were conducted in general accordance with Caltrans Method No. 532. Electrical resistivity tests were conducted at in situ moisture content on all samples. The results of pH, chloride content, soluble sulfate content and electrical resistivity evaluations are presented in Table B-7.

TABLE B-1. RESULTS OF GRAIN SIZE AND FINES CONTENT TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Percent Passing #200 Sieve ASTM D1140 (%)
SD-15	S-1	5	SM	Qa	0:53:47	
	D-2	10	SM			45
	S-3	15	SM		0:65:35	
	S-5	25	ML		1:46:53	
	S-7	35	SM		18:69:13	
	S-9	45	(SP-SM)			12
	D-10	50	SP-SM			7
	S-11	55	(SM)		25:62:13	
	S-13	65	(SM)			13
	S-15	75	(SW-SM)		20:68:12	
	D-18	90	SM			37
	S-19	95	SM		1:76:23	
	D-22	110	CL			59
SD-16	D-1	10	CL	Qa		52
	S-2	15	SC		2:60:38	
	D-3	19	(SP-SM)			10
	S-4	25	(SM)		16:68:16	
	S-6	35	SC		3:51:46	
	D-7	39.0	SM			19
	S-8	45	SW-SM		18:71:11	
	S-10	55	SW-SM		31:57:12	
	D-11	60	SW-SM			10
	S-12	65	(SM)		1:81:18	
	D-13	68.5	CH			82
	S-15	75	SM		1:76:23	
	D-16	80	SP-SM			8
	S-17	85	CL			85
	S-19	94	CL			60
	S-21	105	SC		0:57:43	

TABLE B-1. RESULTS OF GRAIN SIZE AND FINES CONTENT TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Percent Passing #200 Sieve ASTM D1140 (%)
SD-17	S-2	10	GM	Qa	52:35:13	
	S-4	20	CL		9:39:52	
	D-5	25	SP-SM			9
	S-6	30	SM		11:71:18	
	S-8	40	ML		0:35:65	
	D-9	45	SM/ML			50
	D-11	55	SP-SM			7
	S-12	60	ML		5:38:57	
	S-14	70	SW-SM		17:71:12	
	S-16	80	ML		0:45:55	
	D-19	95	SM			34
SD-18	S-4	20	SM	Qa	1:57:42	
	S-6	30	SM			15
	S-8	40	SM		27:52:21	
	D-9	45	GP-GM			6
	S-10	50	(ML)		4:41:55	
	D-11	55	SM			21
	S-12	60	SM		14:63:23	
	S-14	65	SM			26
	S-16	75	ML		0:29:71	
	D-17	80	(CL)			51
	S-20	90	SM		8:77:15	
DD-27	D-1	10	SM	Qa		20
	S-4	21	SM		14:73:13	
	S-9	35	CL		1:19:80	
	D-11	42	CL			66
	S-12	47	CL		1:47:52	
	S-14	53.5	SM		9:50:41	
DD-27S	S-17	60	ML	Qa	0:44:56	

TABLE B-1. RESULTS OF GRAIN SIZE AND FINES CONTENT TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Percent Passing #200 Sieve ASTM D1140 (%)
DD-27S	S-19	66.5	SM	Qa	1:70:29	
	S-21	73	ML		0:35:65	
	S-23	80	SM			14
	D-24	81.3	SM			27
	S-25	86	SM		16:62:22	
	S-27	92.5	SM		0:79:21	
	S-30	100.5	ML		0:49:51	
	S-33	112.5	(SC)		5:61:34	
	D-36	124	CL		0:35:65	
DD-28	D-3	20	CL	Qa		60
	D-5	30	SM			40
	D-7	40	CL			56
	S-8	44	CL		0:35:65	
	D-9	50	CL			96
	S-10	53.5	SM		5:80:15	
	D-11	60	(SP-SM)			9
	S-12	63.5	CL		1:36:63	
	S-14	70	(CL)		0:49:51	
	S-16	76	ML			67
	S-18	82	SM		0:59:41	
	D-19	85	SP			3
	S-20	88	ML		0:49:51	
	D-21	90.5	SM			29
	S-22	94	ML		0:27:73	
	D-23	97	ML			63
	S-24	100	ML		0:32:68	
	D-25	105	ML			54
	S-26	107	ML		0:39:61	
	S-28	115	SM		0:76:24	

TABLE B-1. RESULTS OF GRAIN SIZE AND FINES CONTENT TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Percent Passing #200 Sieve ASTM D1140 (%)
DD-62	S-9	41	SM	Qa		44-47
	S-11	47	SM		6:68:26	
	S-13	53	(SP-SM)		27:62:11	
	S-15	59	(SP-SM)		45:47:8	
	S-17	65	(SM)		20:61:19	
	S-19	71	ML		0:20:80	
	S-21	77	ML		0:38:62	
	D-22	80	SM			15
	S-23	83	SM			33

NOTES:

1. GR:SA:FI=Gravel: Sand: Fines (percent passing #200 sieve)
2. USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).
3. Some of the material classifications (identified in parentheses) shown in column 4 of this table are not consistent with the general classification of that interval in the boring logs. This generally occurs in gravelly and cobbly layers where due to presence of gravels and cobbles larger than sampler size (2.4-inch inner diameter), laboratory gradations and classifications only reflect the finer matrix materials in alluvium.

TABLE B-2. RESULTS OF ATTERBERG LIMITS TEST

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Atterberg Limits ASTM D4318 LL,PL,PI
SD-15	D-4	20	CL	Qa	48,22,26
	S-17	85	CL		32,19,13
	S-21	105	CH		63,23,40
SD-16	D-13	68.5	CH	Qa	53,21,32
	D-18	89.0	CL		40,15,25
	D-23	113.0	CL		28,14,14
	T-27	124	CL	Tf/Tp	34,15,19
SD-17	D-17	85	CL	Qa	44,13,31
	D-21	105	CL	Tf/Tp	43,18,25
SD-18	D-3	15	CL	Qa	47,24,23
	D-17	80	CL		31,13,18
	D-21	94	CL		33,18,15
DD-27	D-11	42	CL	Qa	40,19,21
DD-28	D-15	73	ML	Qa	38,30,8
DD-62	D-2	10	SC	Qa	30,20,10
	S-9	41	SC-SM		21,14,7
	D-10	44	CL-ML		25,20,5

NOTES:

1. LL,PL,PI = Liquid Limit, Plastic Limit, Plasticity Index
2. USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).

TABLE B-3 . RESULTS OF COMPRESSION TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Unconfined Compressive Strength ASTM D2166 (ksf)
SD-16	T-27	124	CL	Tf/Tp	12.224

NOTE:

USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).

TABLE B-4. RESULTS OF DIRECT SHEAR TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Direct Shear ASTM D3080 (Peak Strength)	
					Effective Cohesion (PSF)	Effective Friction Angle (DEGREES)
SD-15	D-4	20	CL	Qa	1400	27
	D-10	50	SP-SM		400	37
SD-16	D-7	39.0	SM	Qa	0	45
	D-26	123.0	CL	Tf/Tp	3000	25
SD-17	D-9	45	SM/ML	Qa	600	31
	D-17	85	CL		2000	25
SD-18	D-17	80	CL	Qa	920	29
DD-27S	D-24	81	SM	Qa	1200	32
DD-28	D-19	85	SM	Qa	300	41

NOTE:

USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).

TABLE B-5. RESULTS OF CONSOLIDATION TESTS AND COLLPASE POTENTIAL

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Consolidation Characteristics, ASTM D2435 & Collapse Potential, ASTM D5333				
					Cc'	Cs'	Cx'	Vertical Stress (KSF)	Swell(+)/ Collapse(-) (%)
SD-15	D-18	90	SM	Qa	0.063	0.006	0.0011	1	-0.1
SD-16	D-13	88.5	CH	Qa	0.082	0.02	0.0006	8	0.3
	D-18	89	CL		0.075	0.020	0.0006	10	0.26
SD-17	D-17	85	CL	Qa	0.08	0.018	0.001	1	0.31
	D-21	105	CL	Tf/Tp	0.05	0.016	0.0004	10	0.88
SD-18	D-21	94	CL	Qa	0.076	0.012	0.0007	1	0.52
DD-62	D-4	20.0	SP-SM	Qa	-	-	-	2.5	-0.17

NOTES:

1. USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).
2. Compression Index (Cc') and Swelling Index (Cs') are based on vertical strain-log stress plot Rate of Secondary Compression (Cx') is based on vertical strain-log time plot.
3. Swell/Collapse indicates measured strains in sample when saturated from in situ moisture conditions at approximately the in situ overburden pressure.

TABLE B-6. RESULTS OF TRIAXIAL PERMEABILITY TESTS

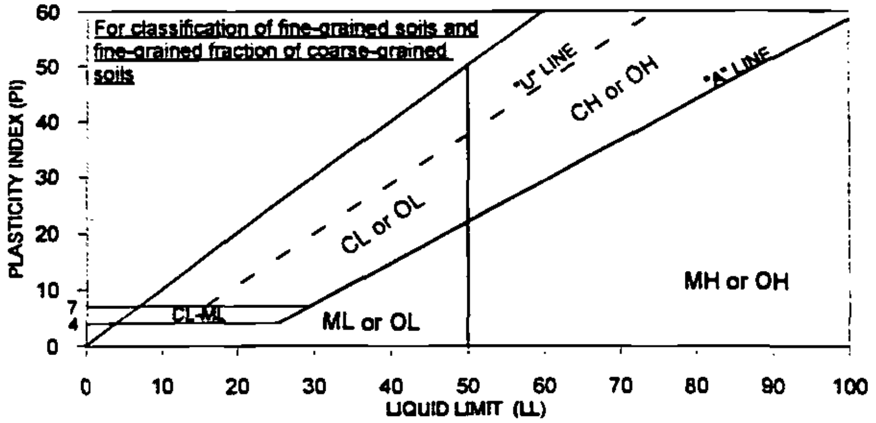
Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Triaxial Permeability ASTM D5084	
					Effective Confining Stress (psi)	Permeability (cm/sec)
SD-15	D-14	70	SP	Qa	30	4.0E-04
DD-28	D-15	73	ML	Qa	60	5.0E-06

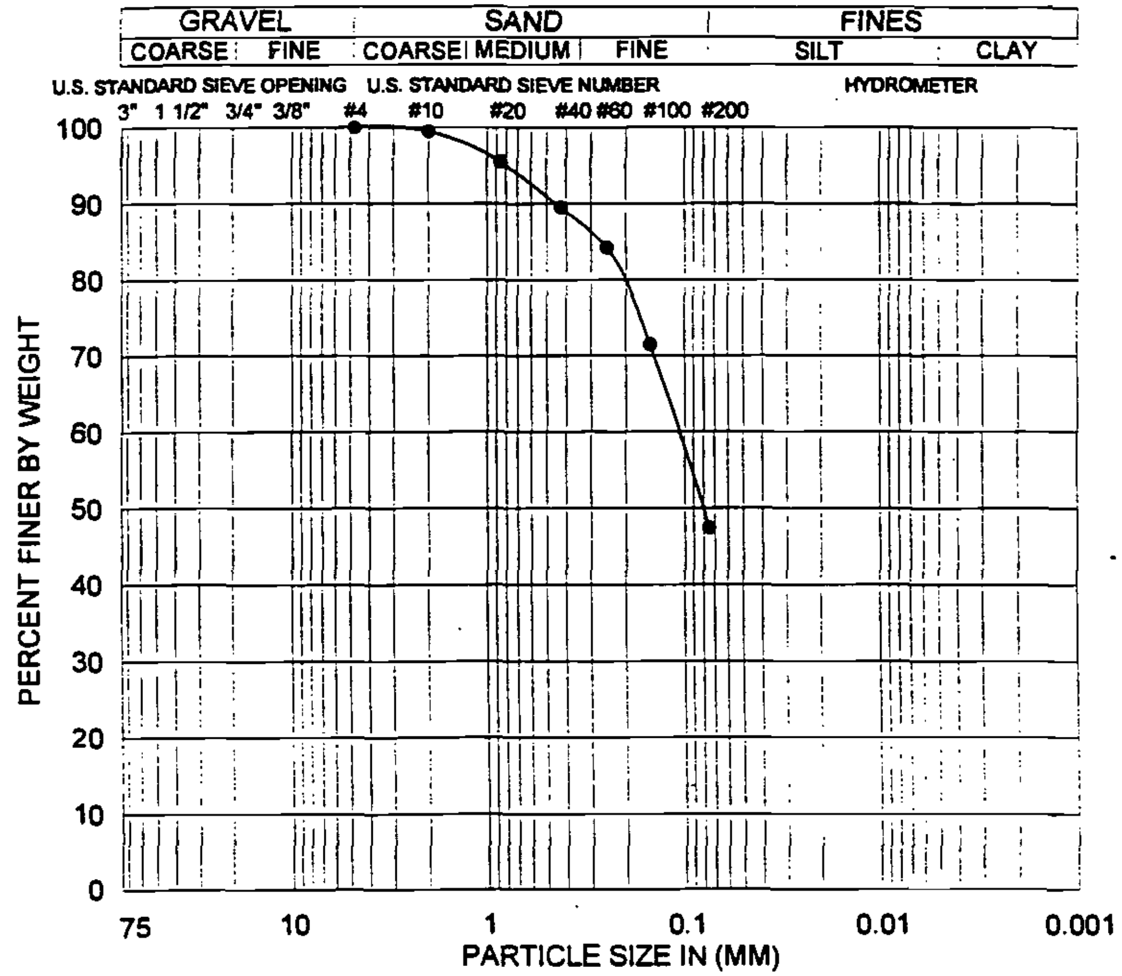
NOTE:

USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487).

TABLE B-7. RESULTS OF CORROSIVITY TESTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 / ASTM D2488	Geologic Unit	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (ppm)	Chloride Content DOT CA Test 422 (ppm)	In-situ/<minimum> Electrical Resistivity @ Insitu/<Adjusted> Moisture Content DOT CA Test 532 (ohm-cm)
SD-15	S-13	65	SM	Qa	8.50	95	110	862
SD-16	D-13	69	CH	Qa	7.44	121	107	1531
SD-17	S-8	40	ML	Qa	7.85	76	57	2329
SD-18	S-10	50	ML	Qa	6.76	138	907	<2941>
DD-27S	D-22	74.5	SM	Qa	6.97	46	90	<2128>
DD-28	S-20	88	ML	Qa	7.42	74	60	<2778>





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	5.0	Bag	SM	0:53:47	N/A

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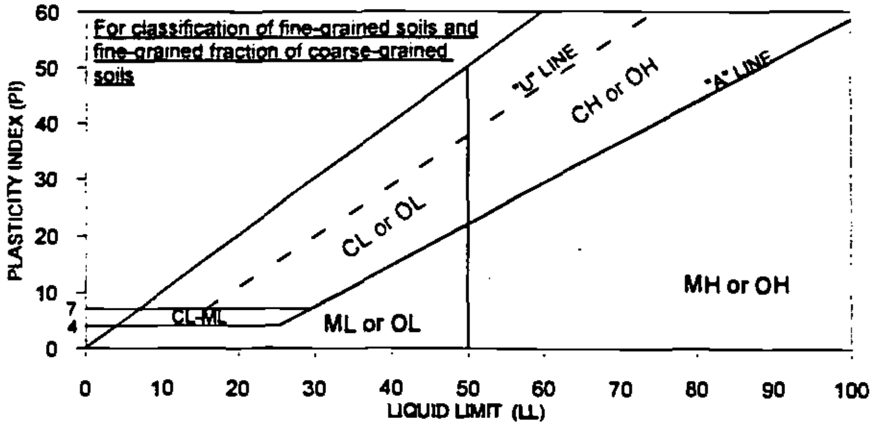
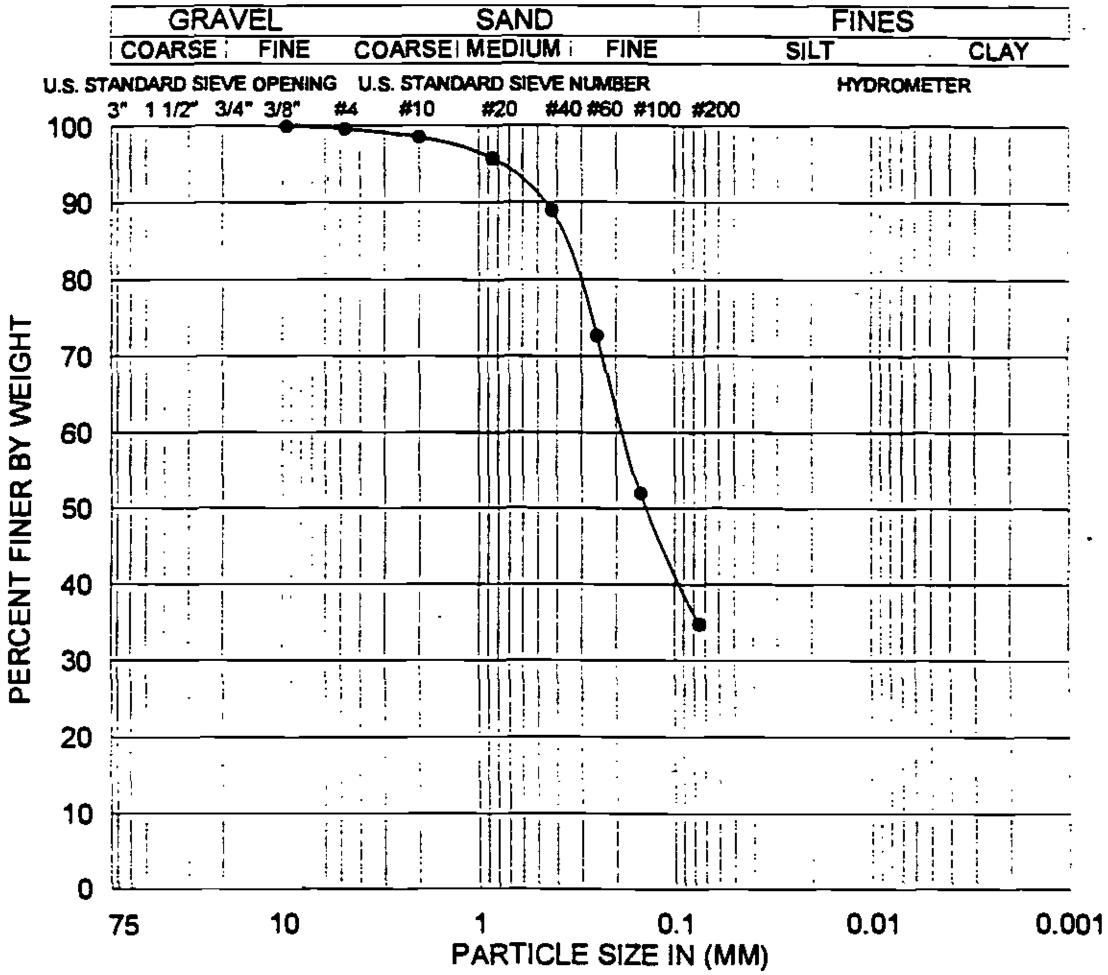
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-1





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	15.0	Bag	SM	0:65:35	N/A

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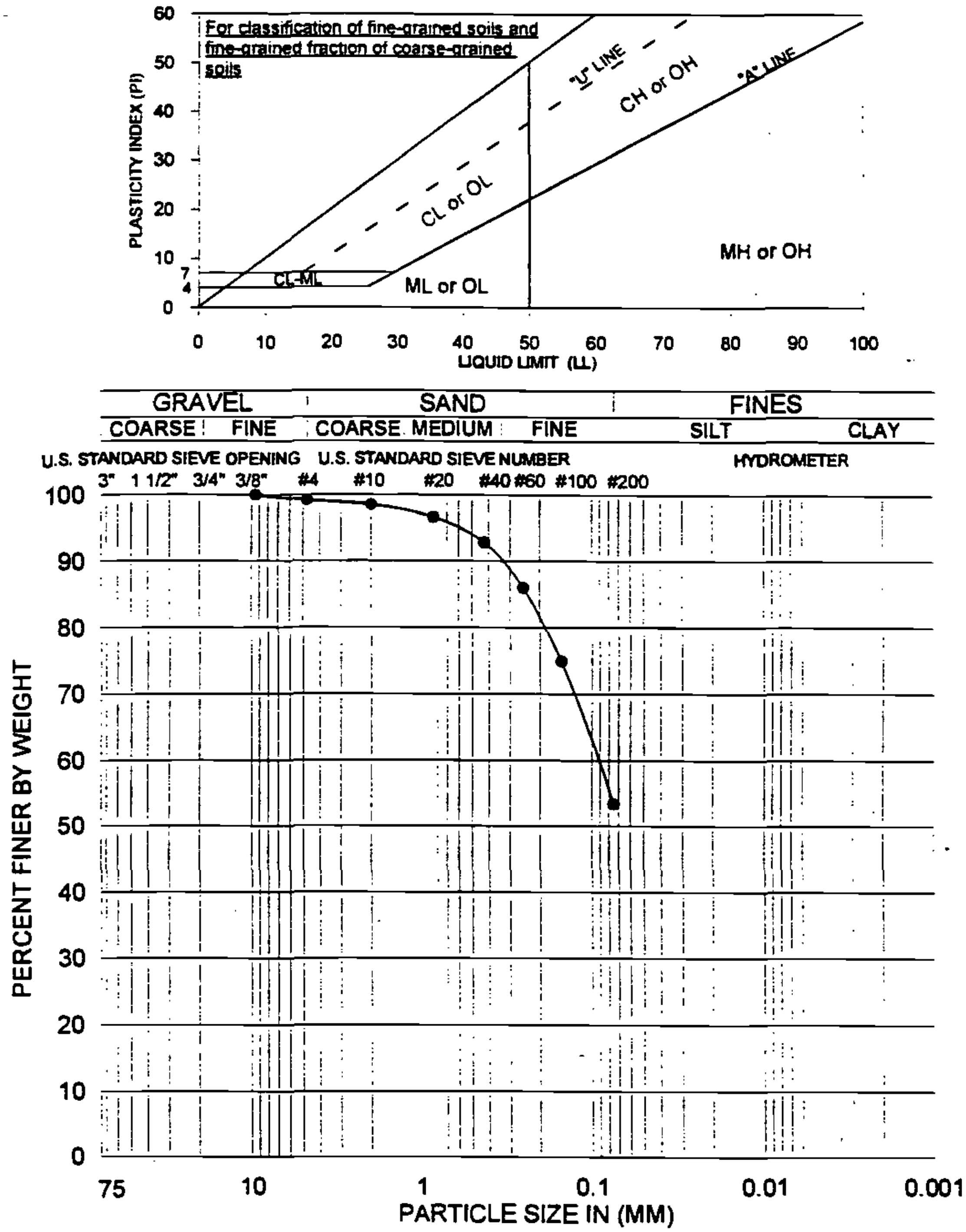
12-95

Figure B-2



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	25.0	Bag	CL	1:46:53	N/A

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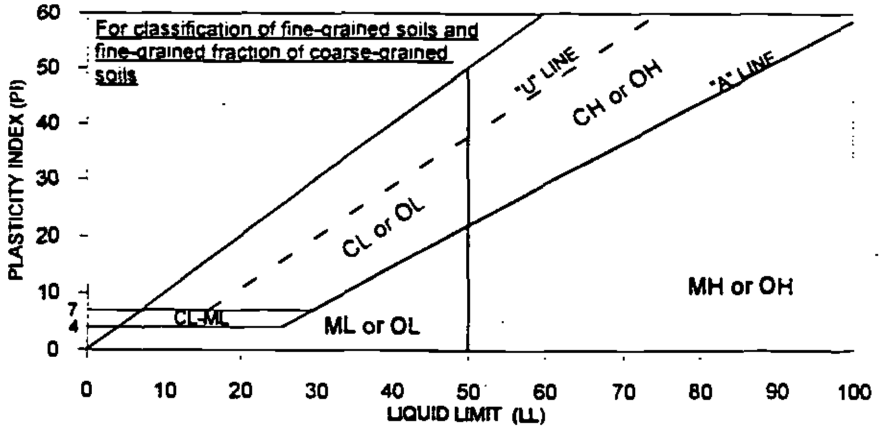
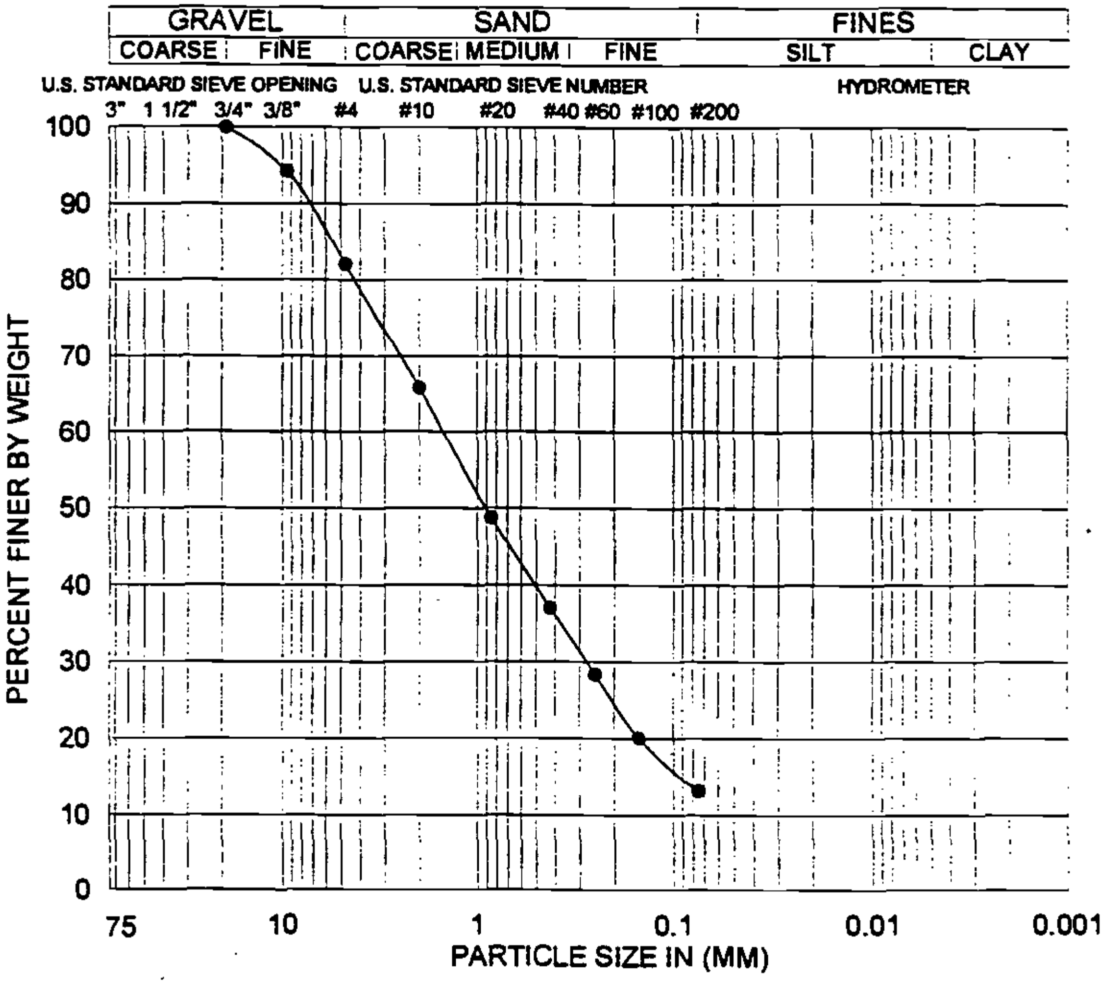
12-95

Figure B-3





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	35.0	Bag	SM	18:69:13	N/A

Not enough sample to meet ASTM specification

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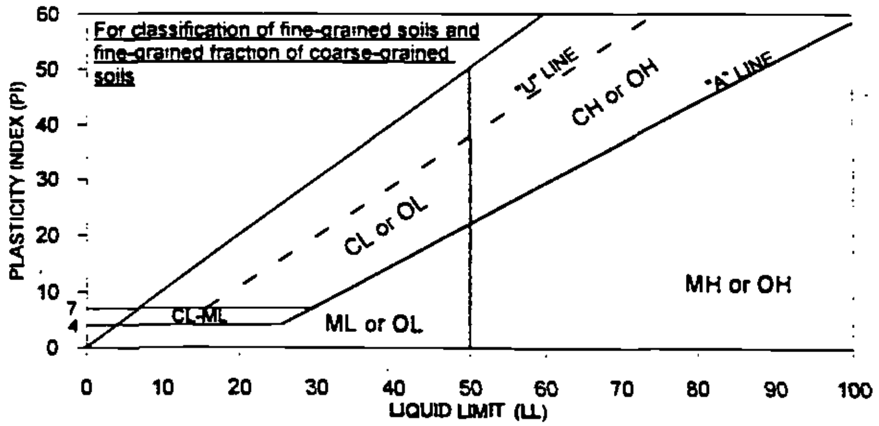
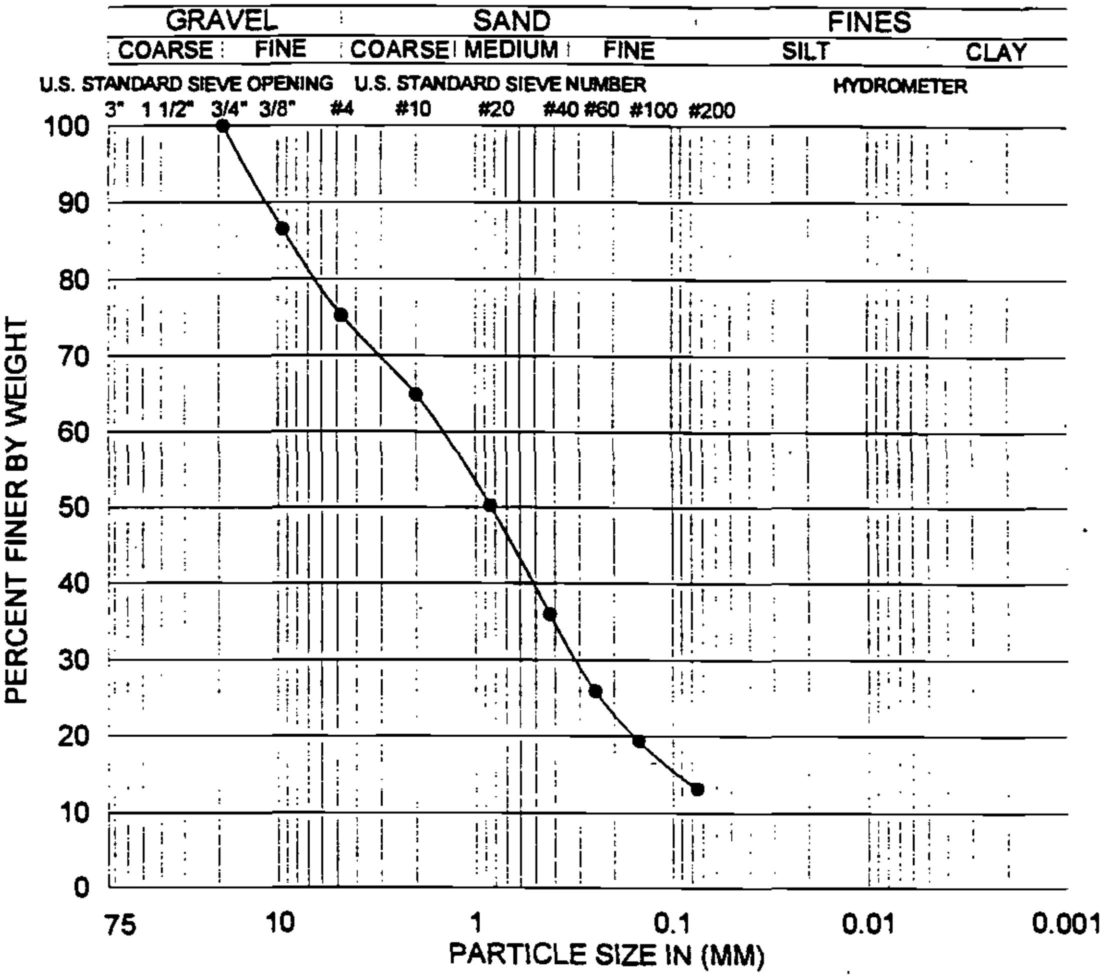
(ASTM D4318, D422)

12-95

Figure B-4





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	55.0	Bag	SM	25:62:13	N/A

Not enough sample to meet ASTM specification

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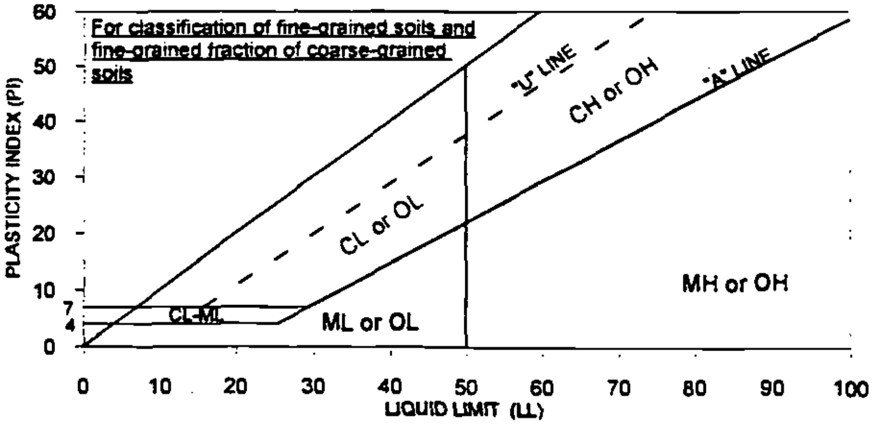
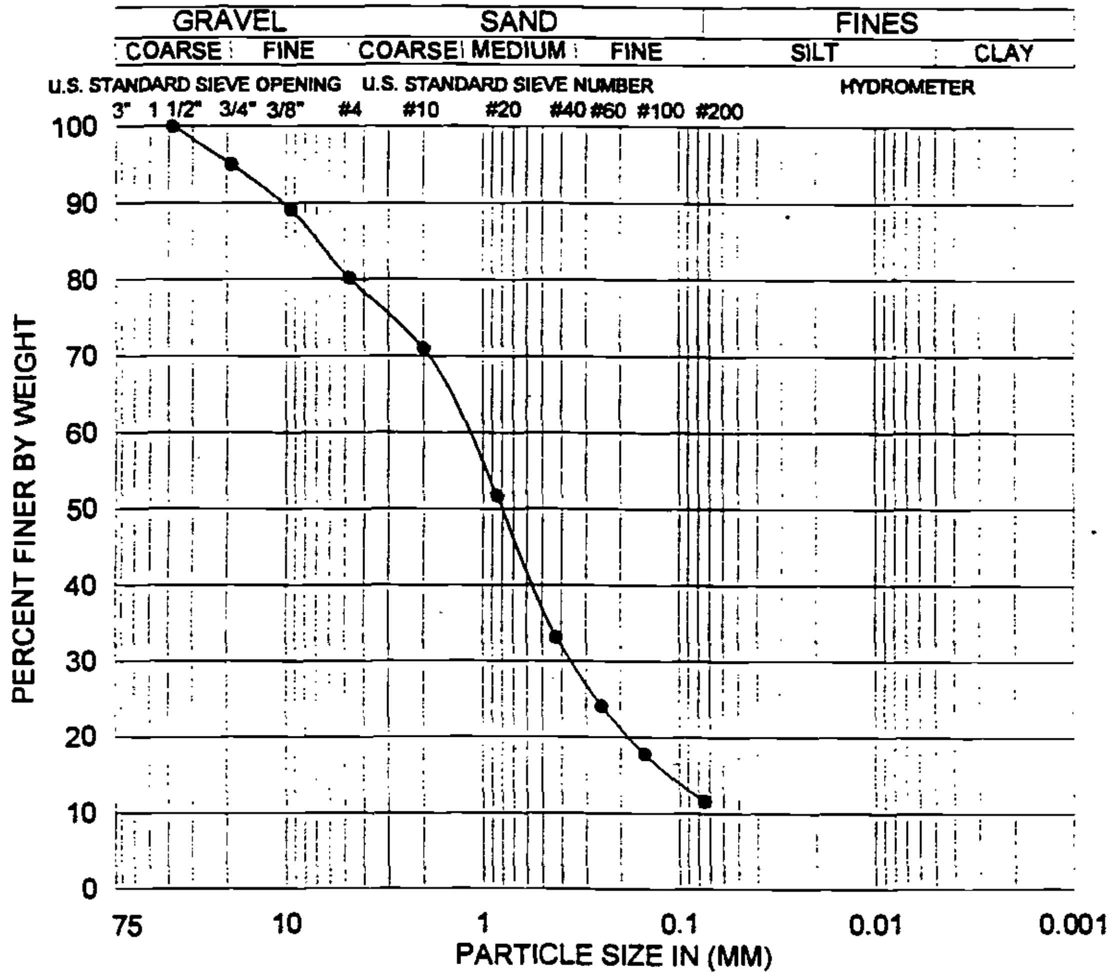
Figure B-5





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	75.0	Bag	SW-SM	20:68:12	N/A

Not enough sample to meet ASTM specification

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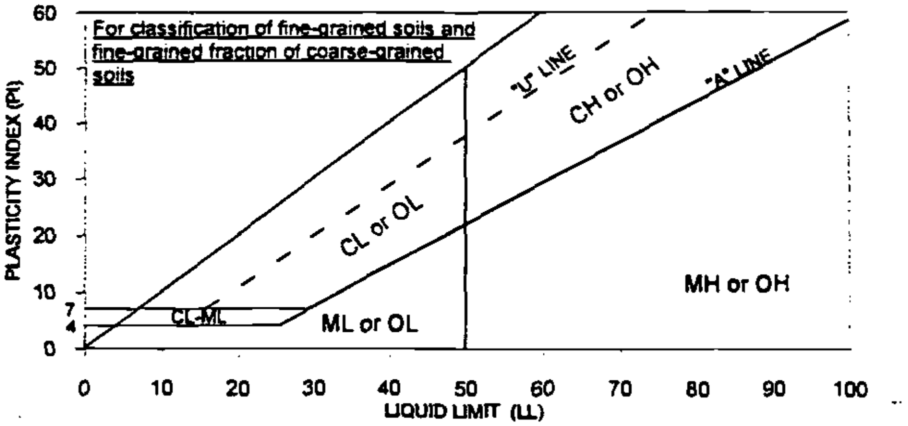
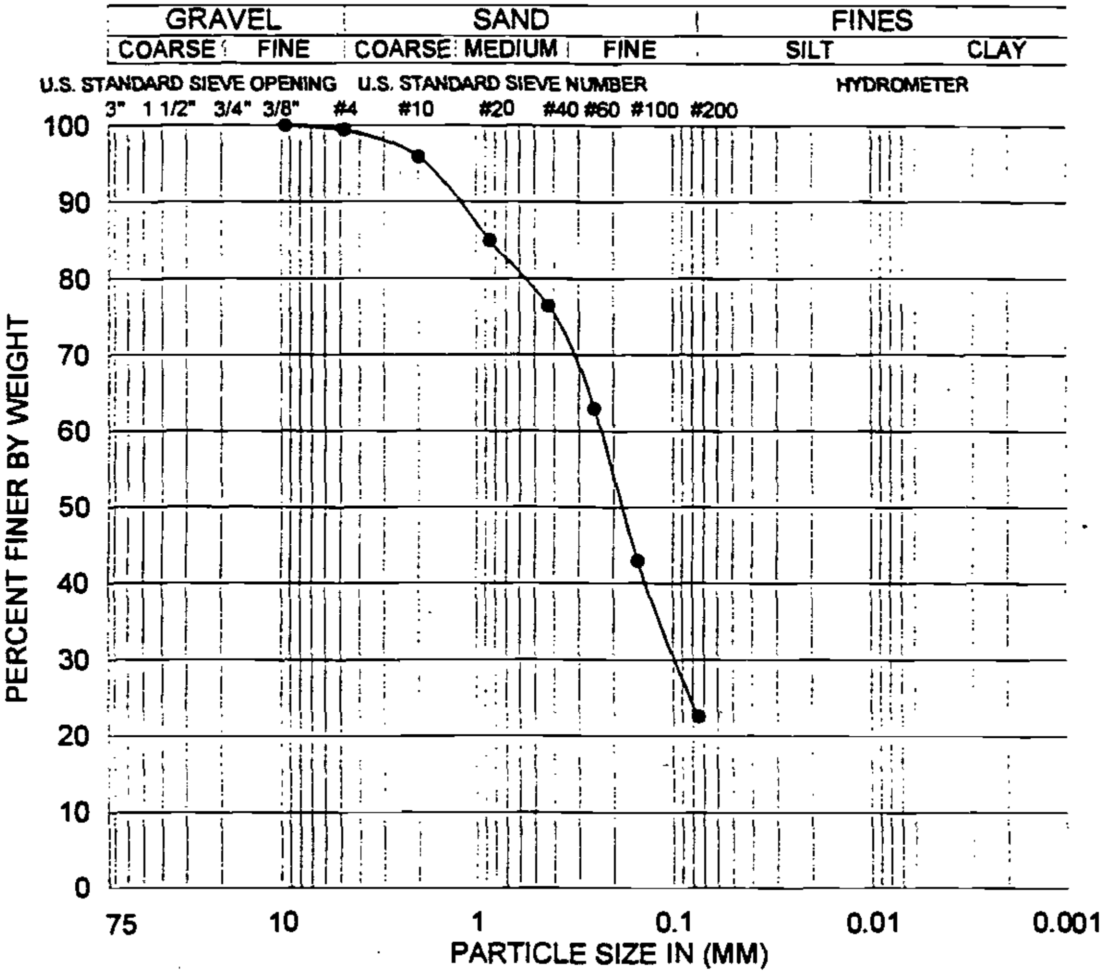
Figure B-6





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-15	N/A	95.0	Bag	SM	1:76:23	N/A

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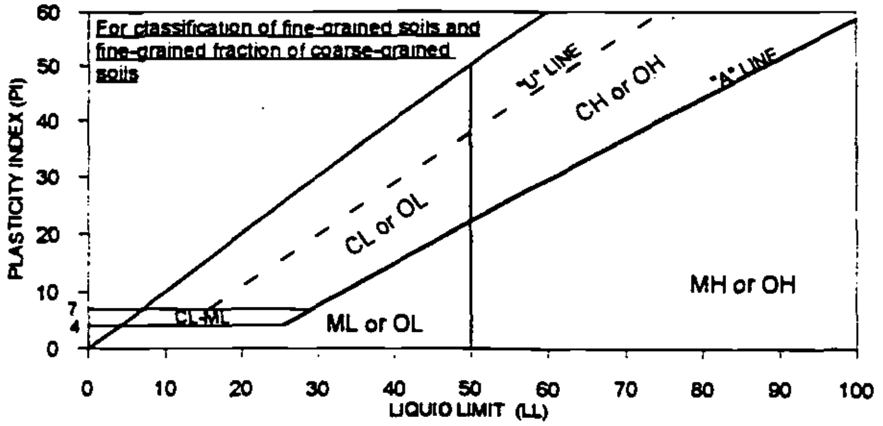
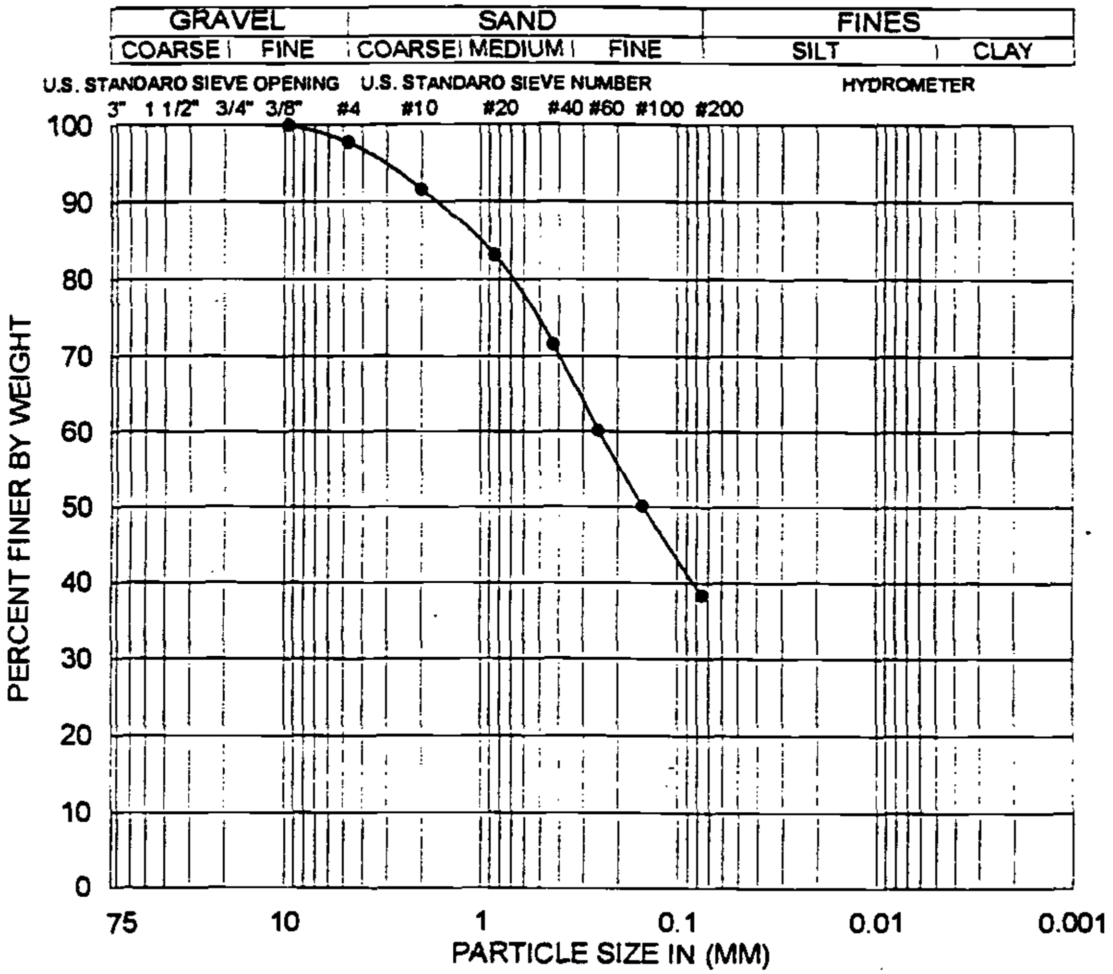
Figure B-7





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-2	15.0	Bag	SC	02:60:38	N/A

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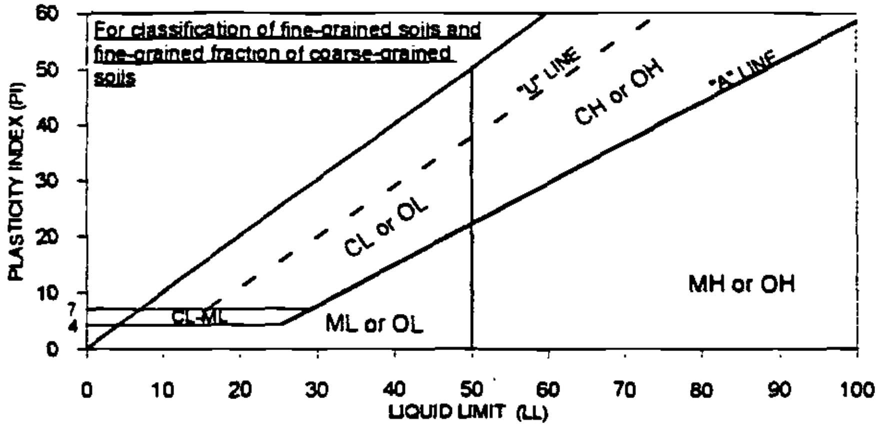
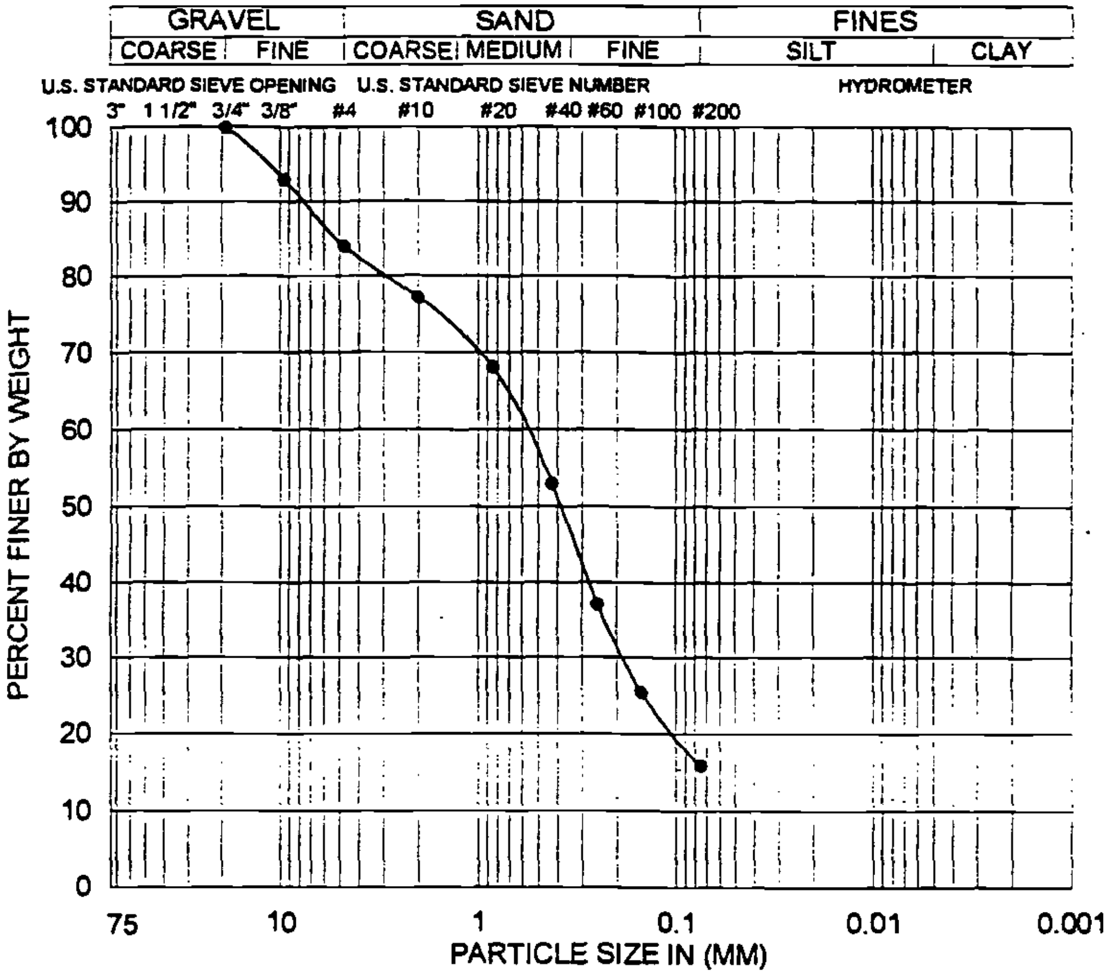
Figure B-8





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-4	25.0	Bag	SM	16:68:16	N/A

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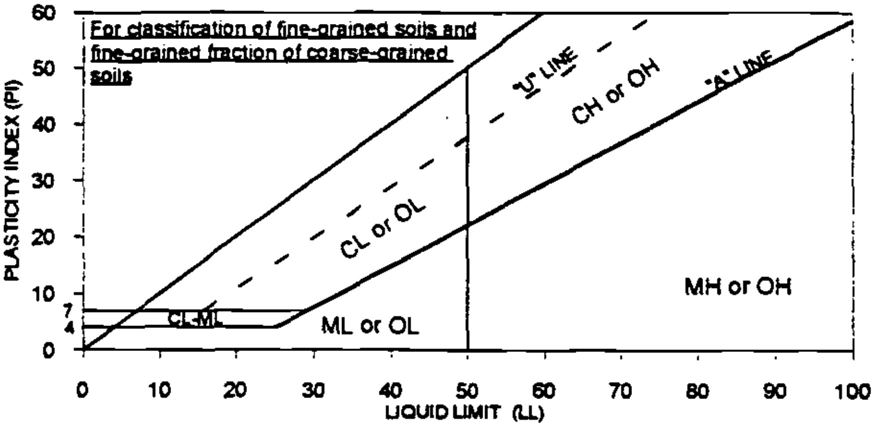
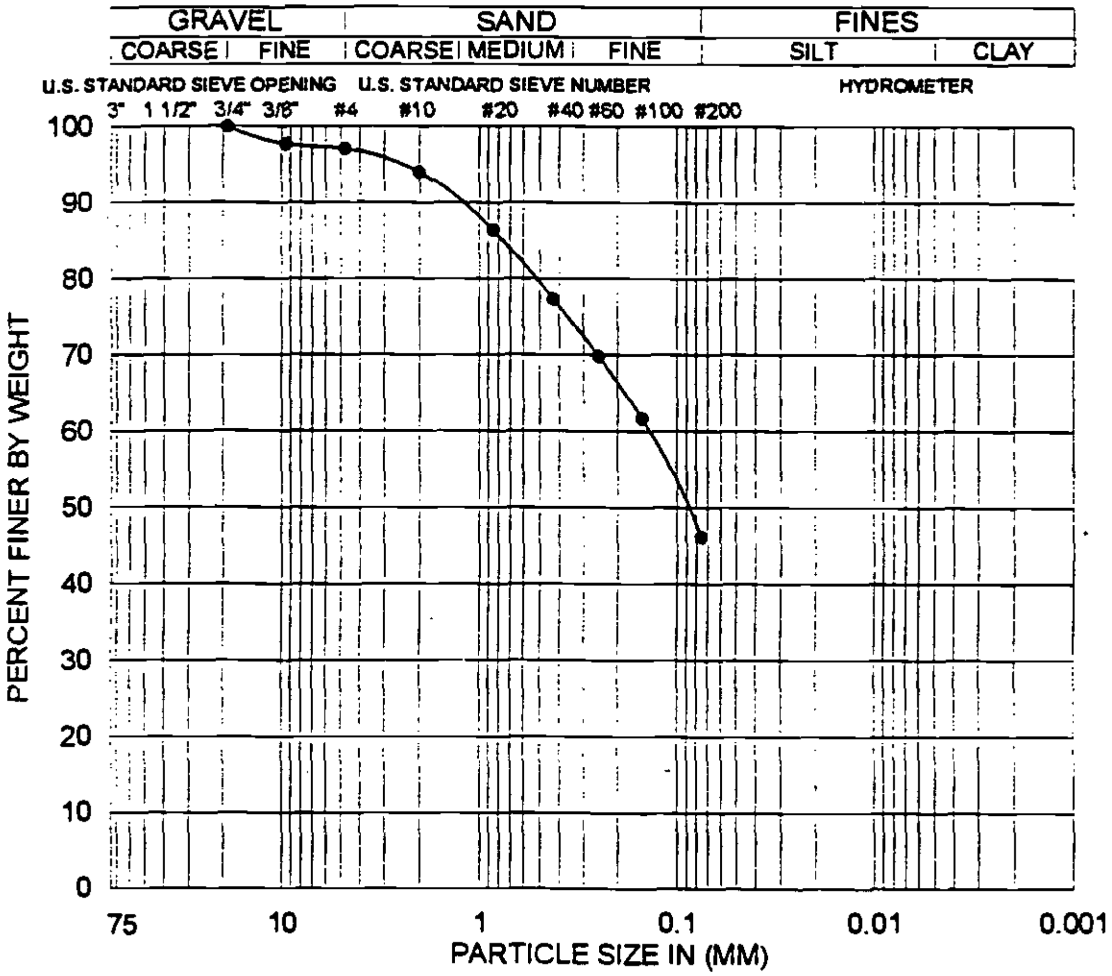
Figure B-9





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-6	35.0	Bag	SM	03:51:46	N/A

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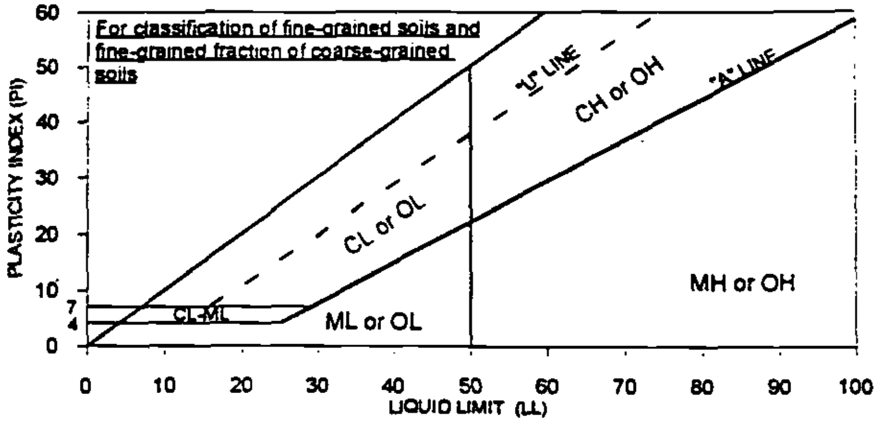
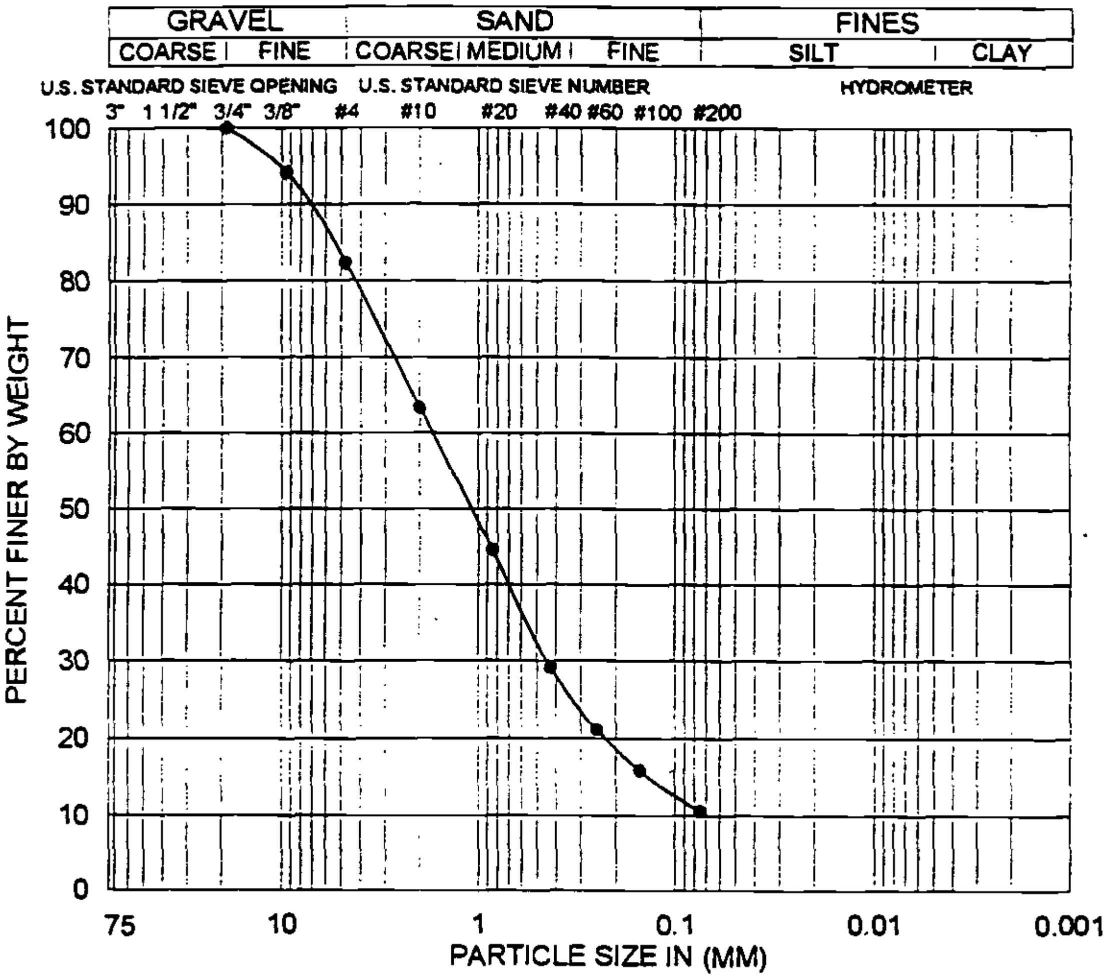
(ASTM D4318, D422)

12-95

Figure B-10





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-8	45.0	Bag	SW-SM	18:71:11	N/A

Not enough sample to meet ASTM specification

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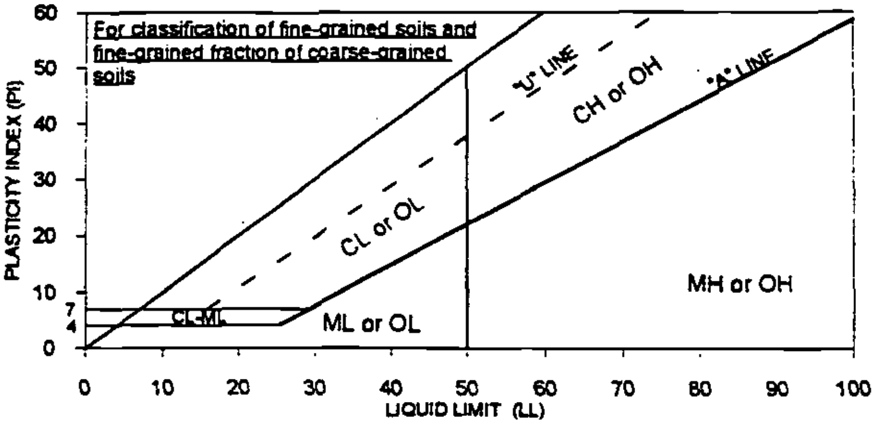
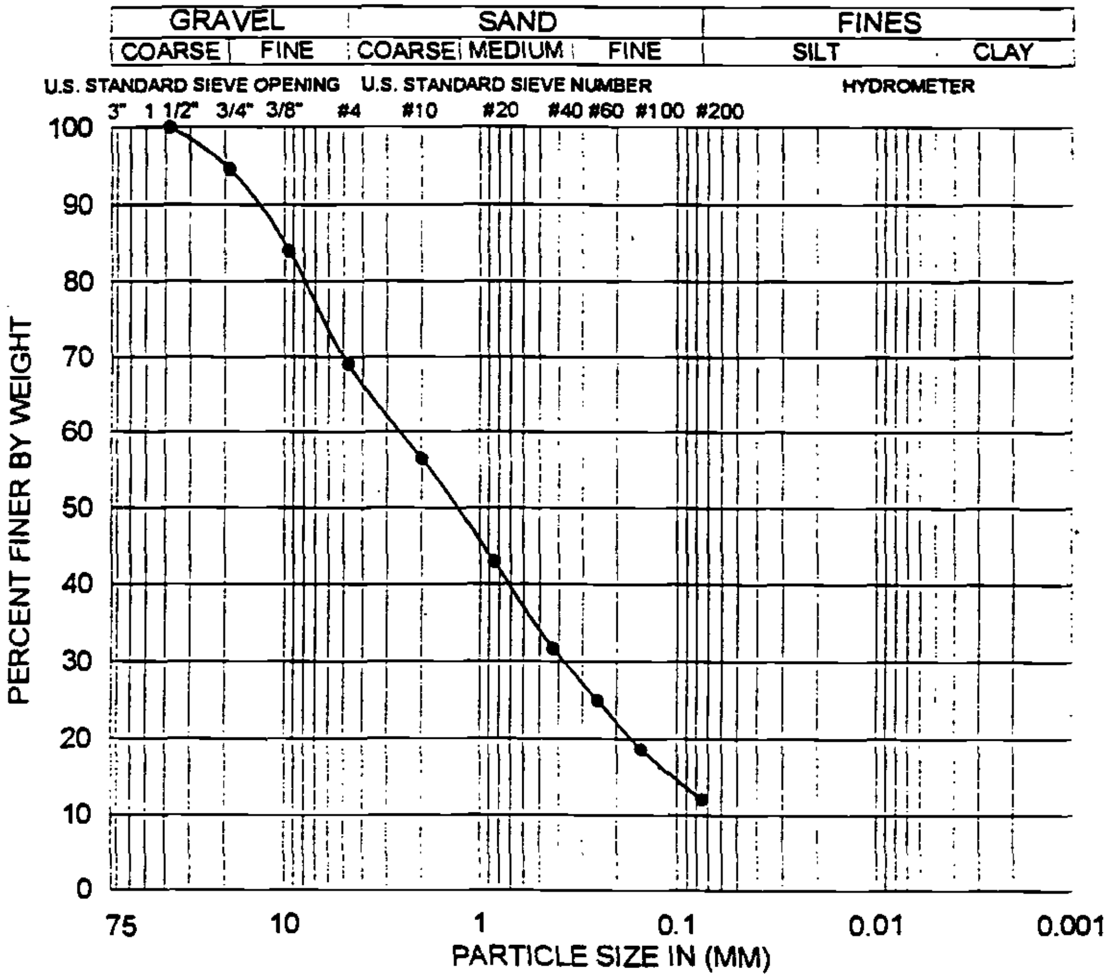
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-11





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-10	55.0	Bag	SW-SM	31:57:12	N/A

Not enough sample to meet ASTM specification

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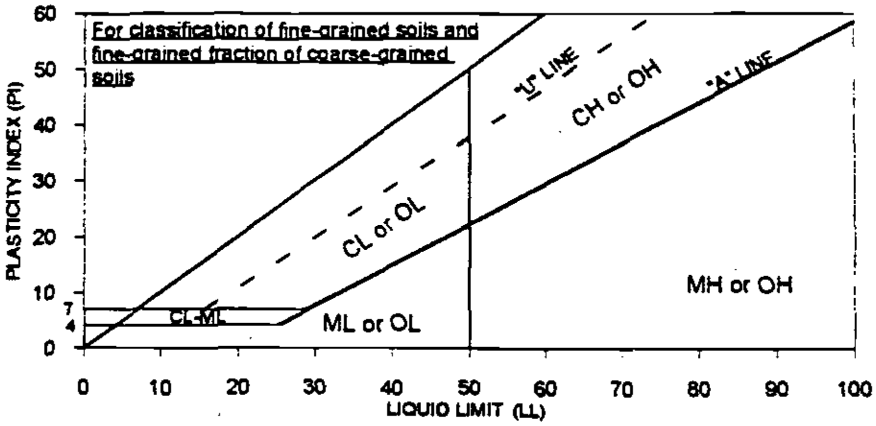
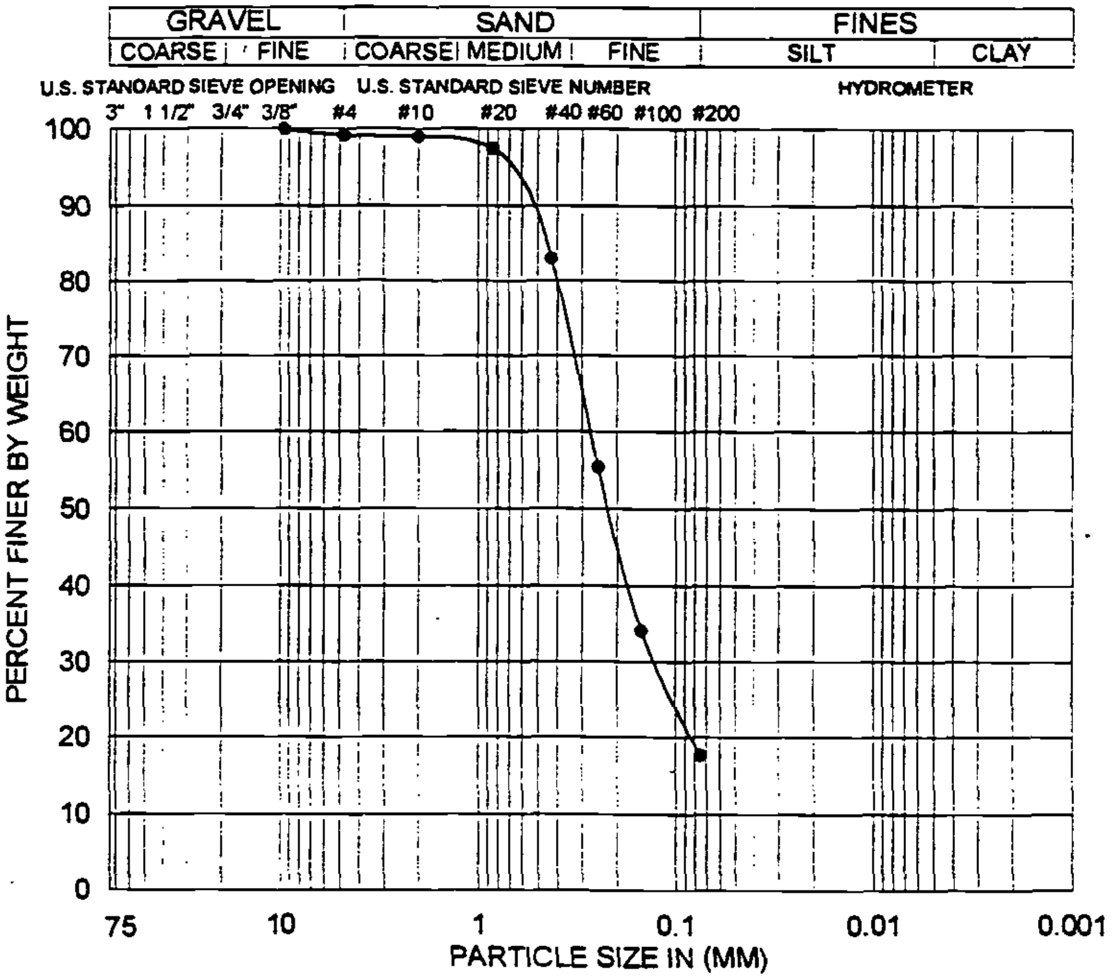
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-12





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-12	65.0	Bag	SM	01:81:18	N/A

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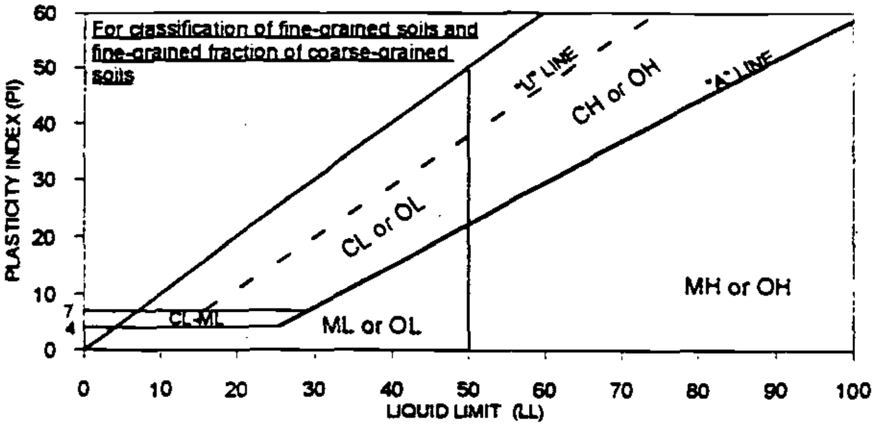
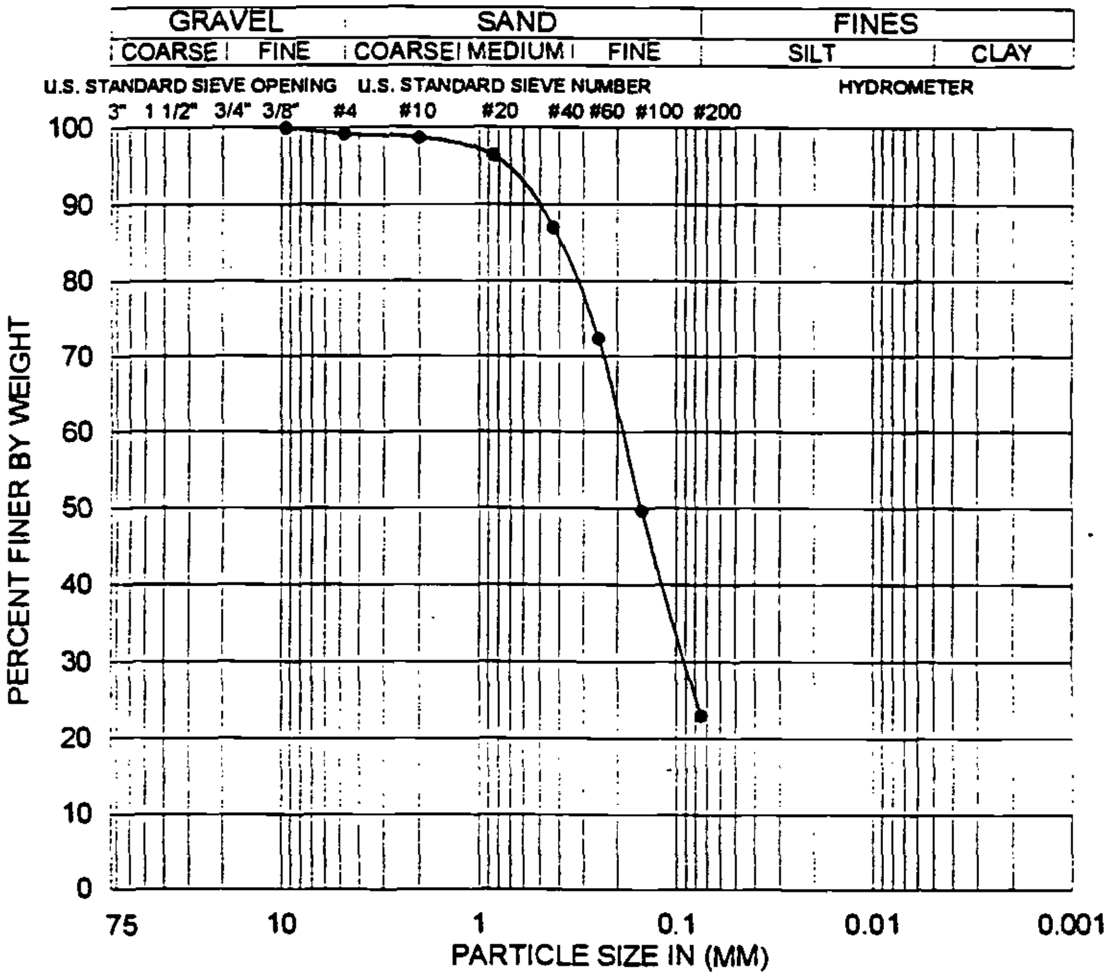
12-95

Figure B-13





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-15	75.0	Bag	SM	01:76:23	N/A

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(ASTM D4318, D422)

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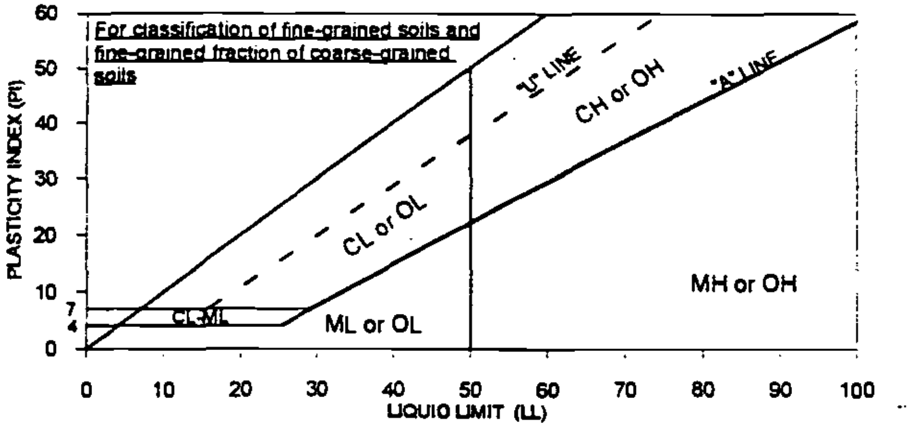
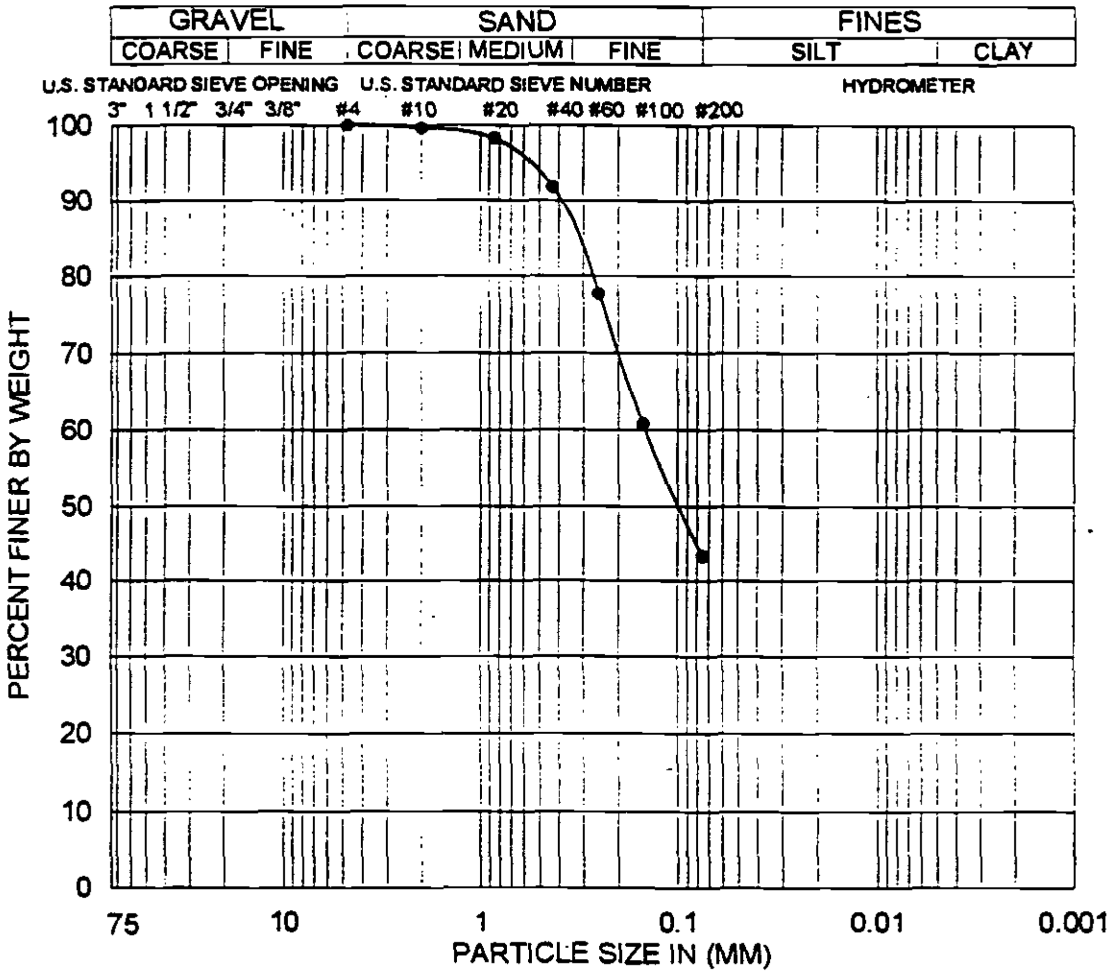
Figure B-14





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-16C	S-21	105.0	Bag	SC	00:57:43	N/A

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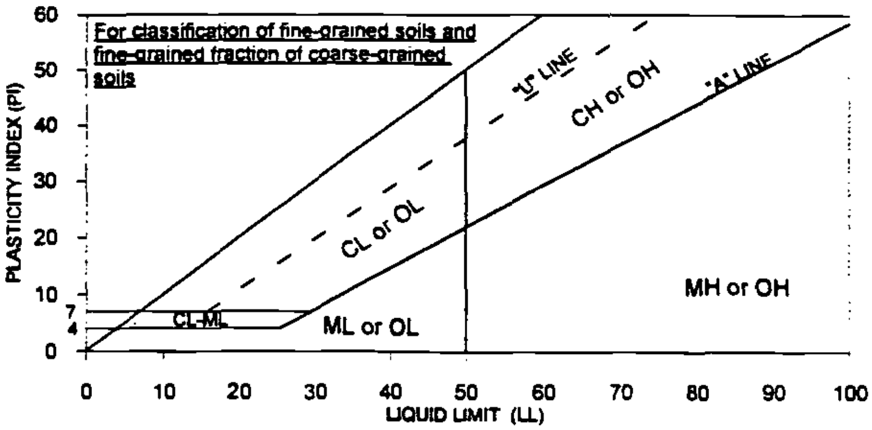
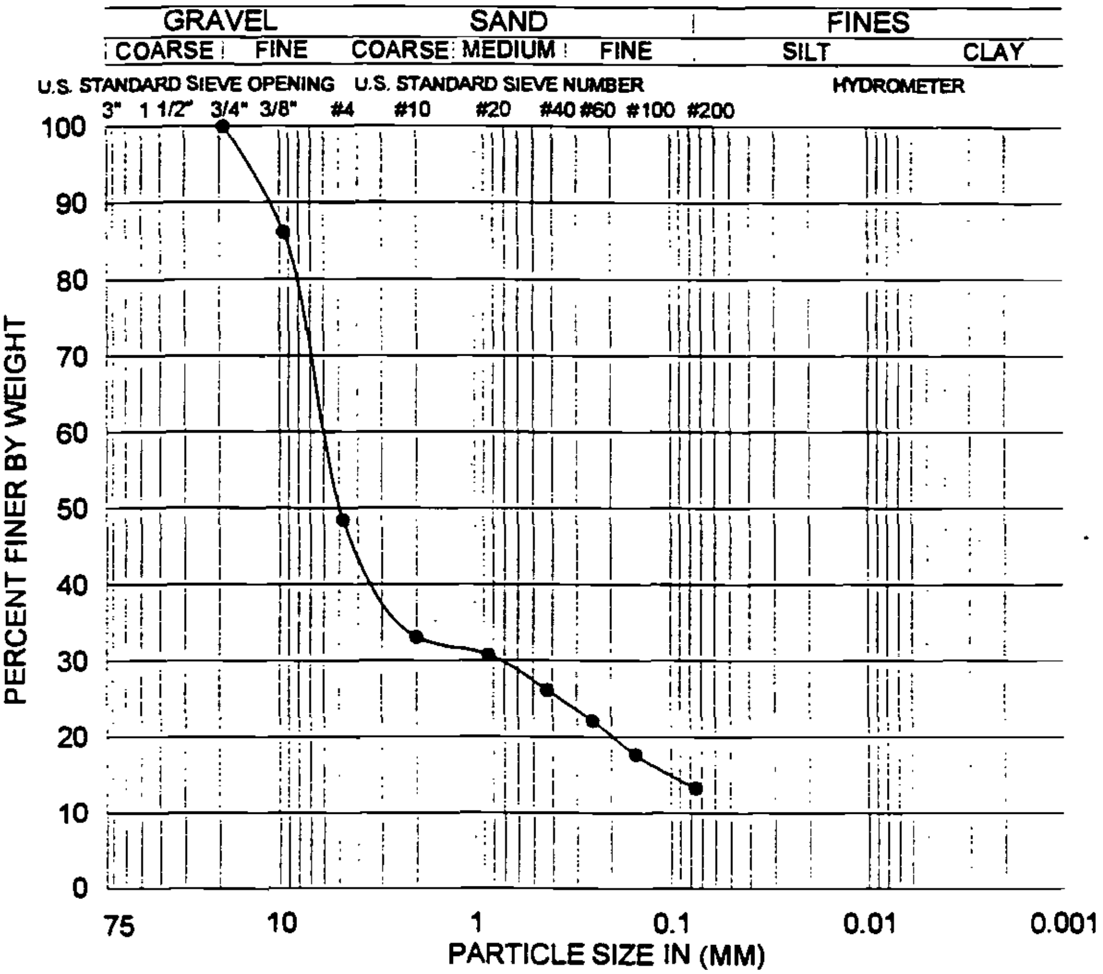
(ASTM D4318, D422)

12-95

Figure B-15





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	10.0	Bag	GM	52:35:13	N/A

Not enough sample to meet ASTM specification

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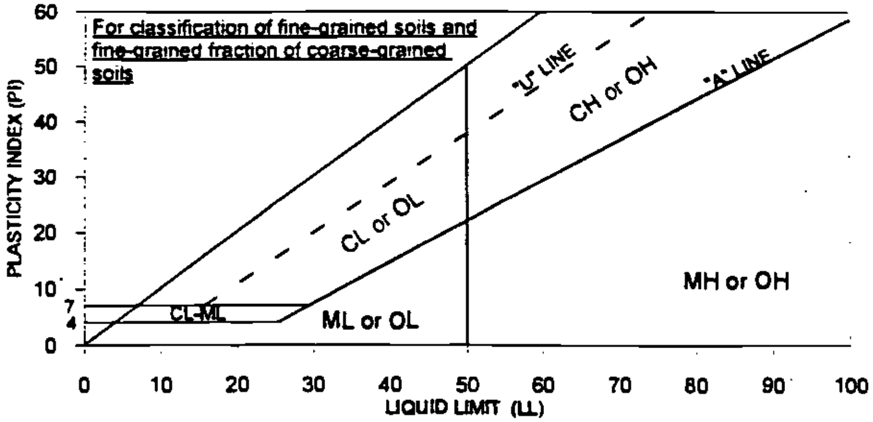
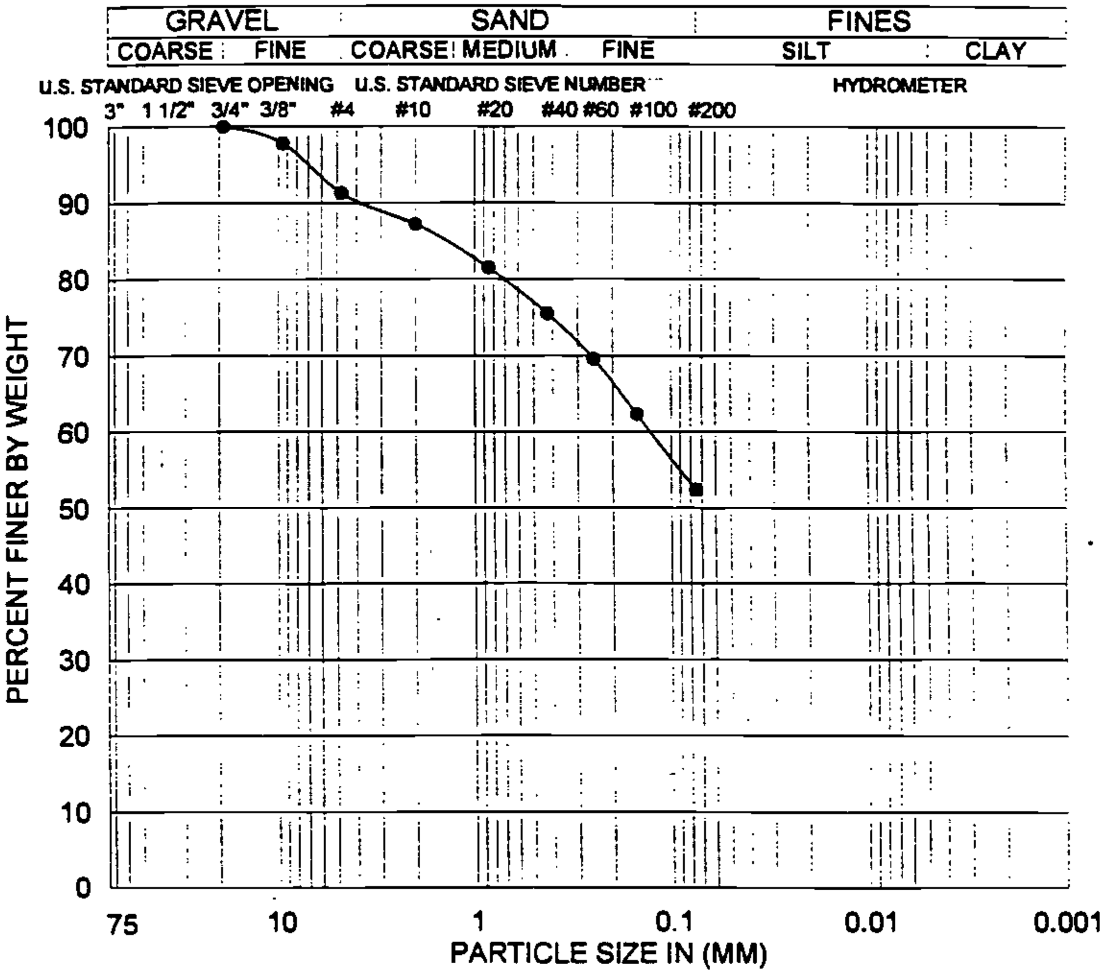
Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

12-95

Figure B-16





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	20.0	Bag	CL	9:39:52	N/A

Not enough sample to meet ASTM specification

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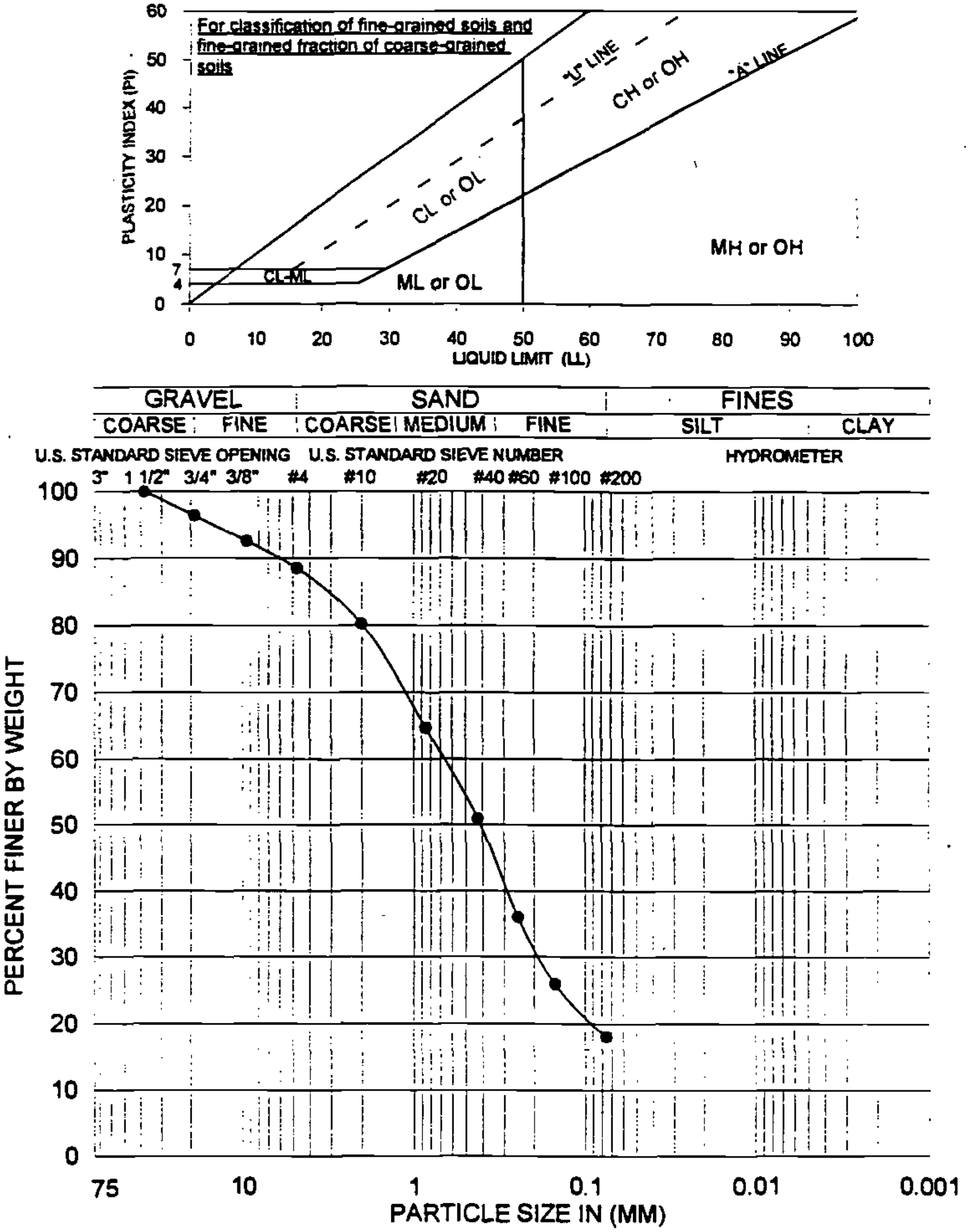
(ASTM D4318, D422)

12-95

Figure B-17



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	30.0	Bag	SM	11:71:18	N/A

Not enough sample to meet ASTM specification

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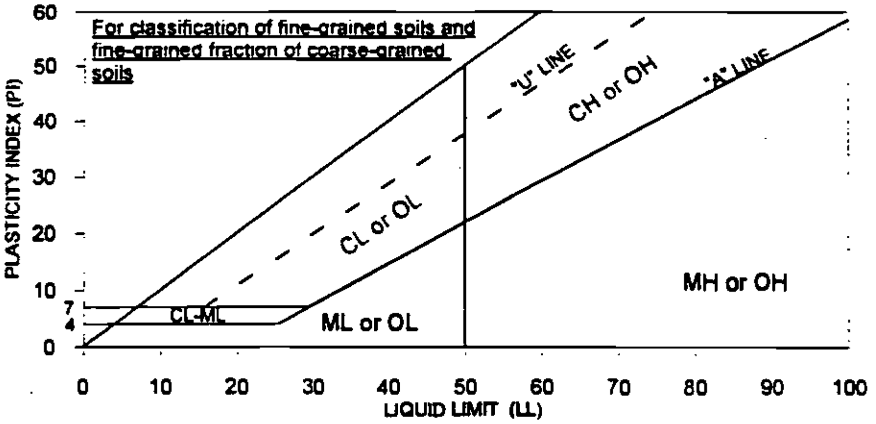
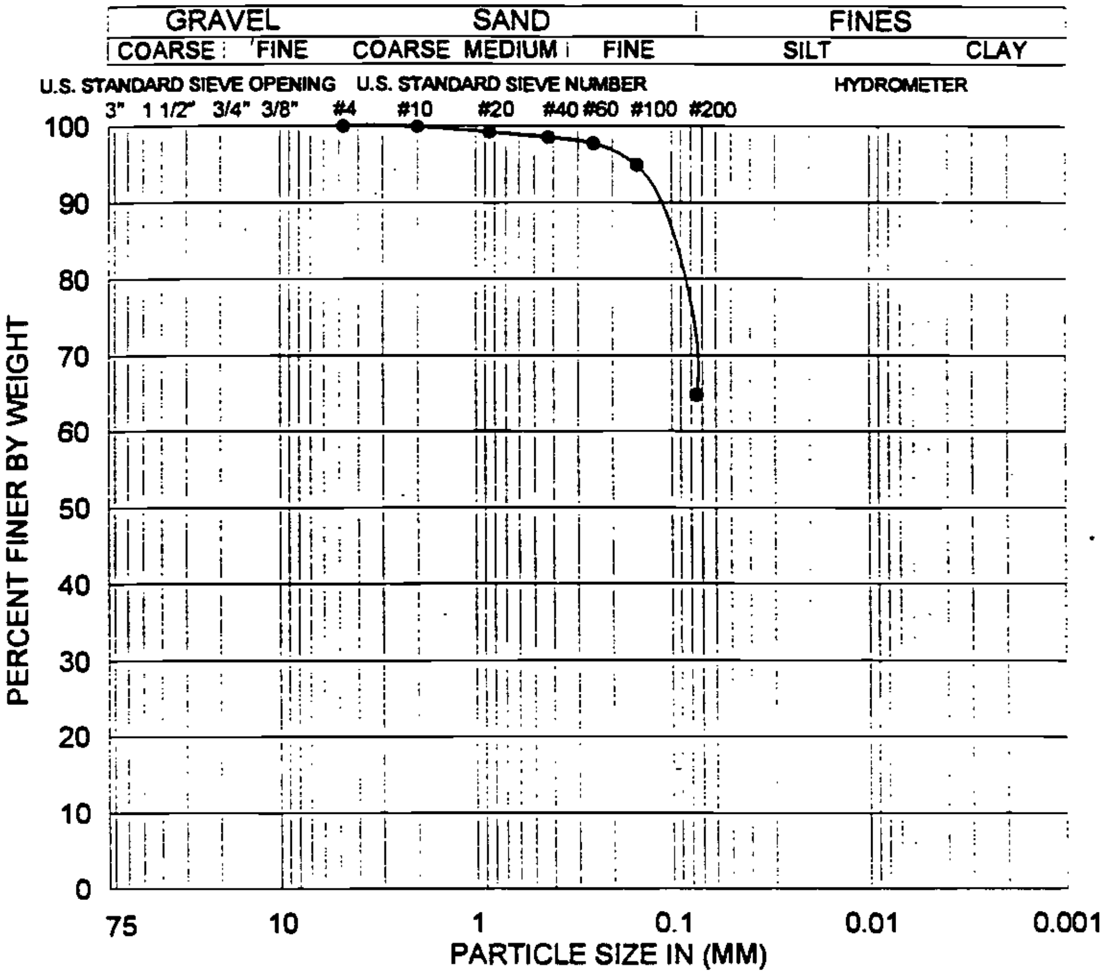
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-18





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	40.0	Bag	CL	0:35:65	N/A

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Atterberg Limits, Particle-Size Curve
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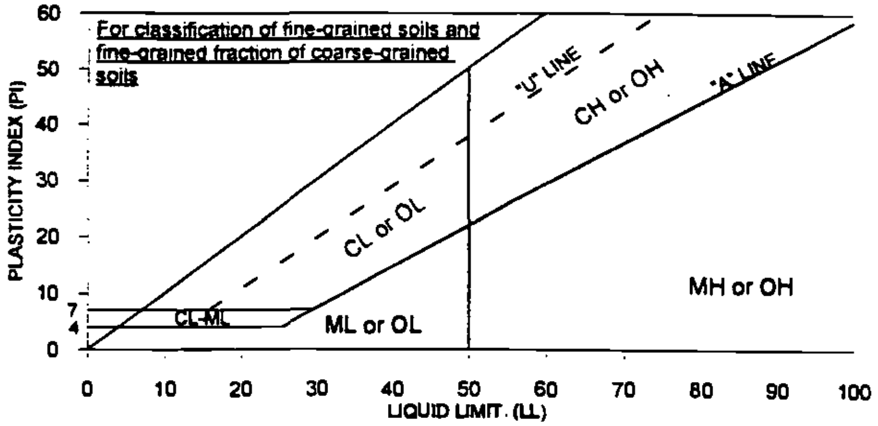
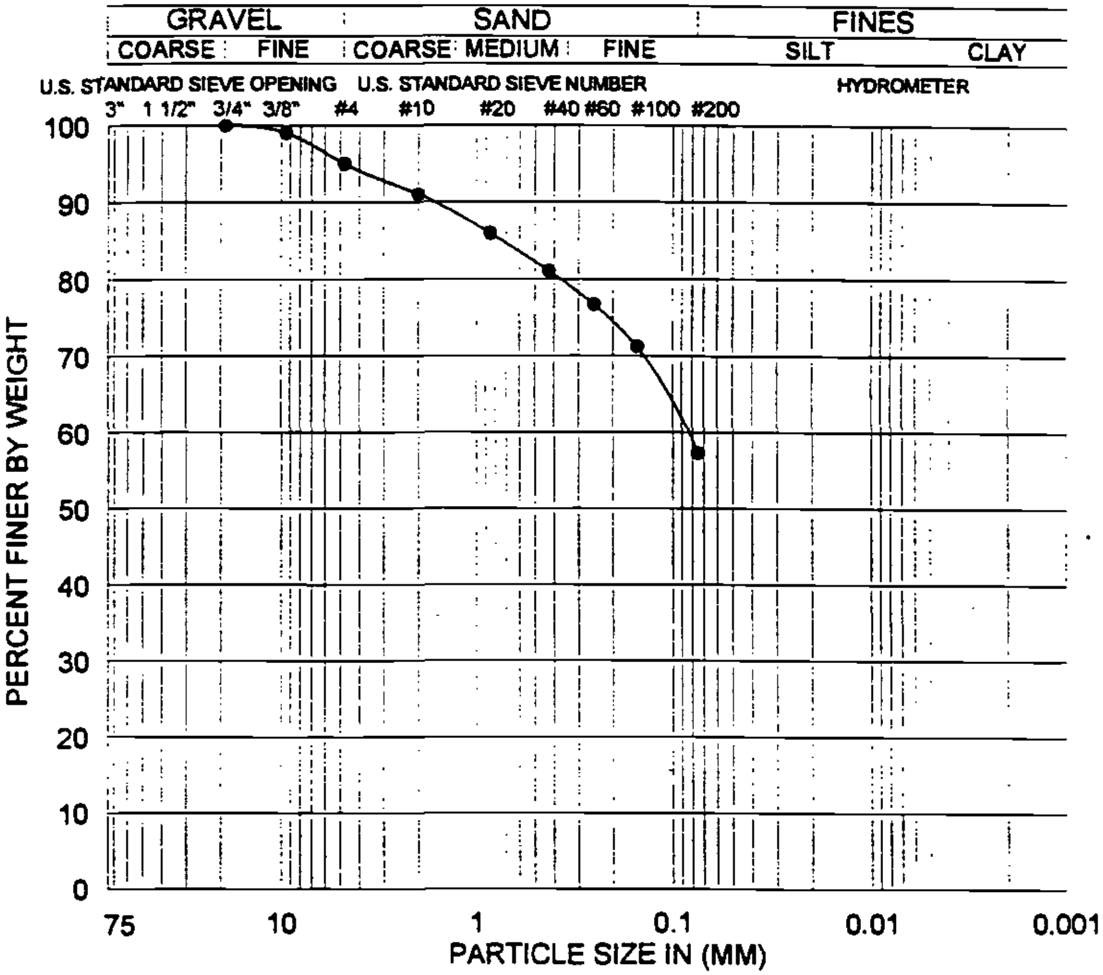
Figure B-19





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	60.0	Bag	CL	5:38:57	N/A

Not enough sample to meet ASTM specification

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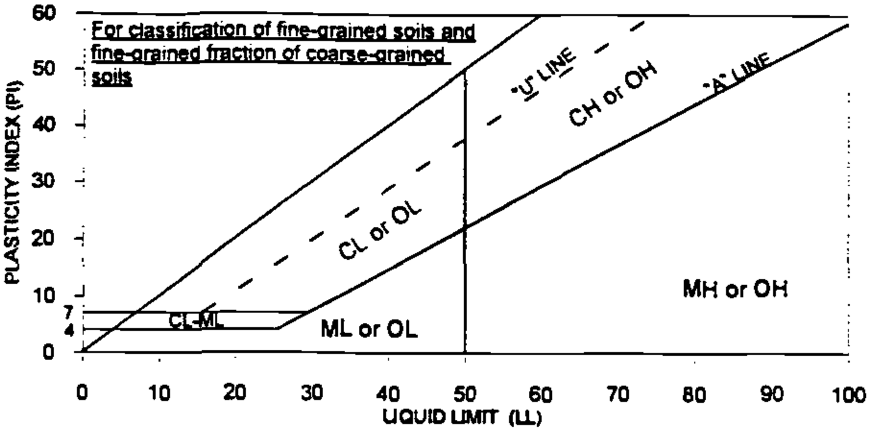
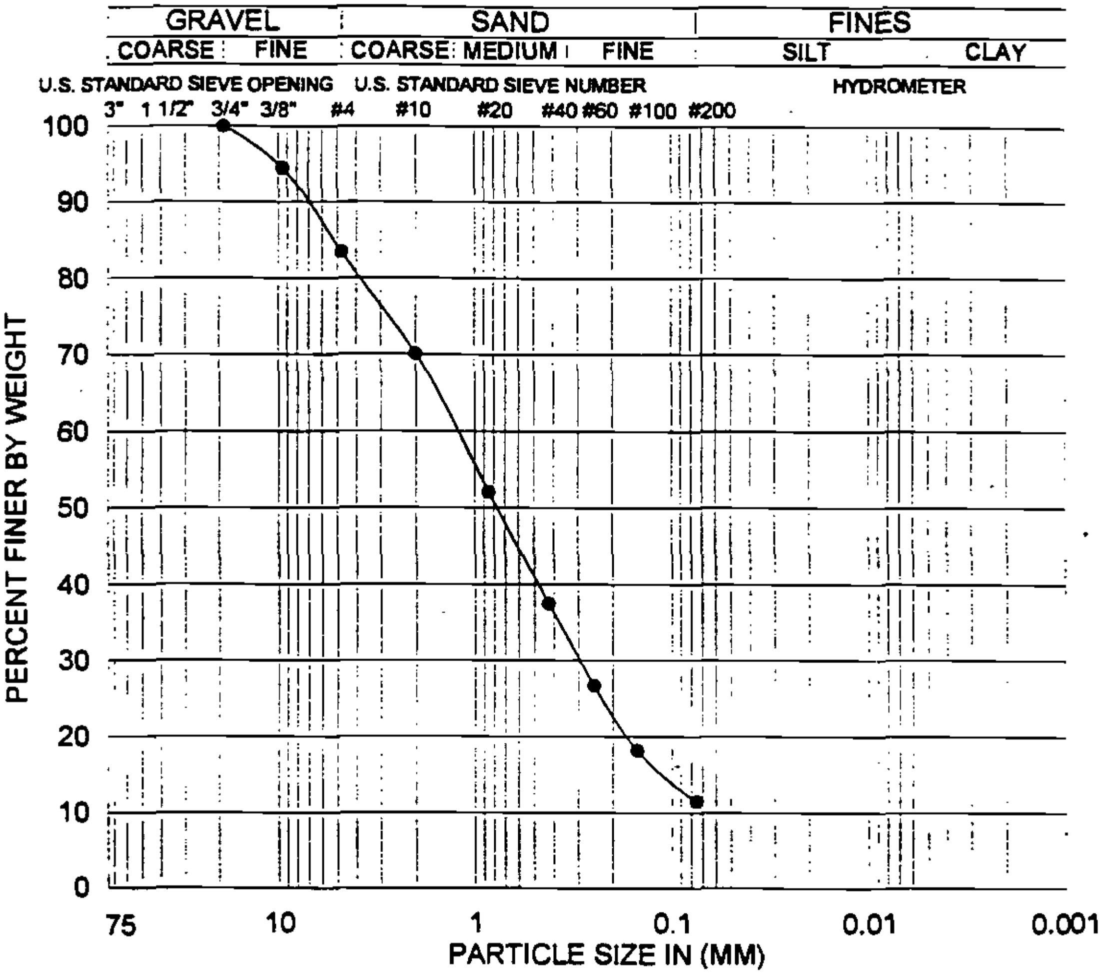
Metro Redline II

Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

12-95

Figure B-20





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	70.0	Bag	SW-SM	17:71:12	N/A

Not enough sample to meet ASTM specification

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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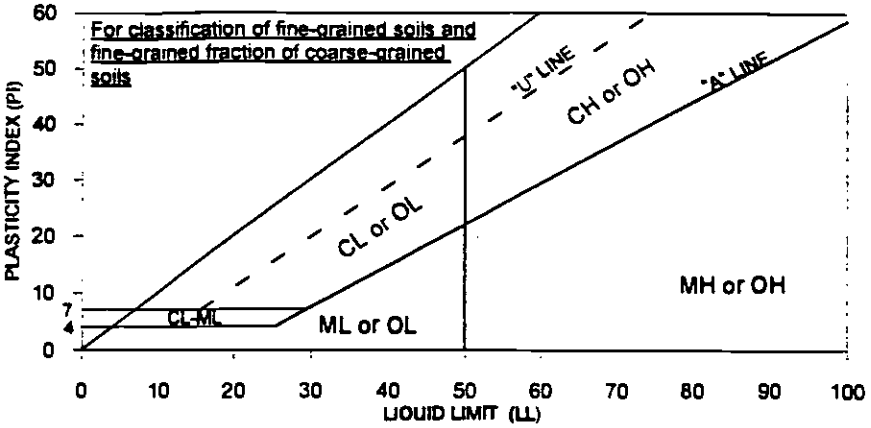
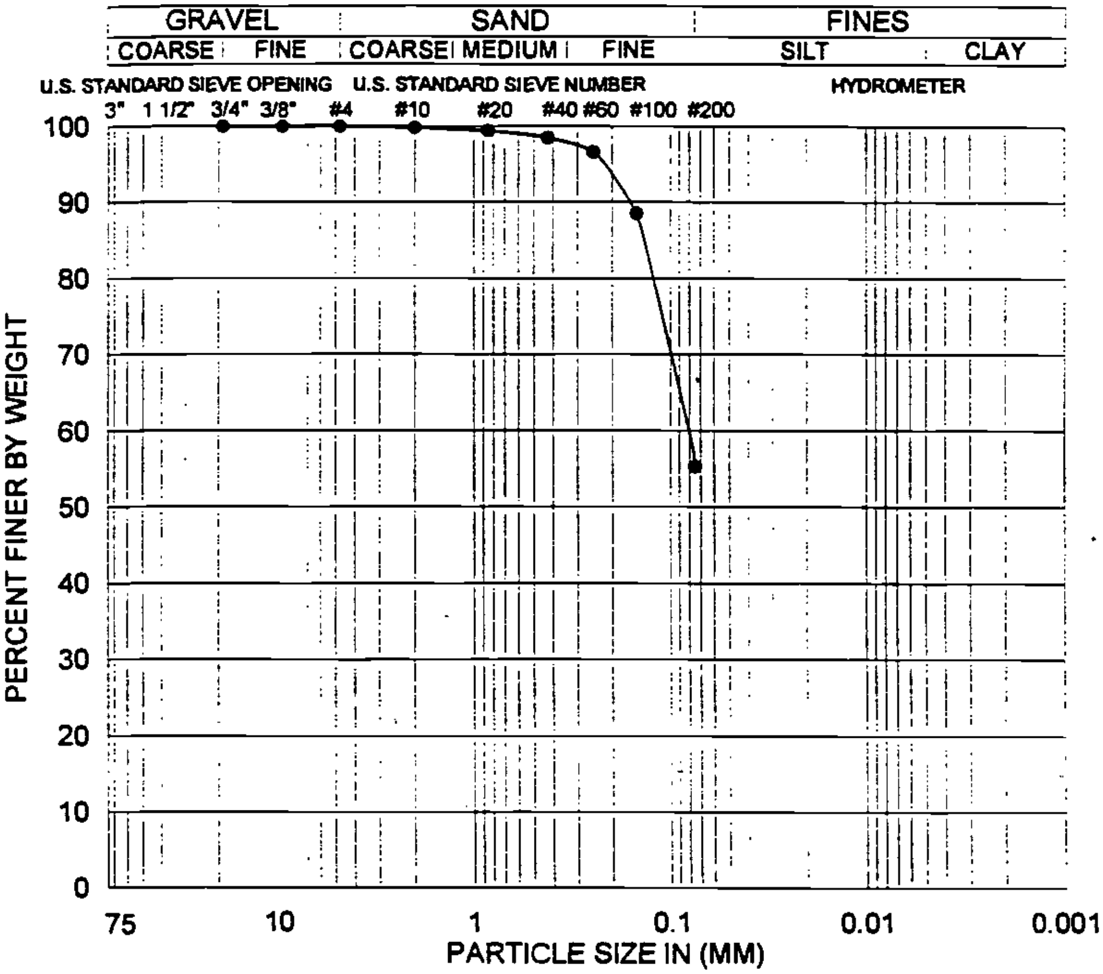
Figure B-21





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-17	N/A	80.0	Bag	ML	0:45:55	N/A

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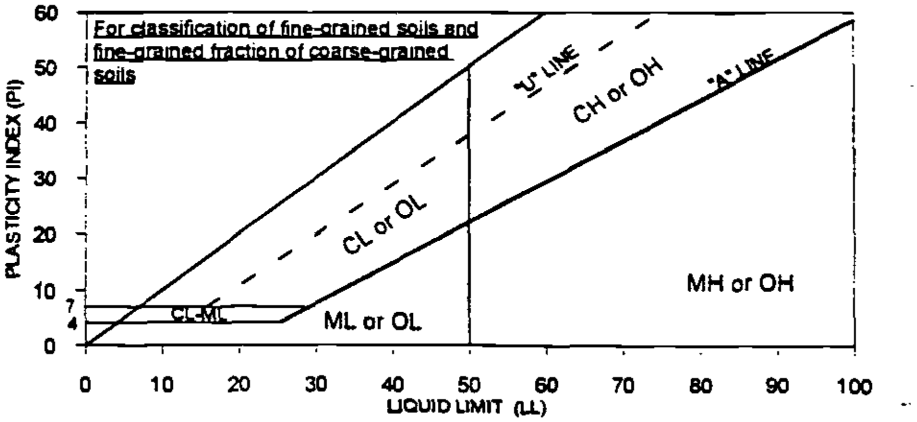
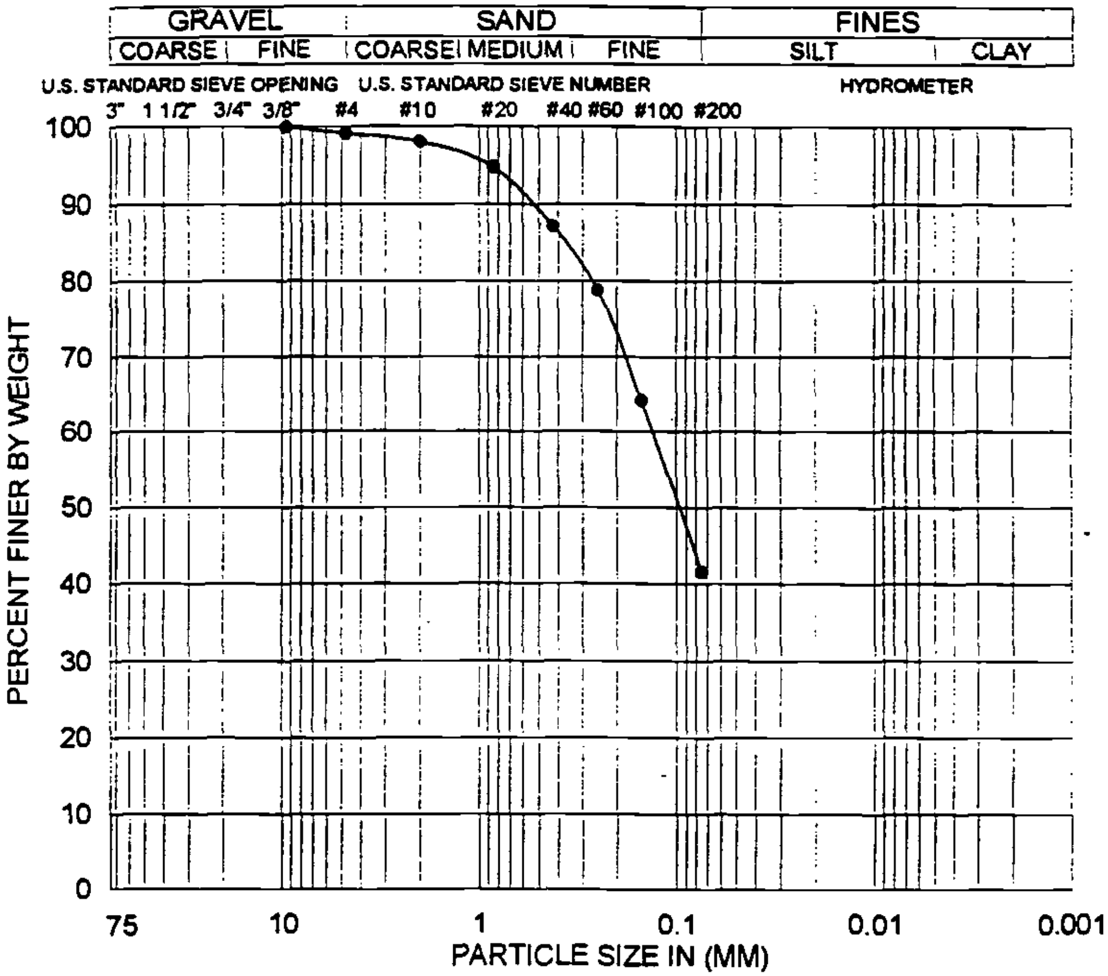
Figure B-22





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-4	20.0	Bag	SM	01:57:42	N/A

Not enough sample to meet ASTM specification

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Metro Redline II

Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

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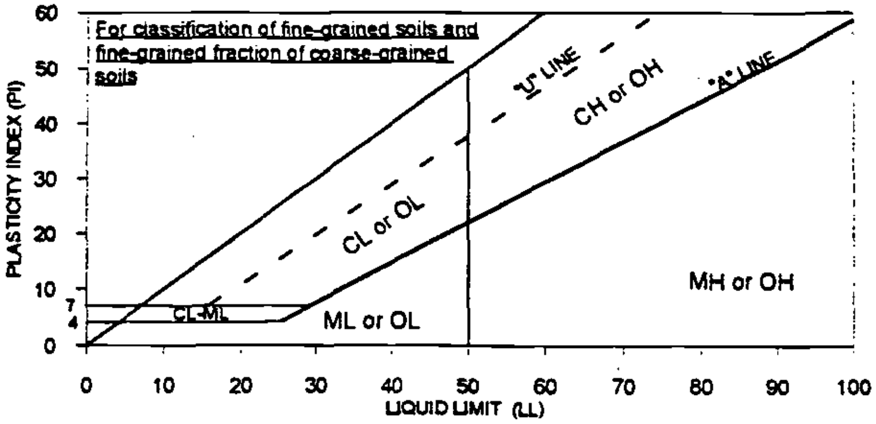
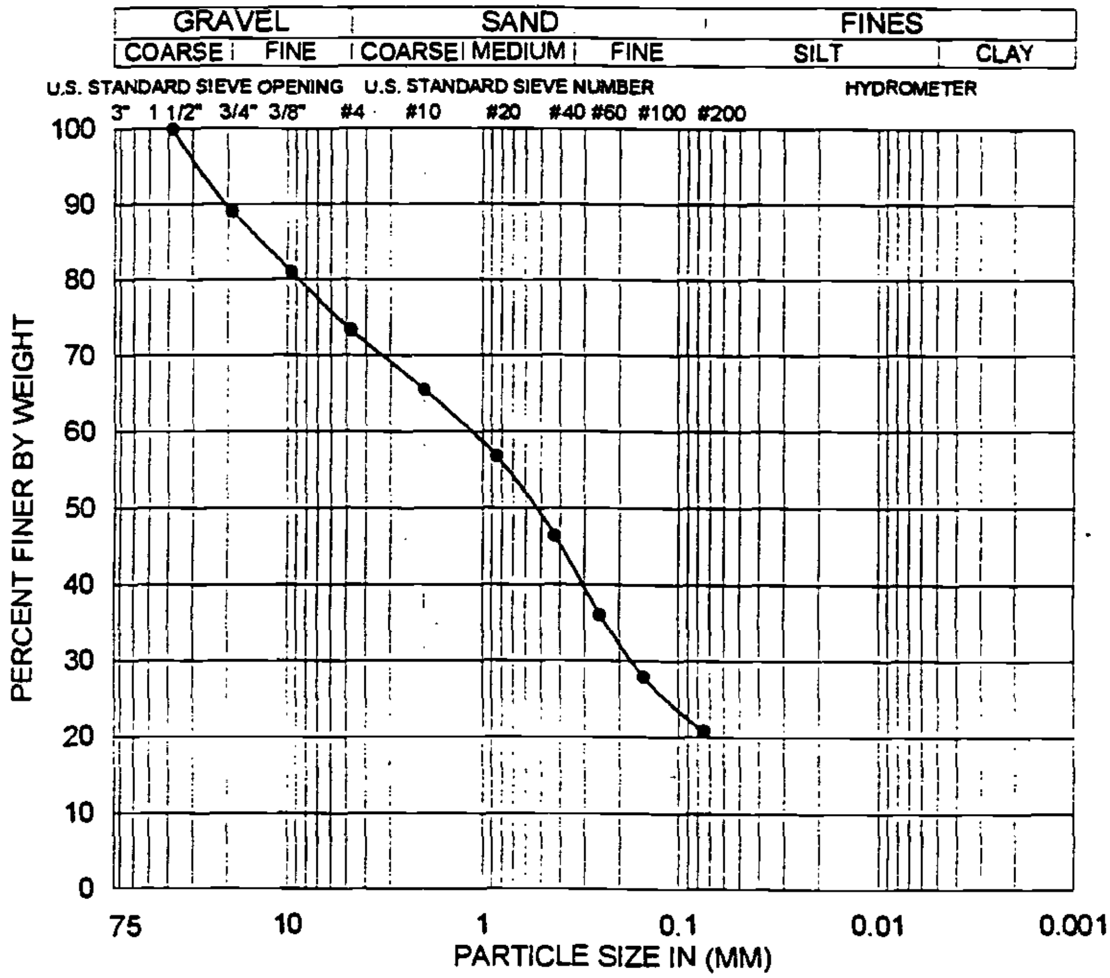
Figure B-23





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-8	40.0	Bag	SM	27:52:21	N/A

Not enough sample to meet ASTM specification

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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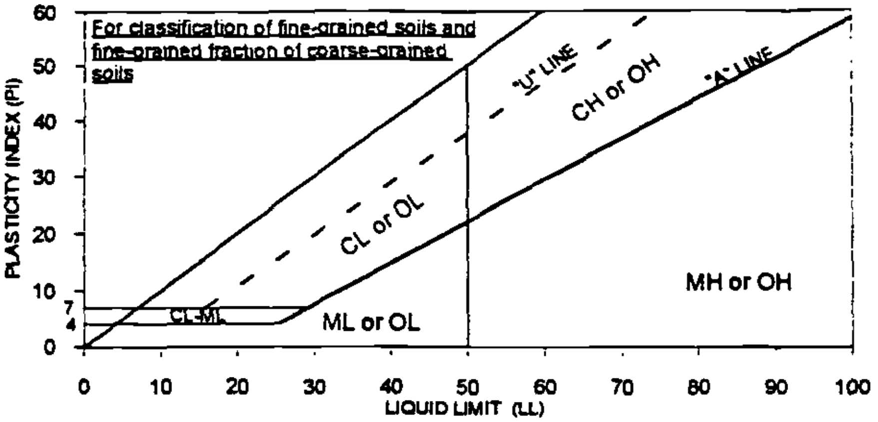
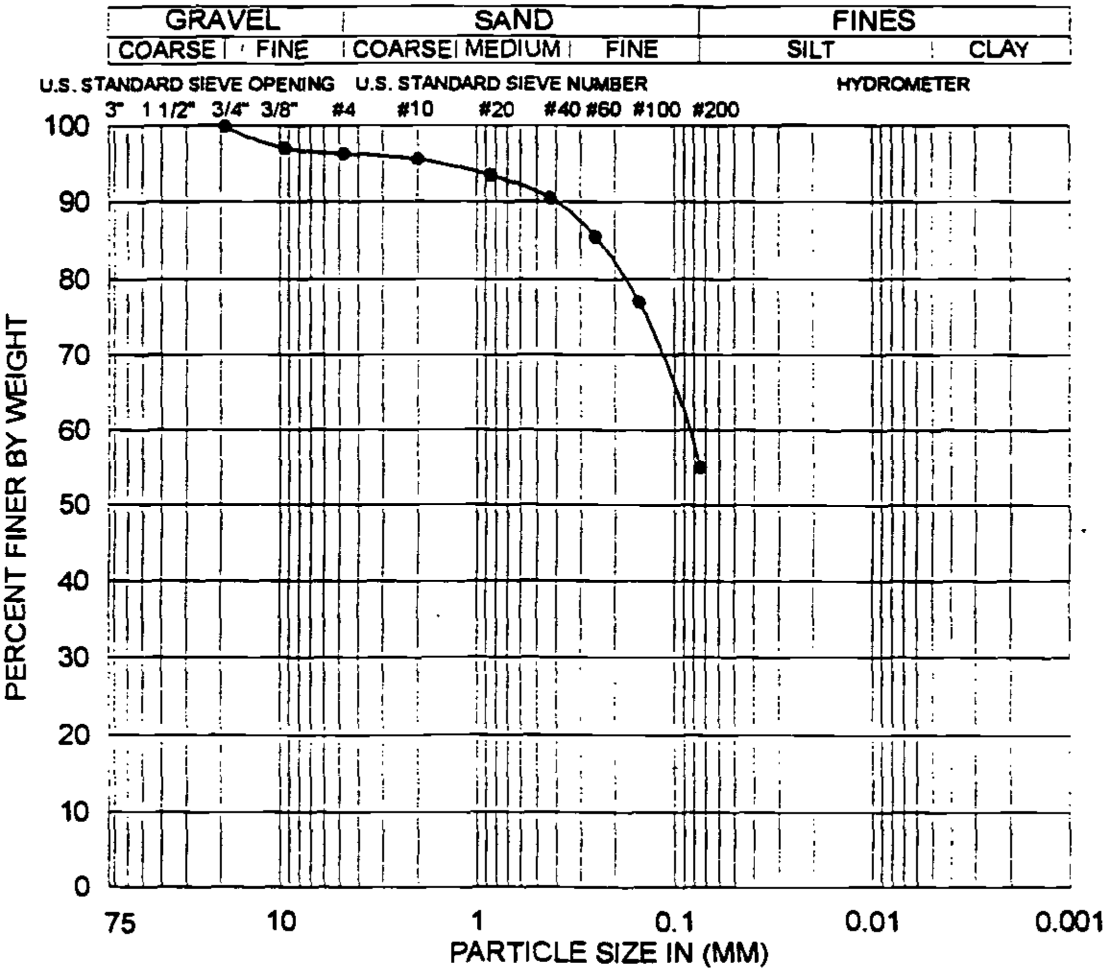
Figure B-24





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-10	50.0	Bag	ML	04:41:55	N/A

Not enough sample to meet ASTM specification

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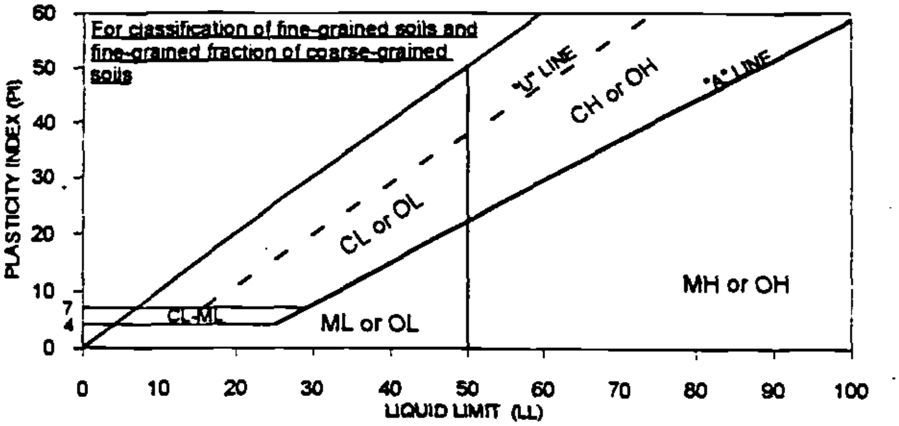
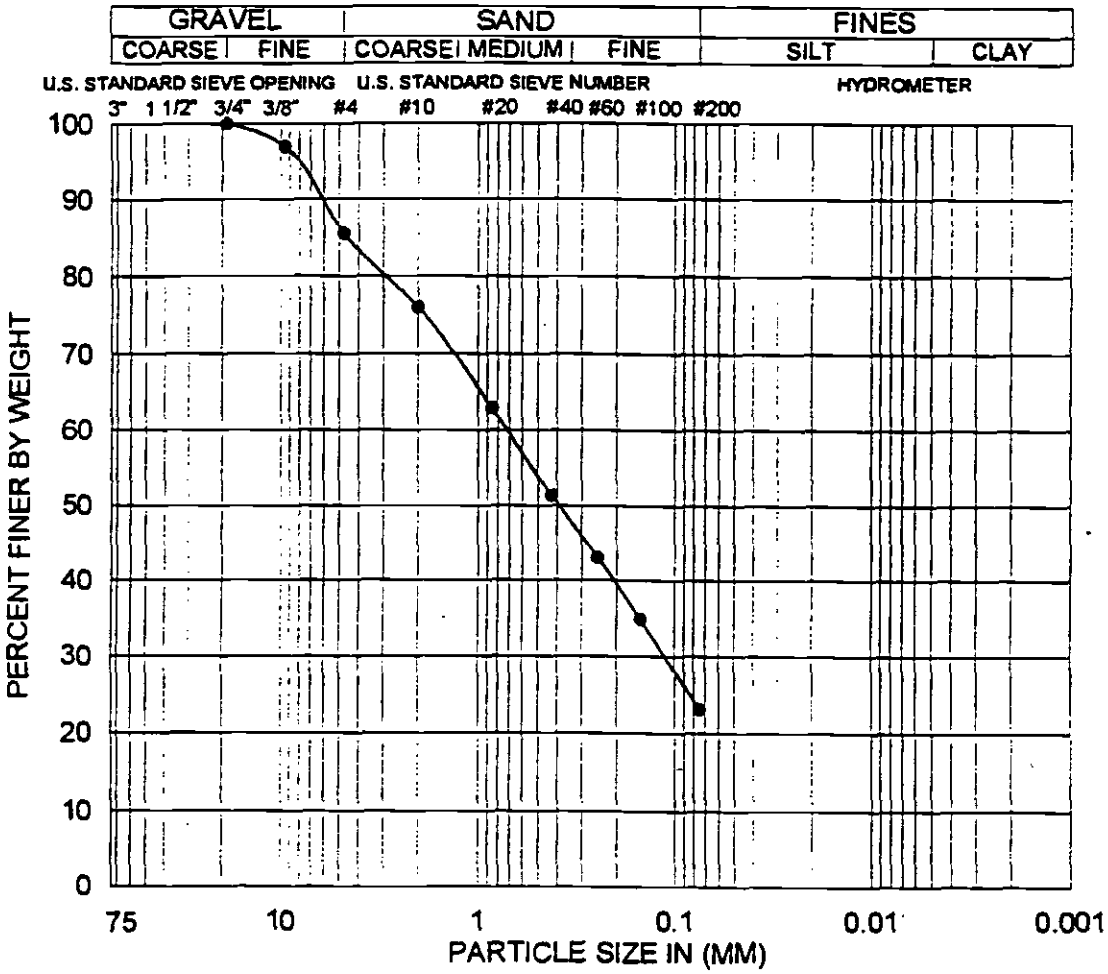
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-25





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-12	60.0	Bag	SM	14:63:23	N/A

Not enough sample to meet ASTM specification

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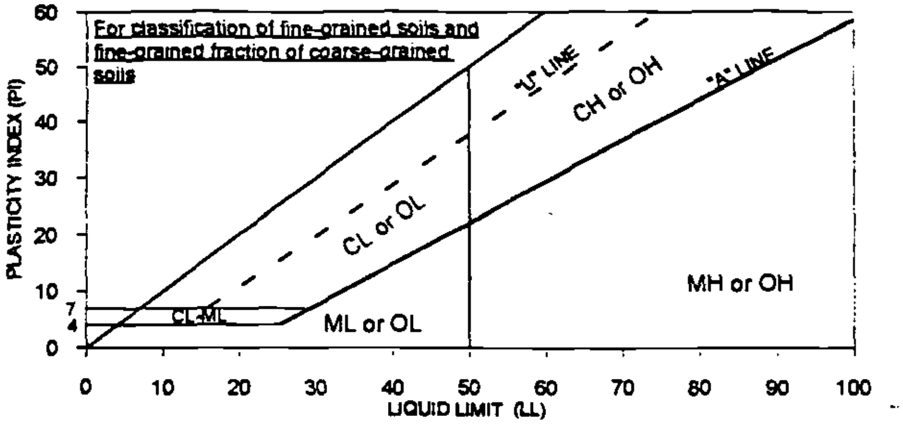
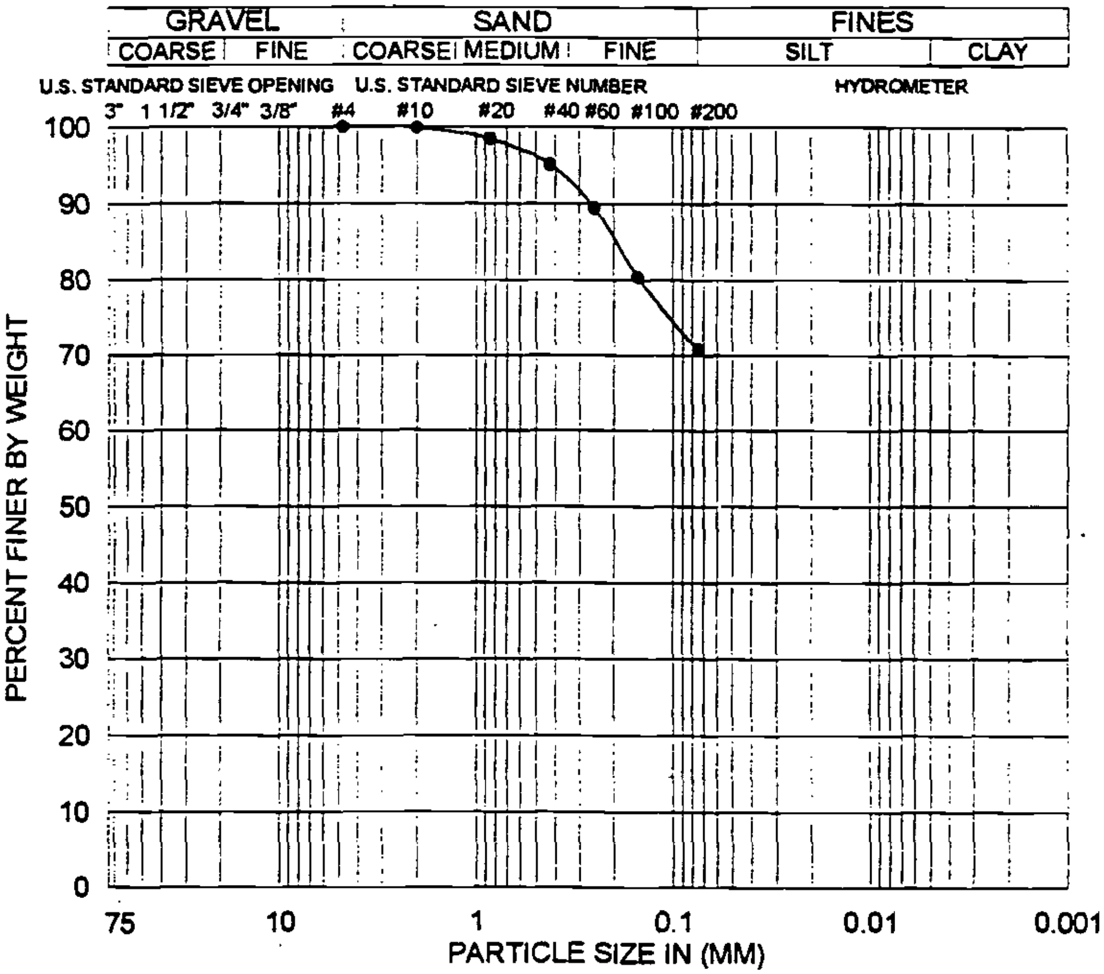
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-26





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-16	75.0	Bag	ML	00:29:71	N/A

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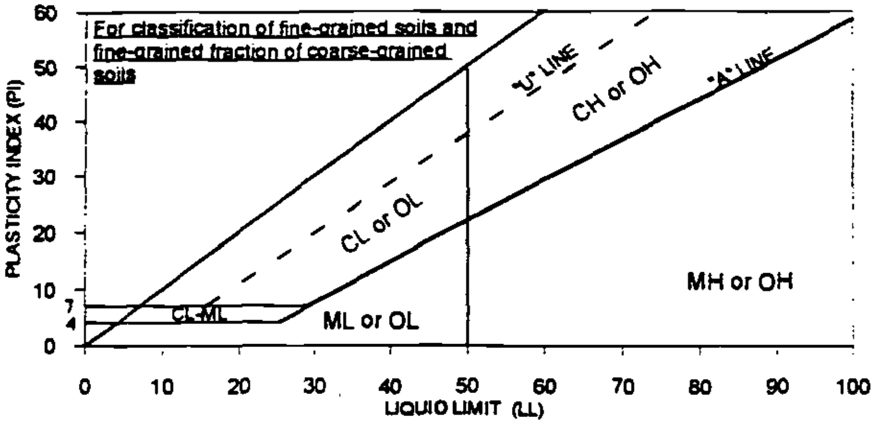
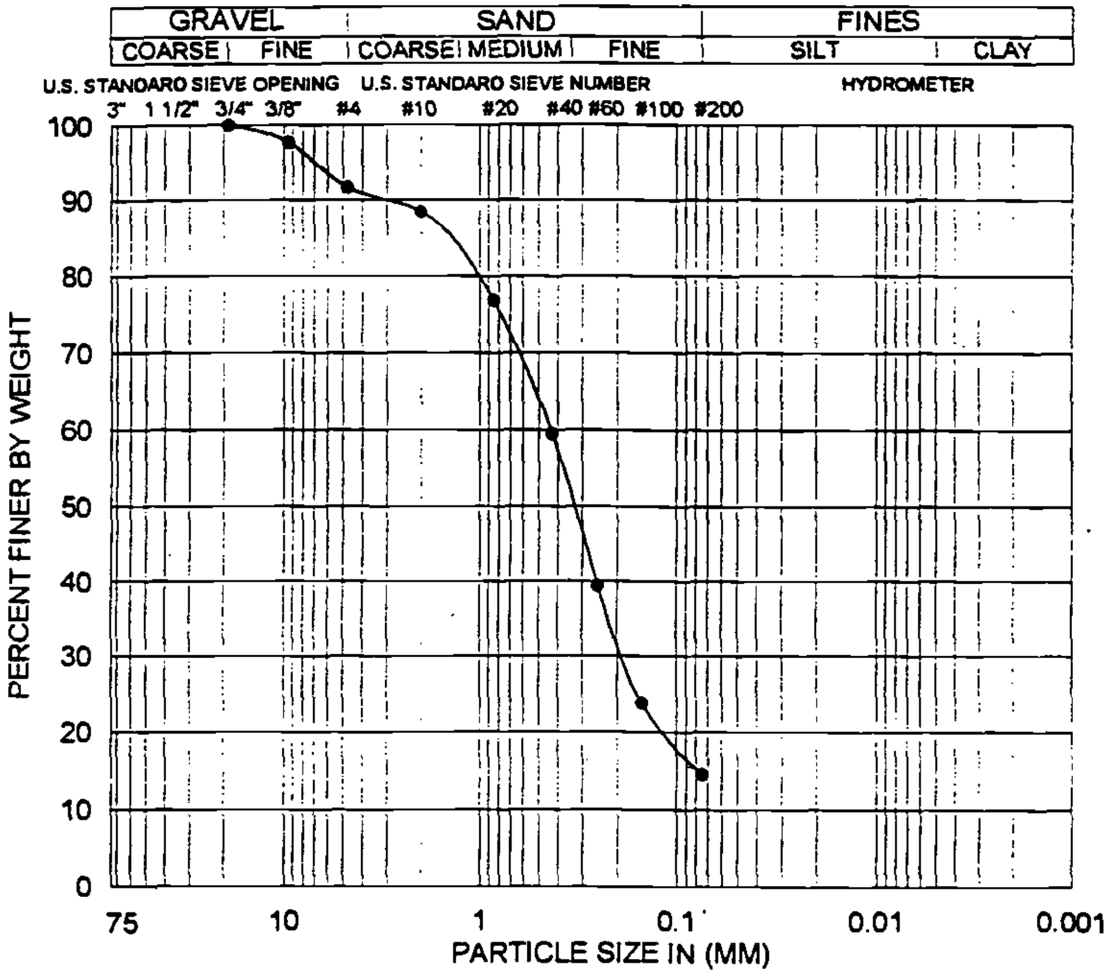
**Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)**

12-95

Figure B-27





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
SD-18C	S-20	90.0	Bag	SM	08:77:15	N/A

Not enough sample to meet ASTM specification

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Metro Redline II

Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

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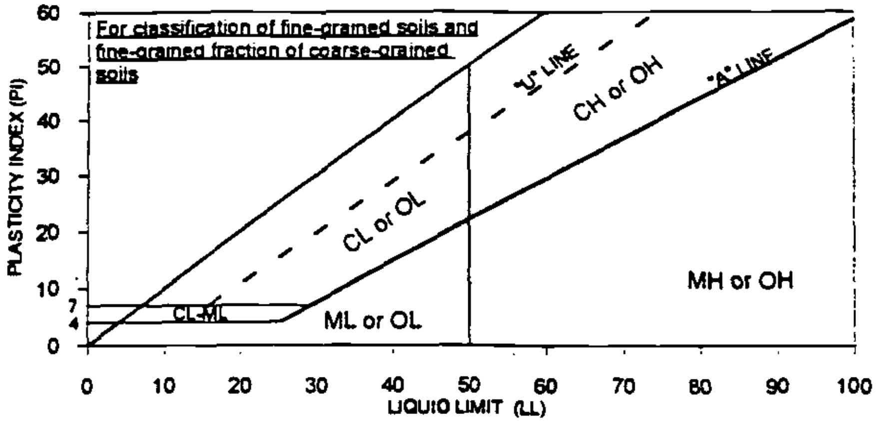
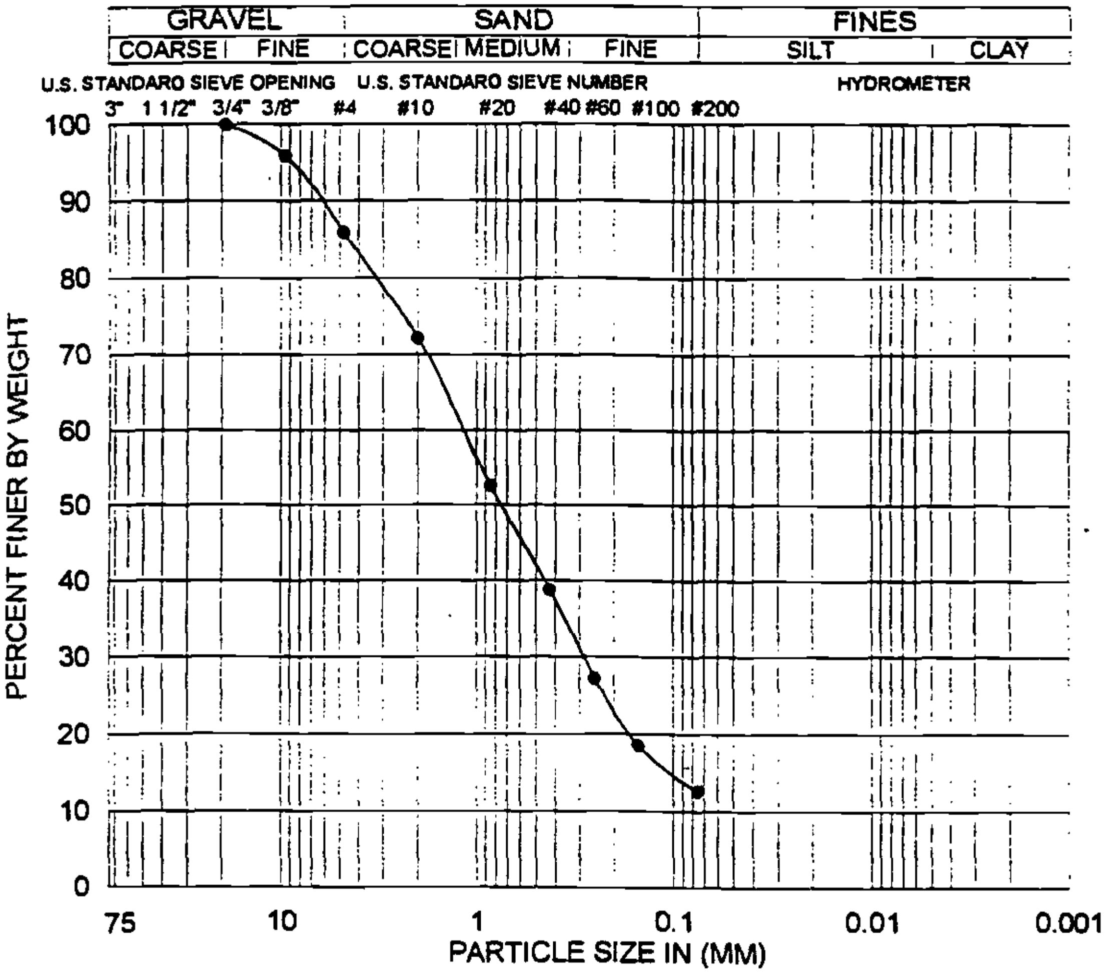
Figure B-28





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27	S-4	21.0	Bag	SM	14:73:13	N/A

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Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

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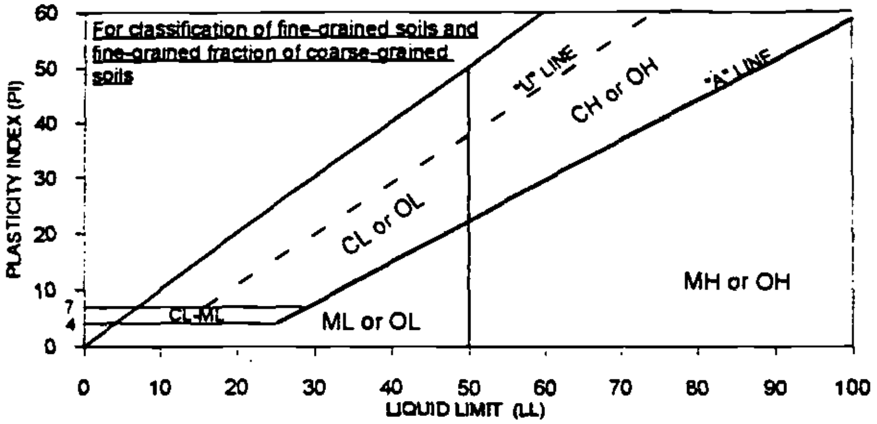
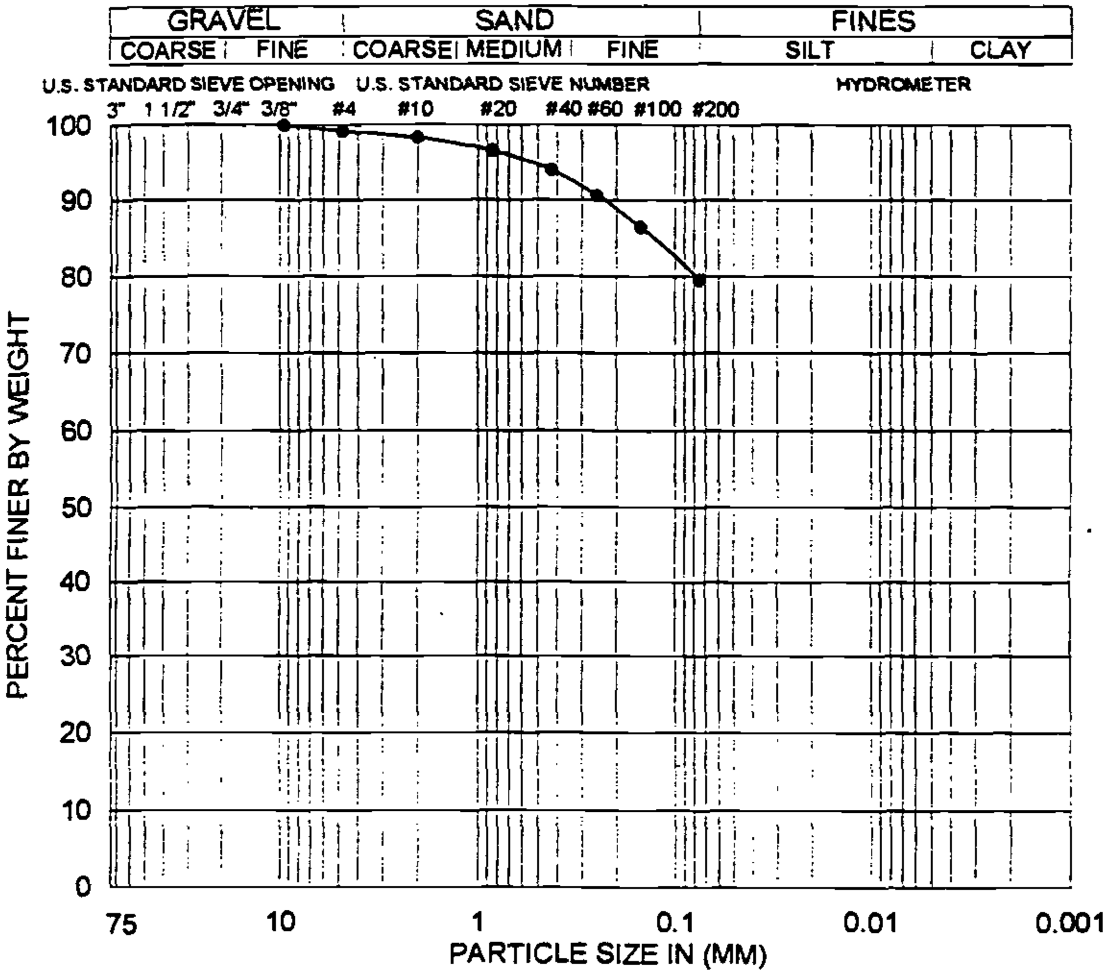
Figure B-29





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27	S-9	35.0	Bag	CL	01:19:80	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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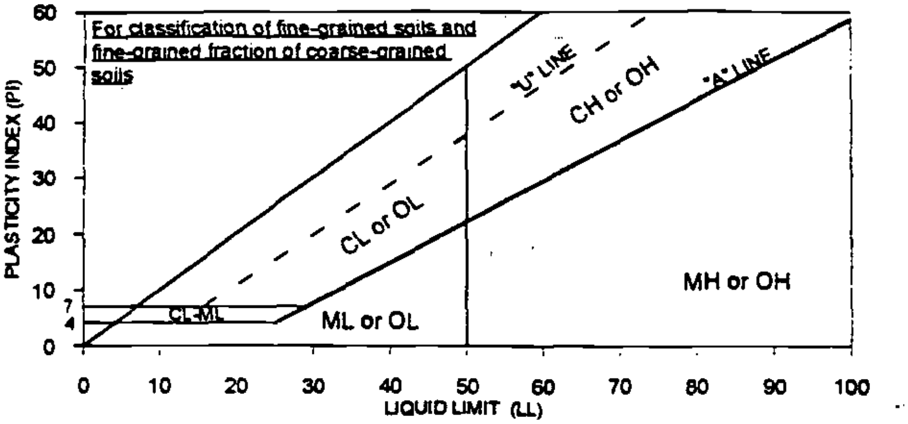
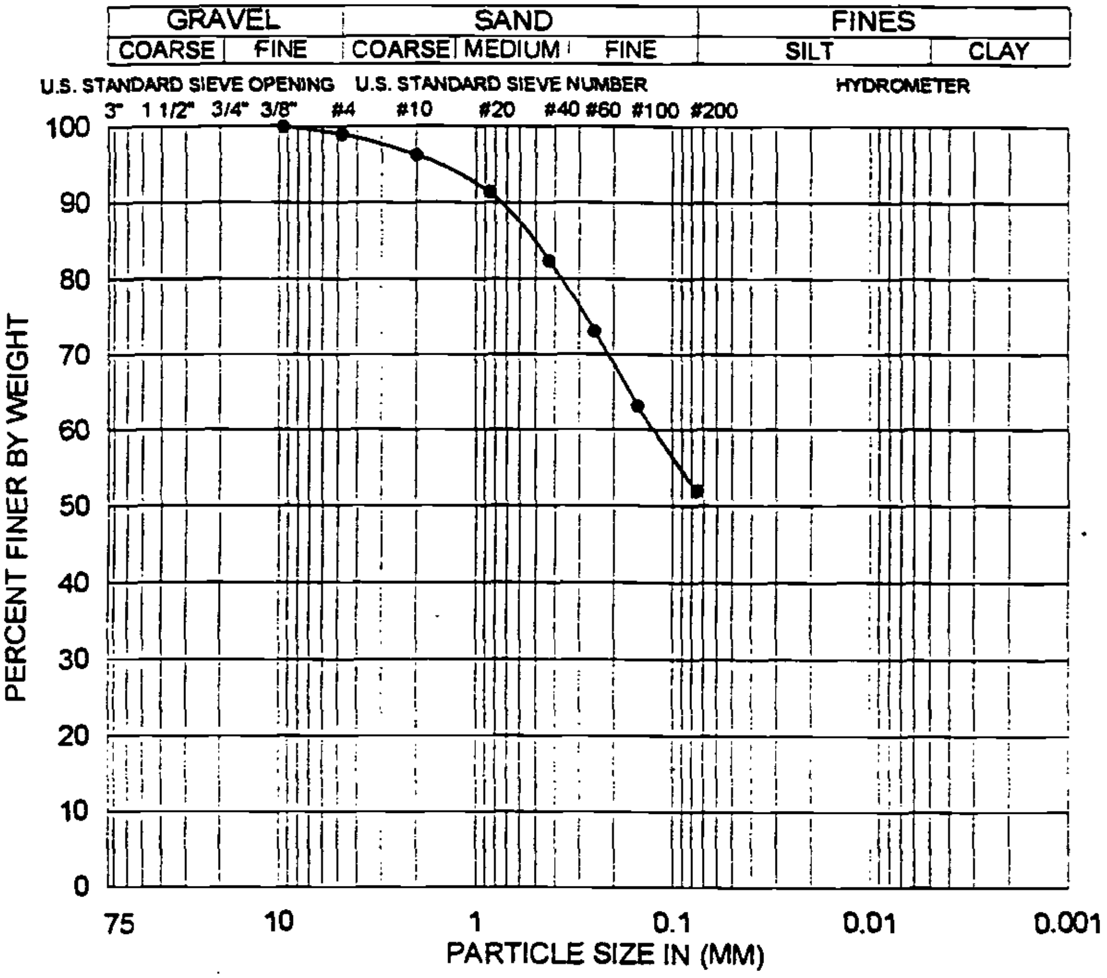
Figure B-30





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27	S-12	47.0	Bag	CL	01:47:52	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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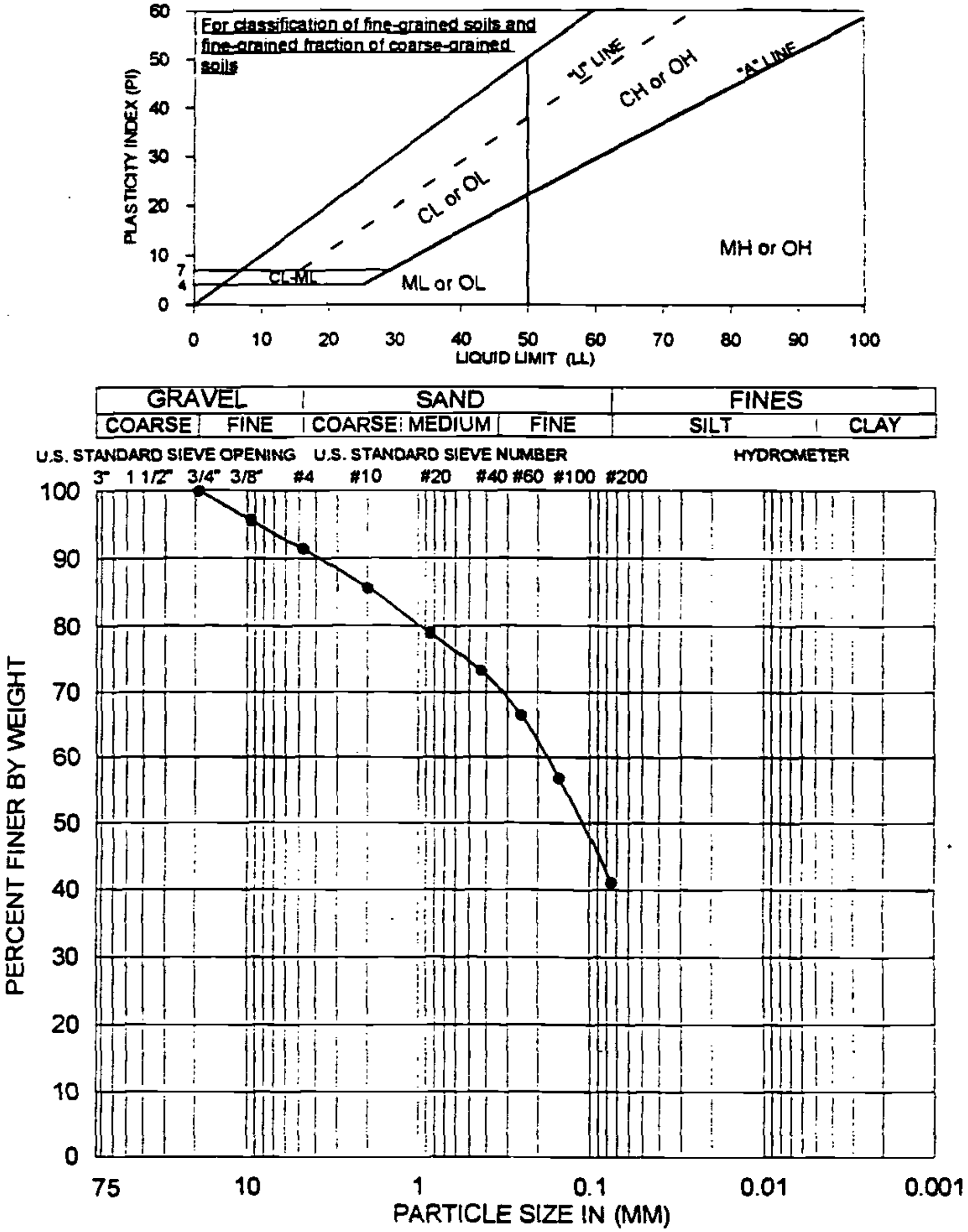
Figure B-31



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27	S-14	53.5	Bag	SC	9:50:41	N/A

Not enough sample to meet ASTM specification

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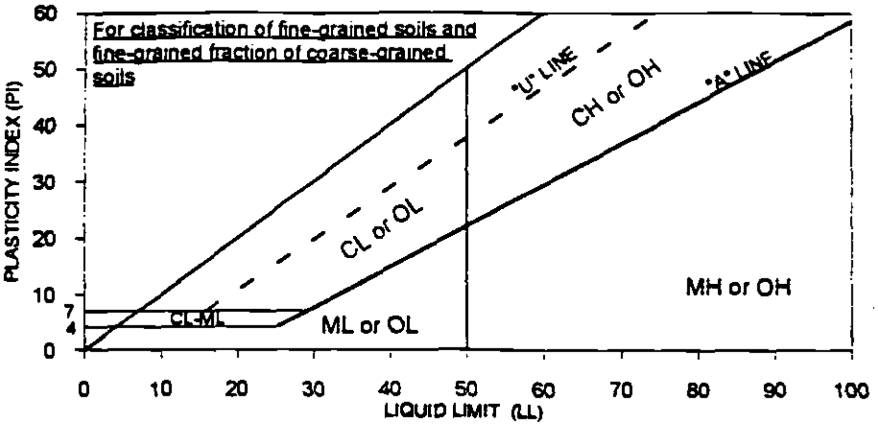
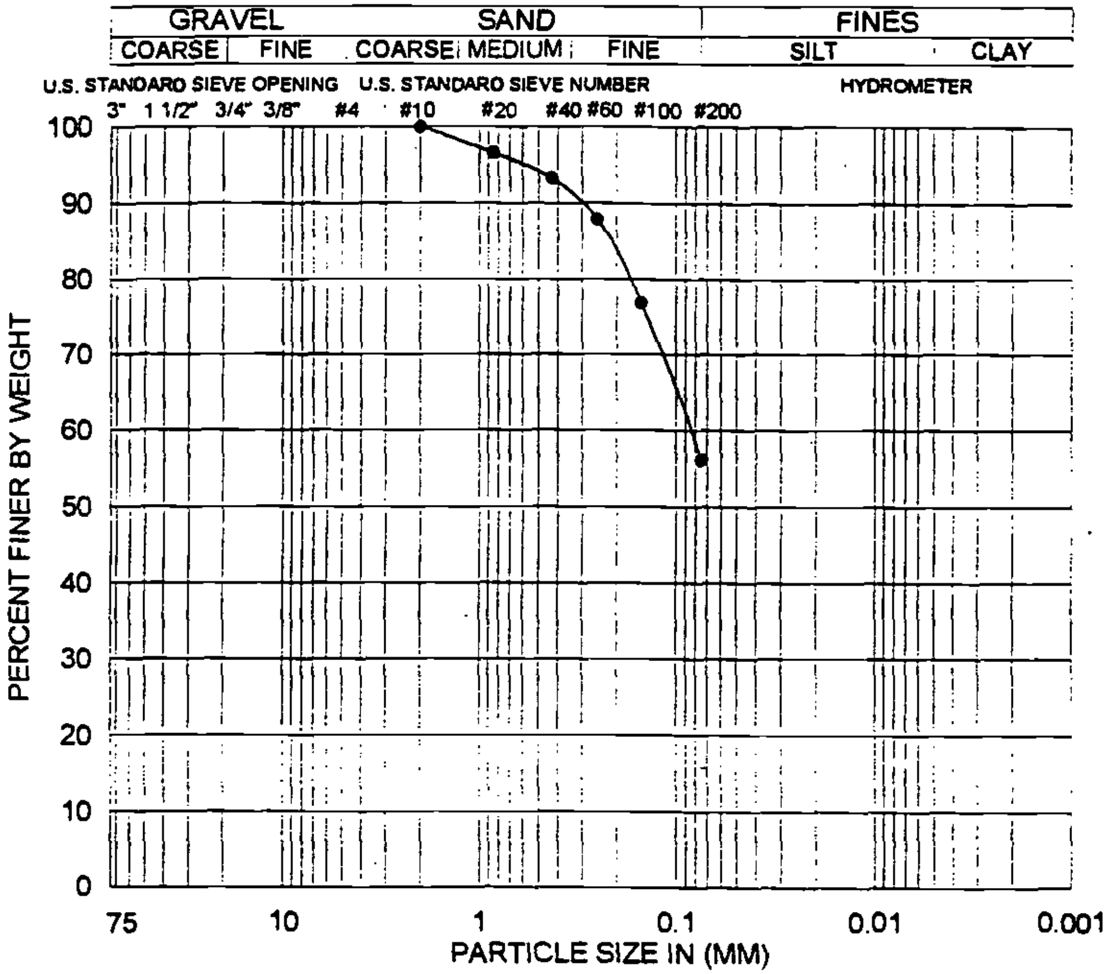
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-17	60.0	Bag	ML	00:44:56	N/A

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Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

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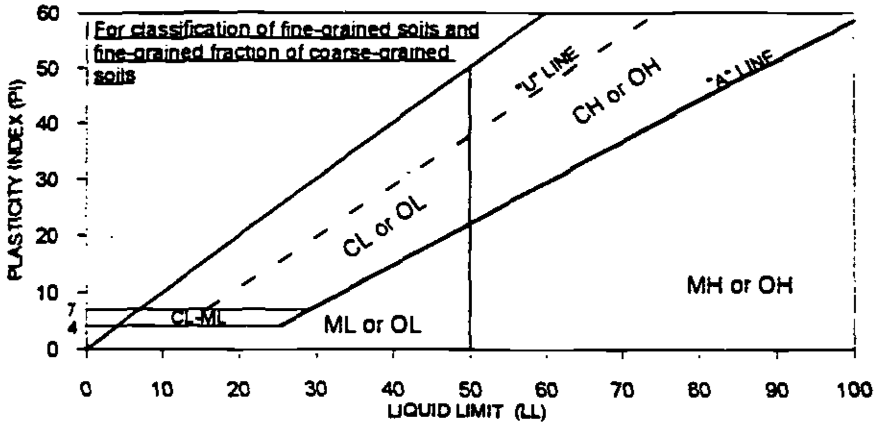
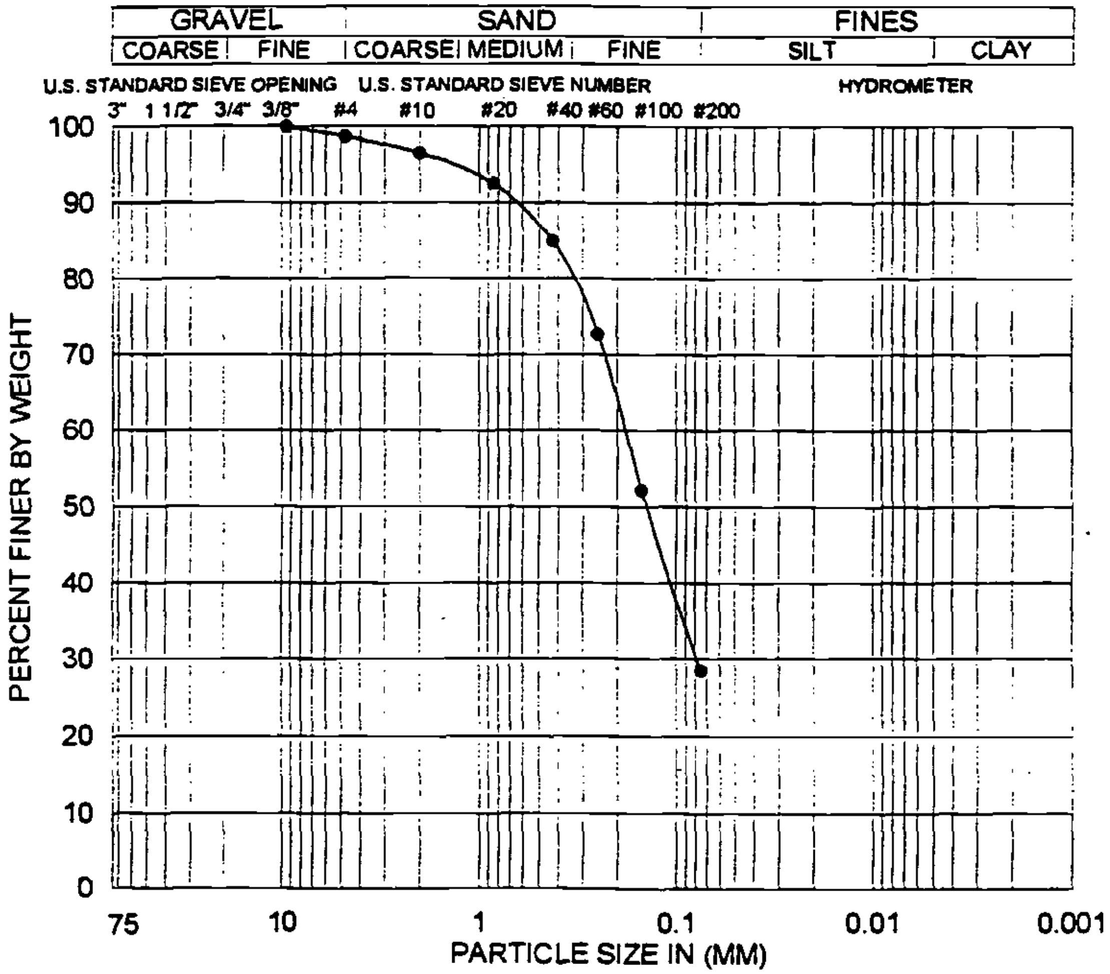
Figure B-33





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-19	66.5	Bag	SM	01:70:29	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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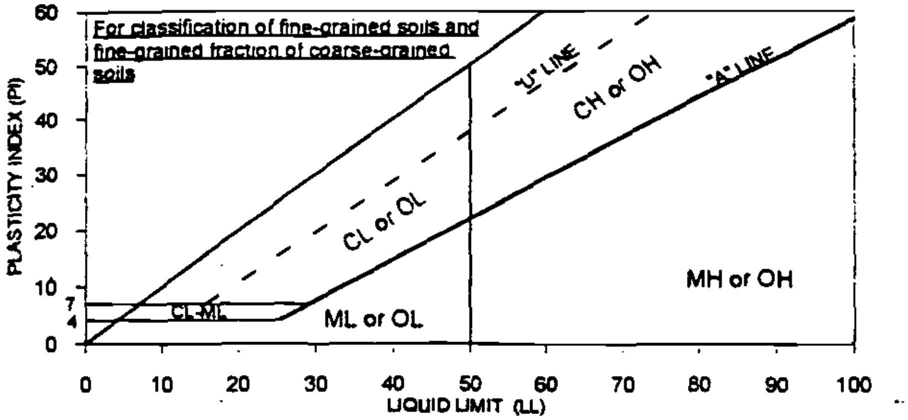
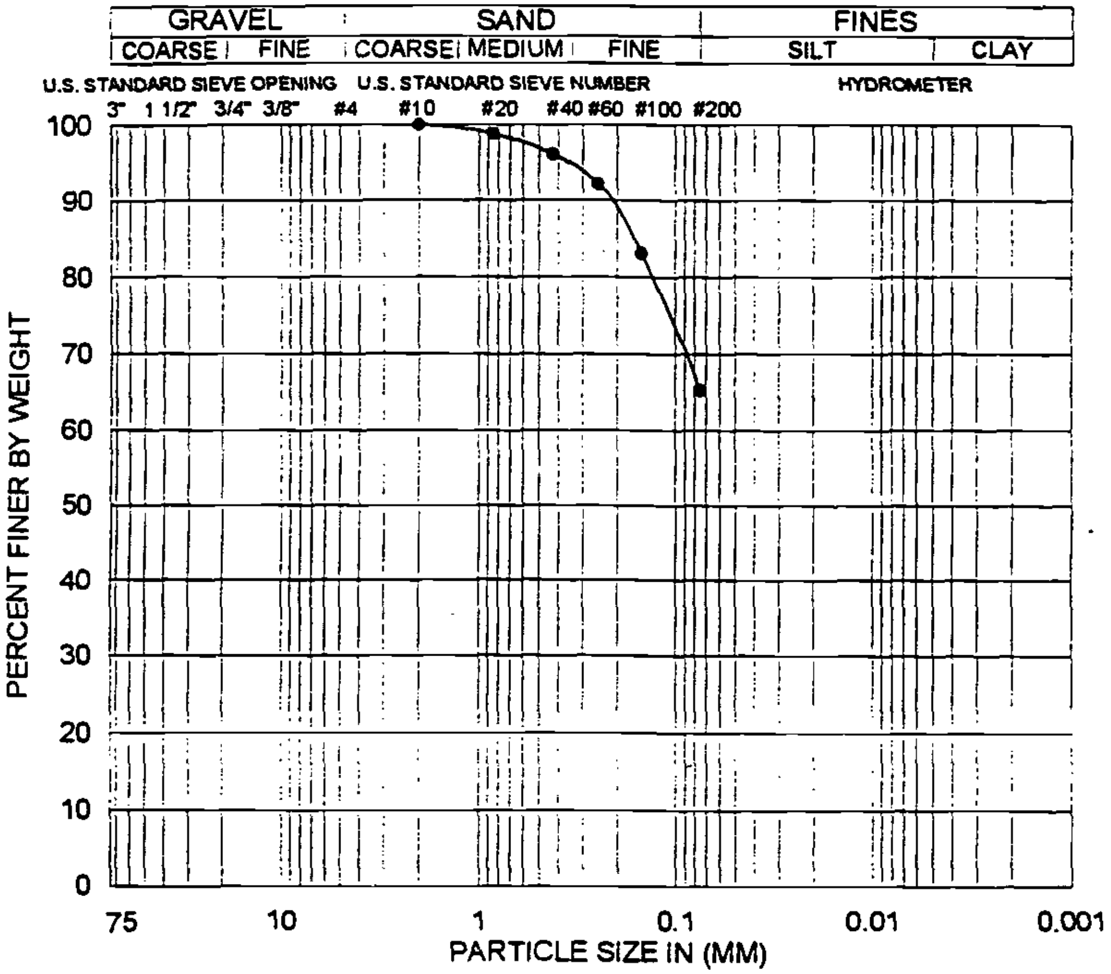
Figure B-34





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-21	73.0	Bag	ML	00:35:65	N/A

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Atterberg Limits, Particle-Size Curve
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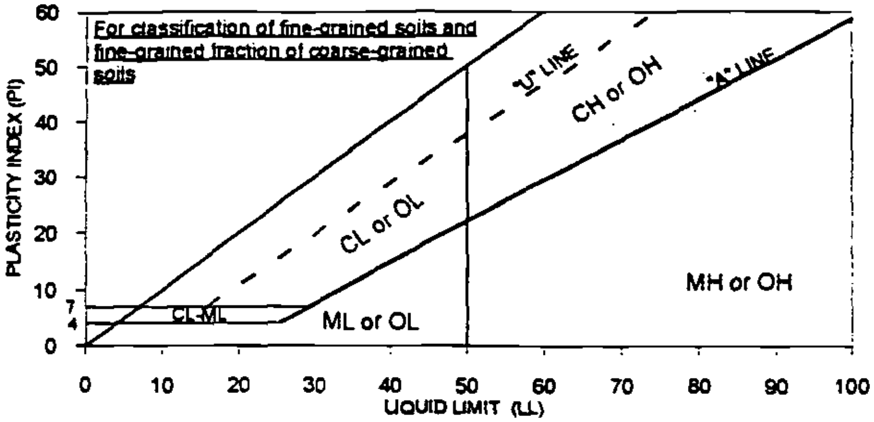
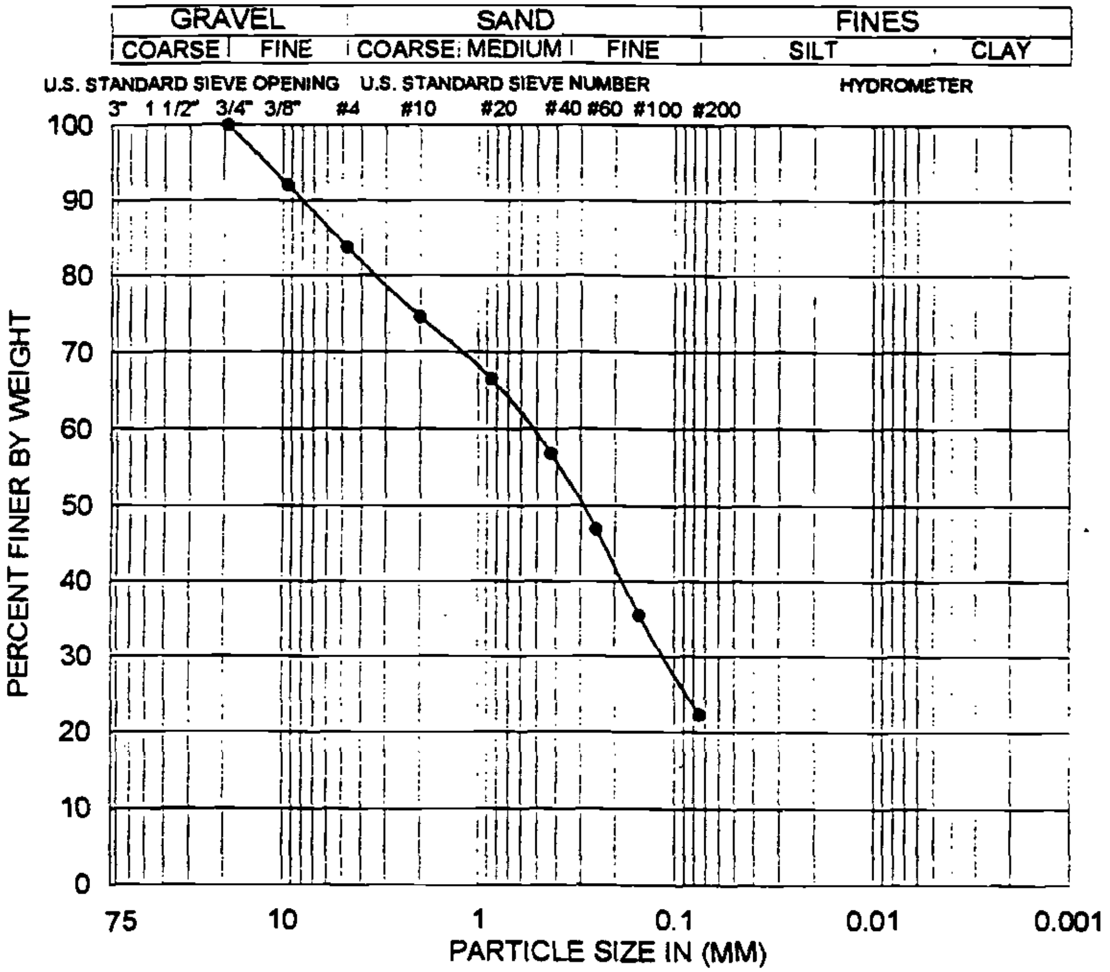
Figure B-35





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-25	86.0	Bag	SM	16:62:22	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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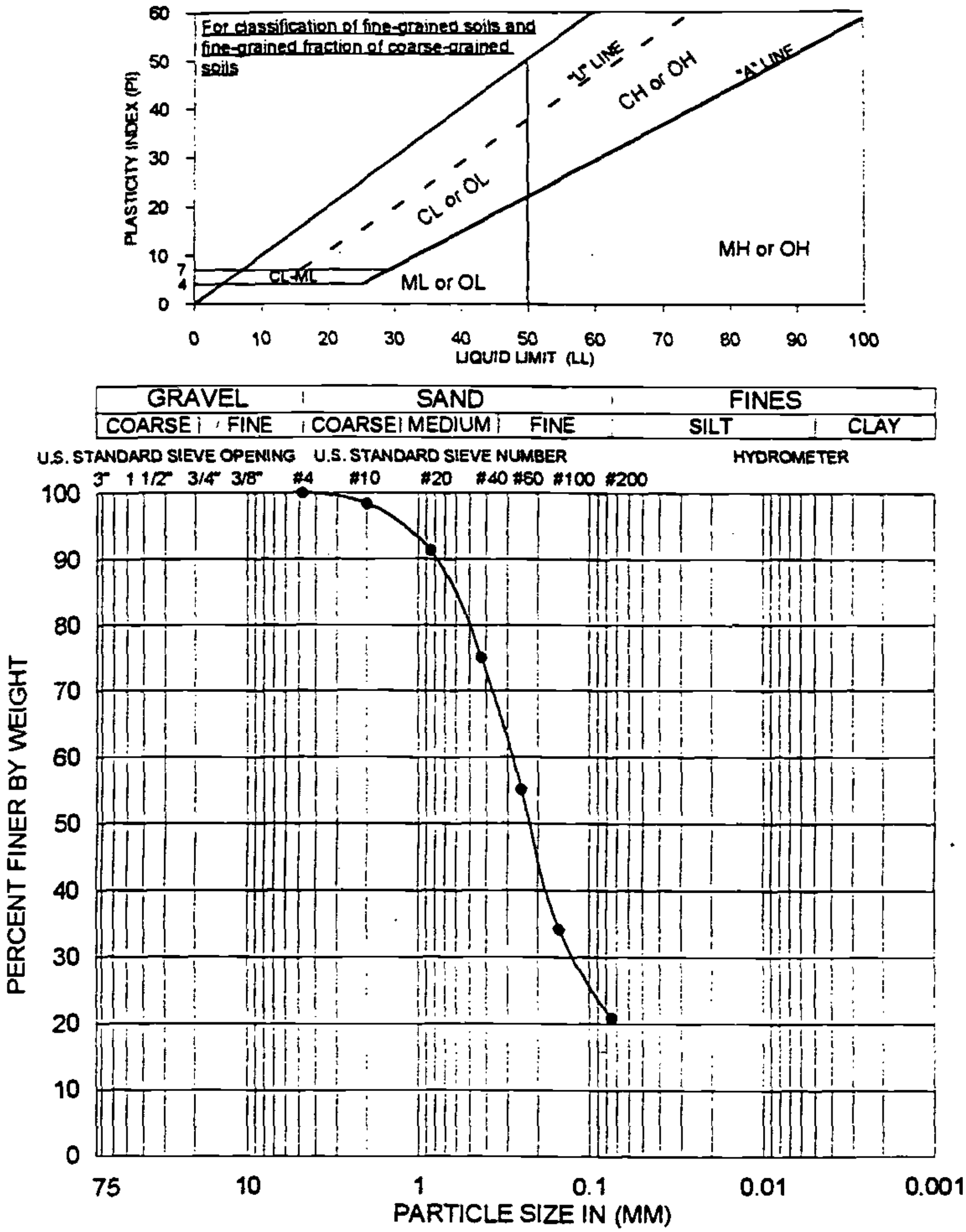
Figure B-36



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-27	92.5	Bag	SM	00:79:21	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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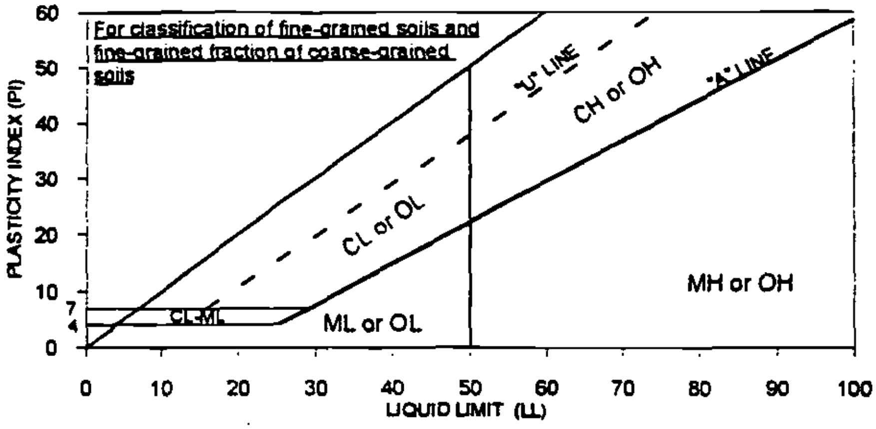
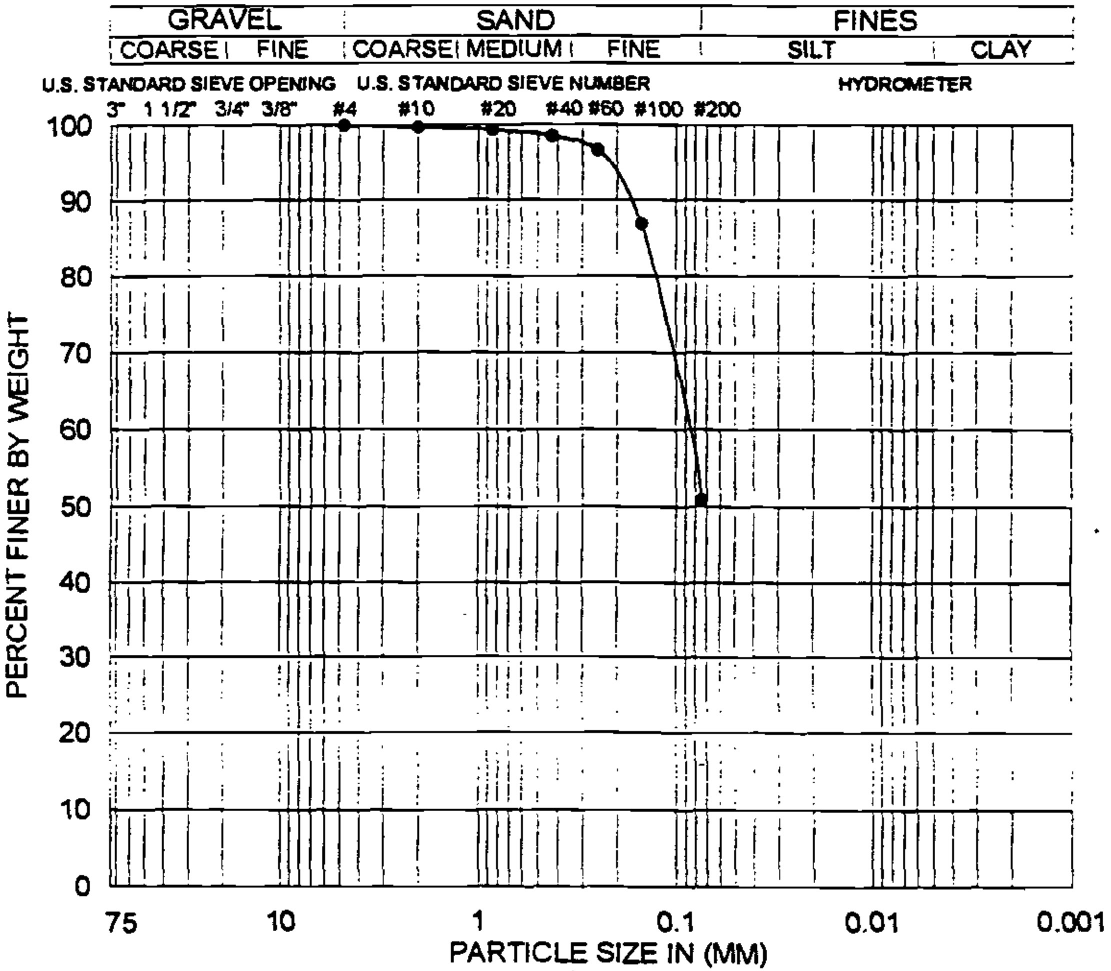
Figure B-37





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-30	100.5	Bag	ML	00:49:51	N/A

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Atterberg Limits, Particle-Size Curve
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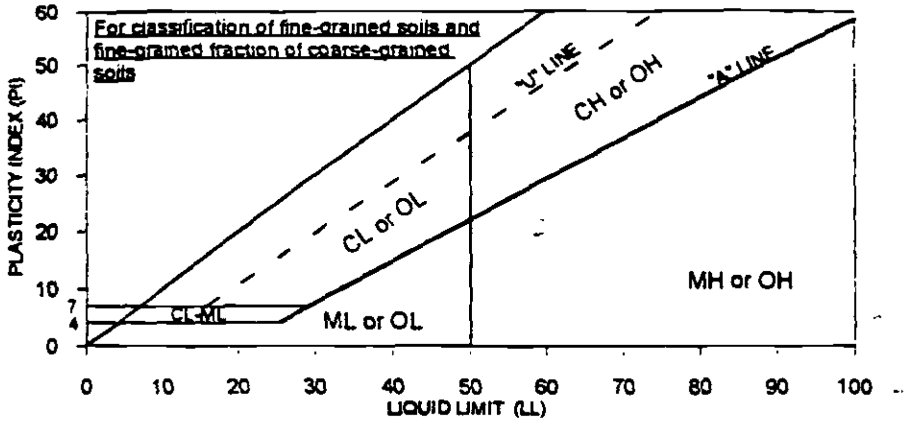
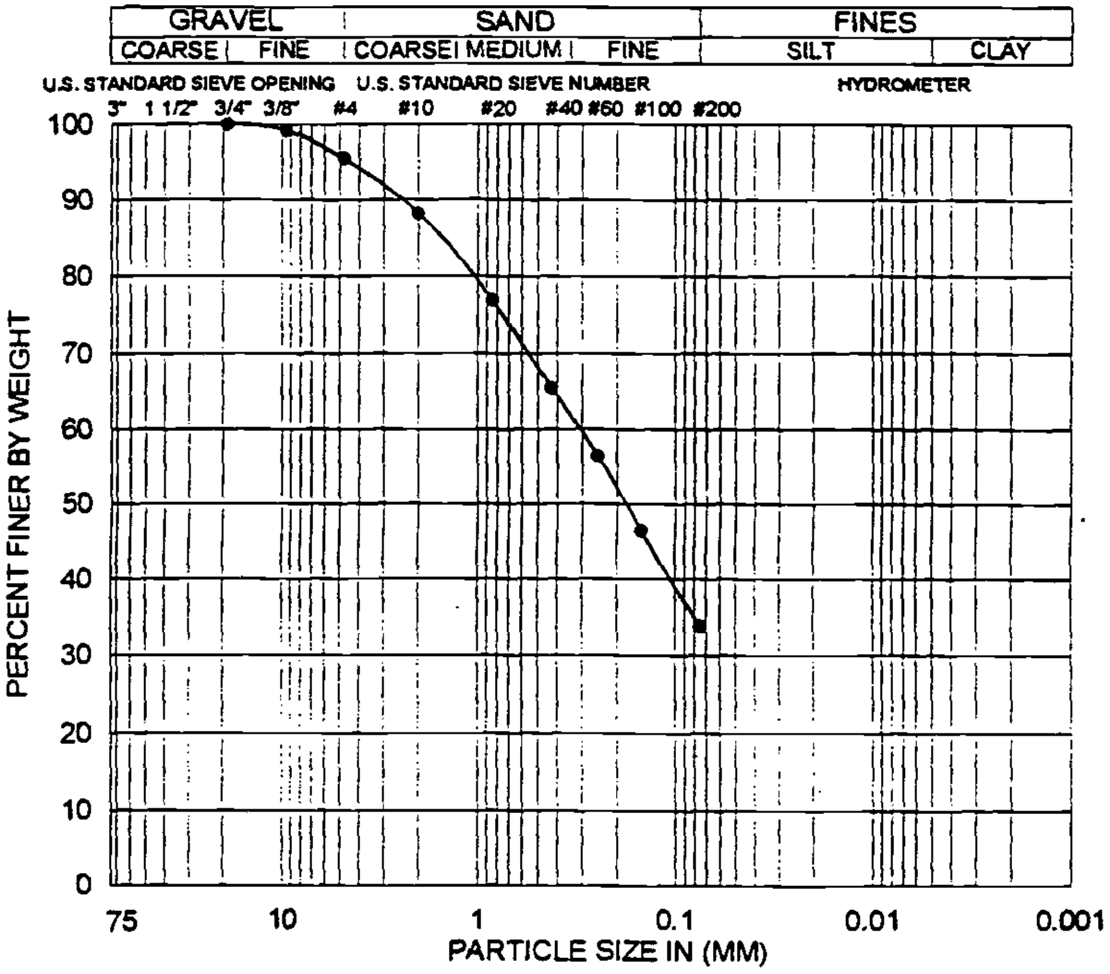
Figure B-38





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	S-33	111.5	Bag	SC	05:61:34	N/A

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Atterberg Limits, Particle-Size Curve
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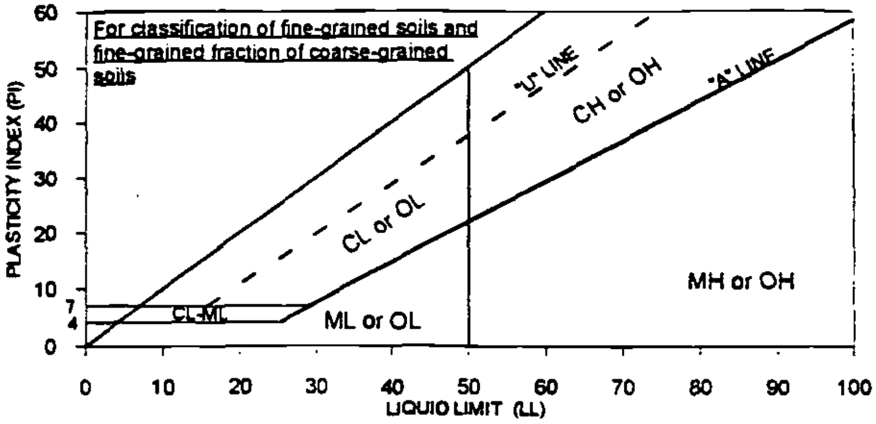
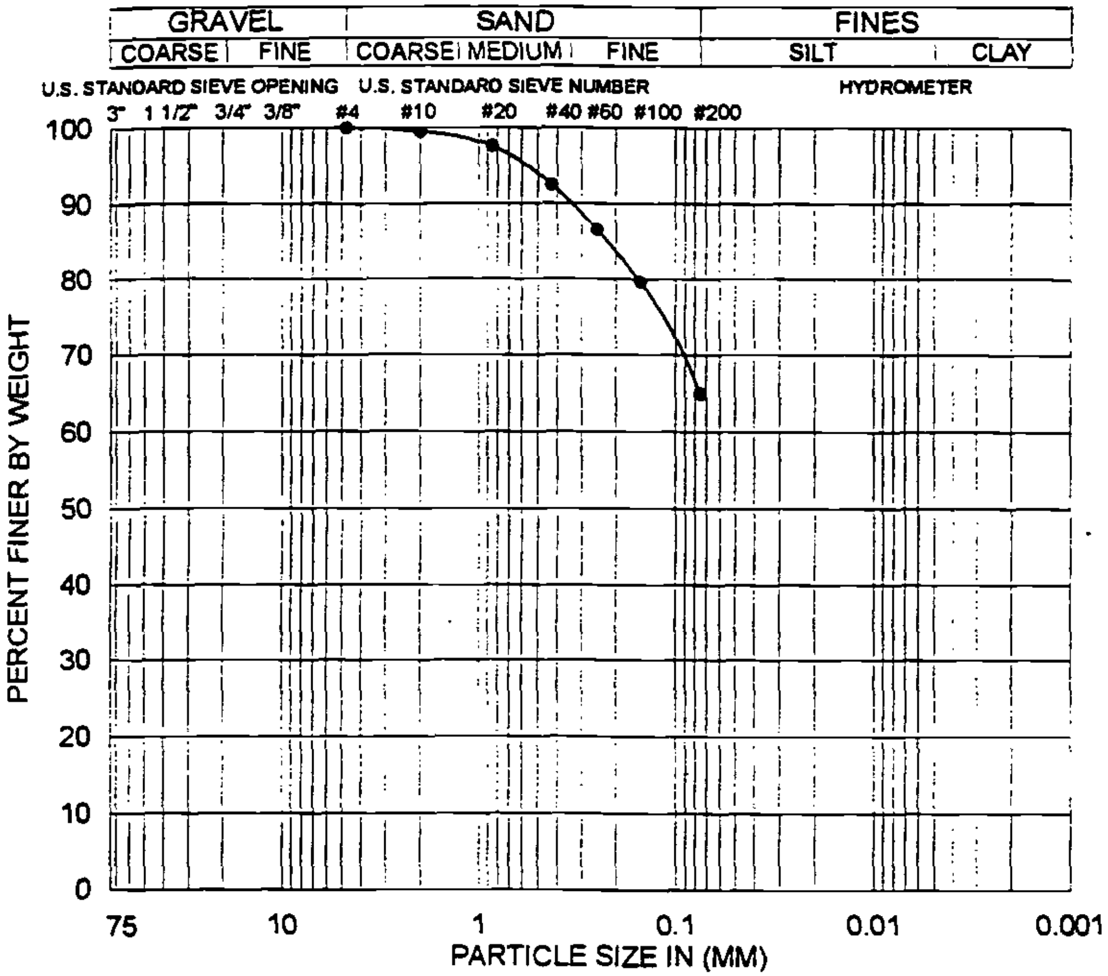
Figure B-39





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-27S	D-36	124.0	Drive	CL	00:35:65	N/A

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Atterberg Limits, Particle-Size Curve
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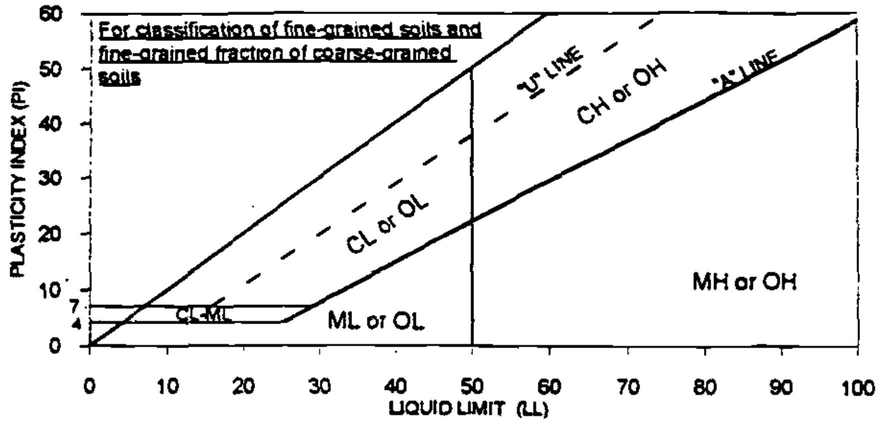
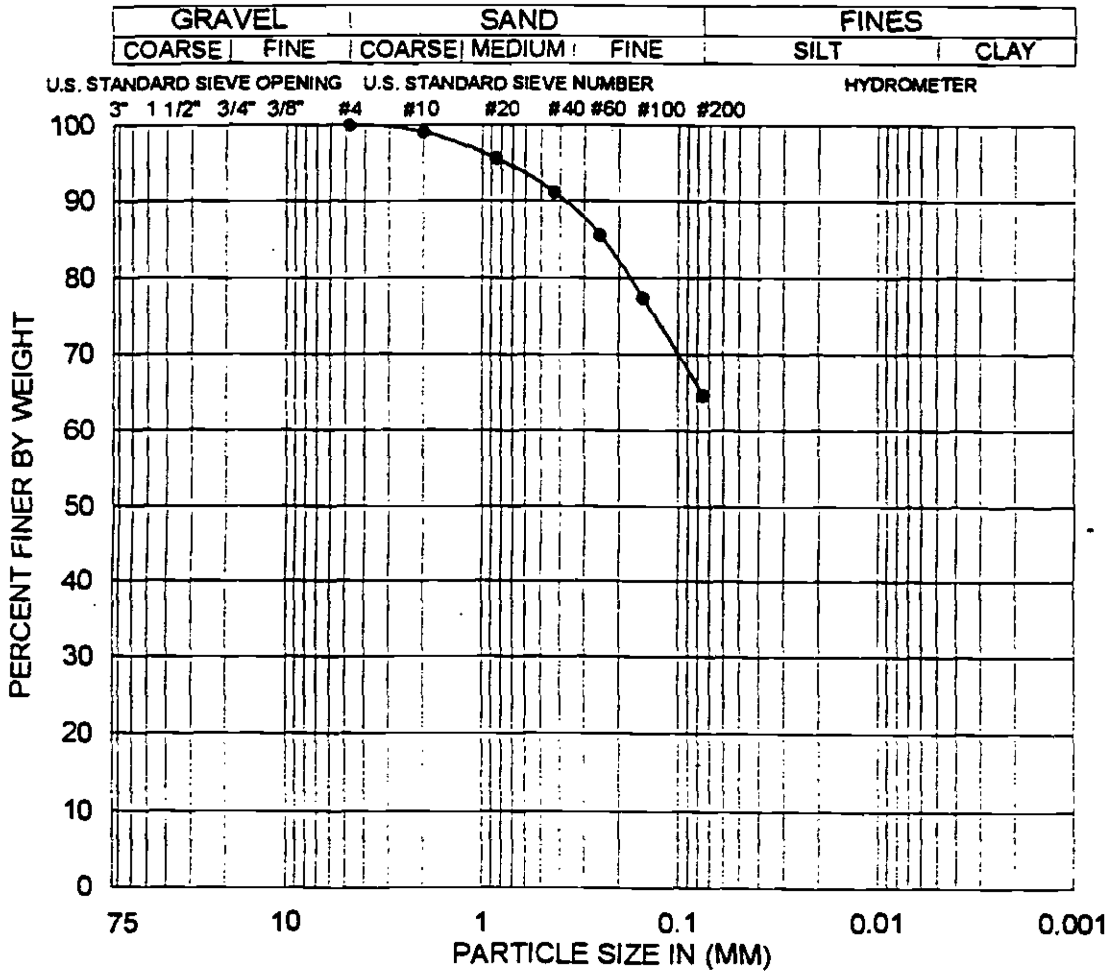
Figure B-40





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-8	44.0	Bag	CL	00:35:65	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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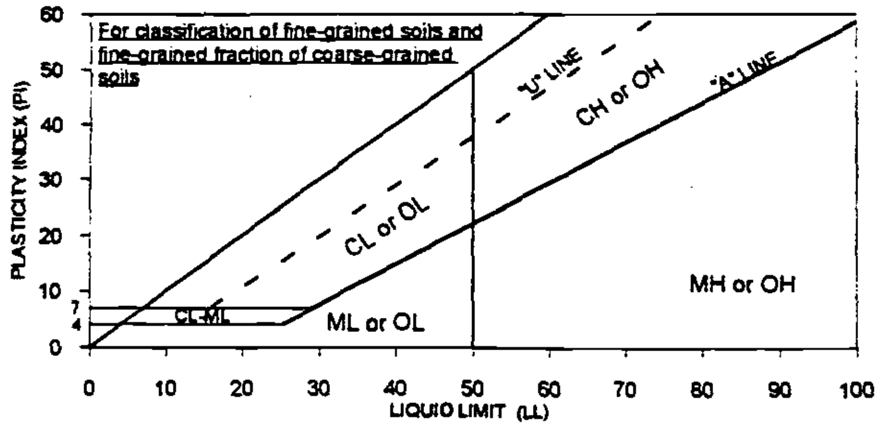
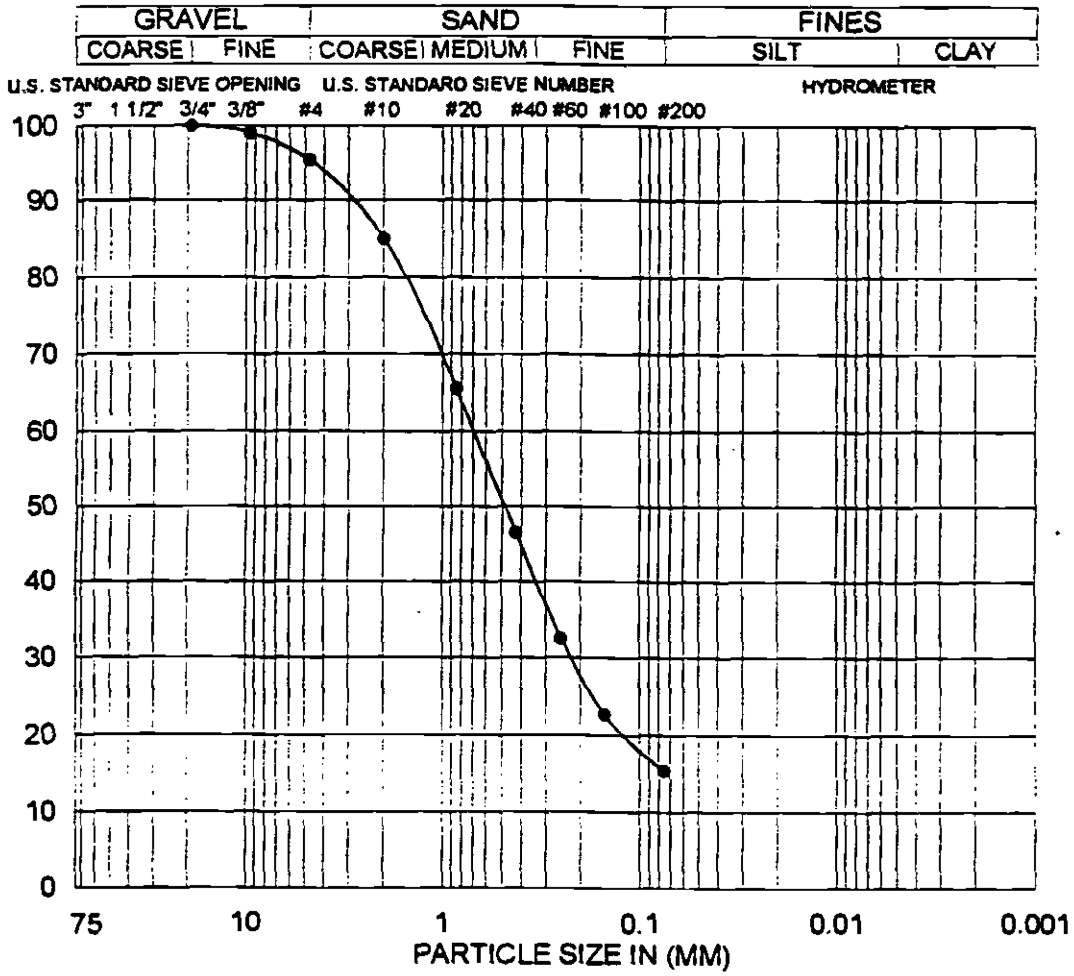
Figure B-41





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-10	53.5	Bag	SC	05:80:15	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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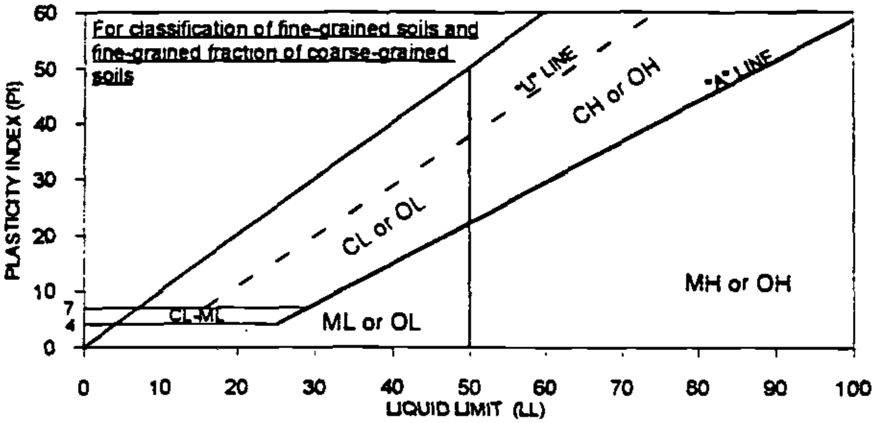
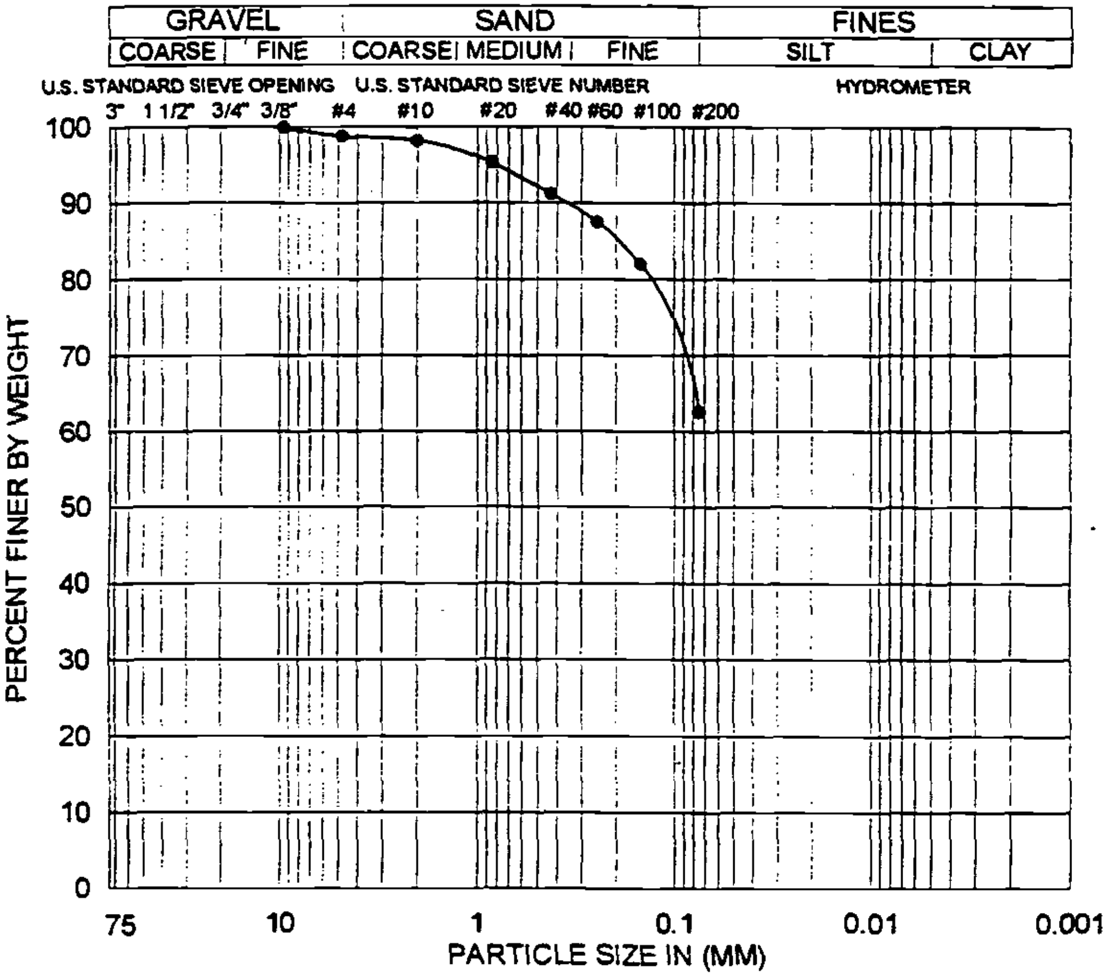
Figure B-42





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-12	63.5	Bag	CL	01:36:63	N/A

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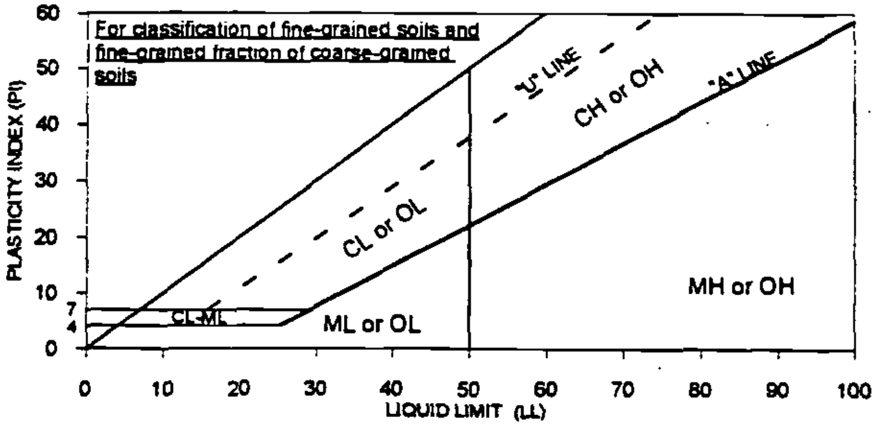
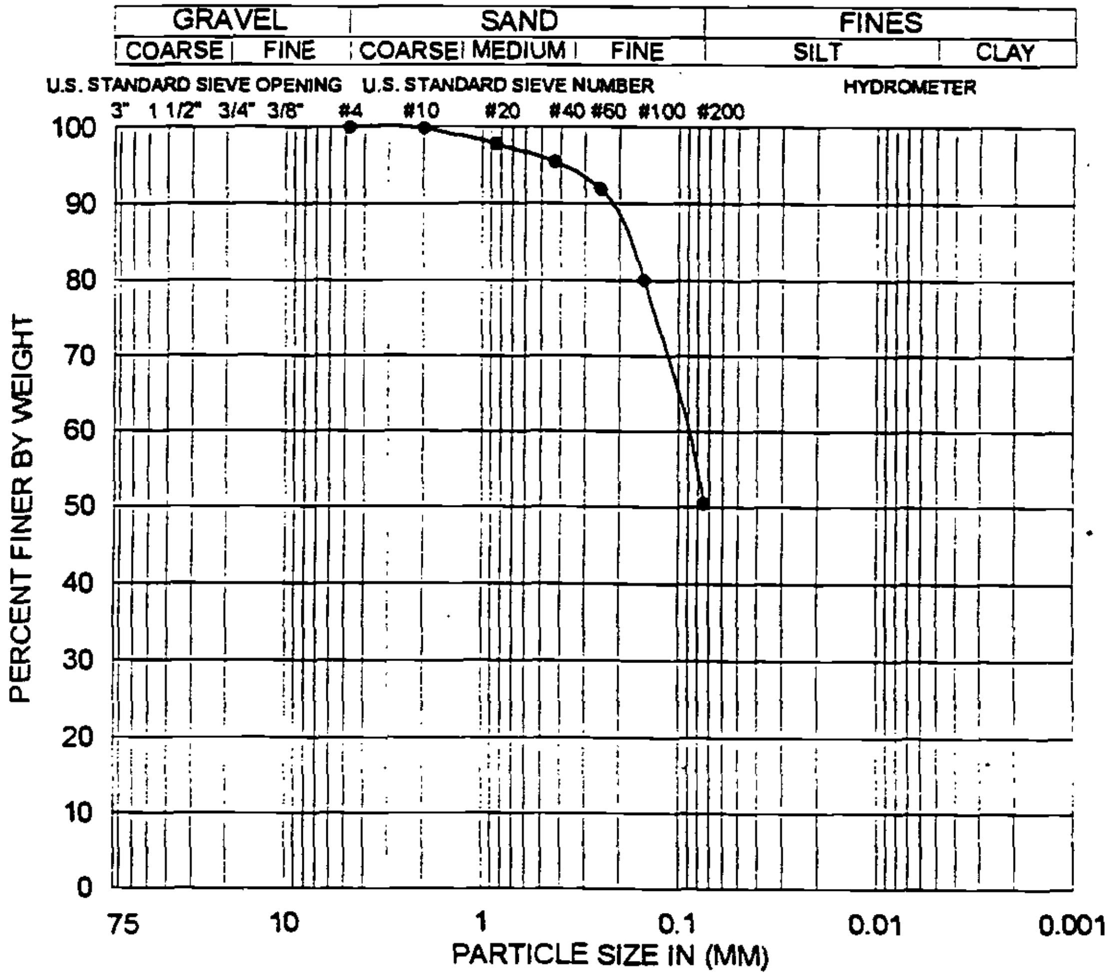
Figure B-43





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-14	70.0	Bag	ML	00:49:51	N/A

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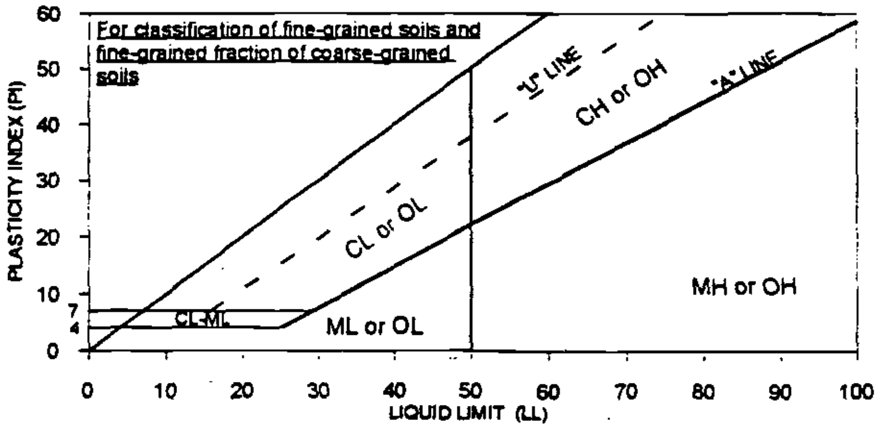
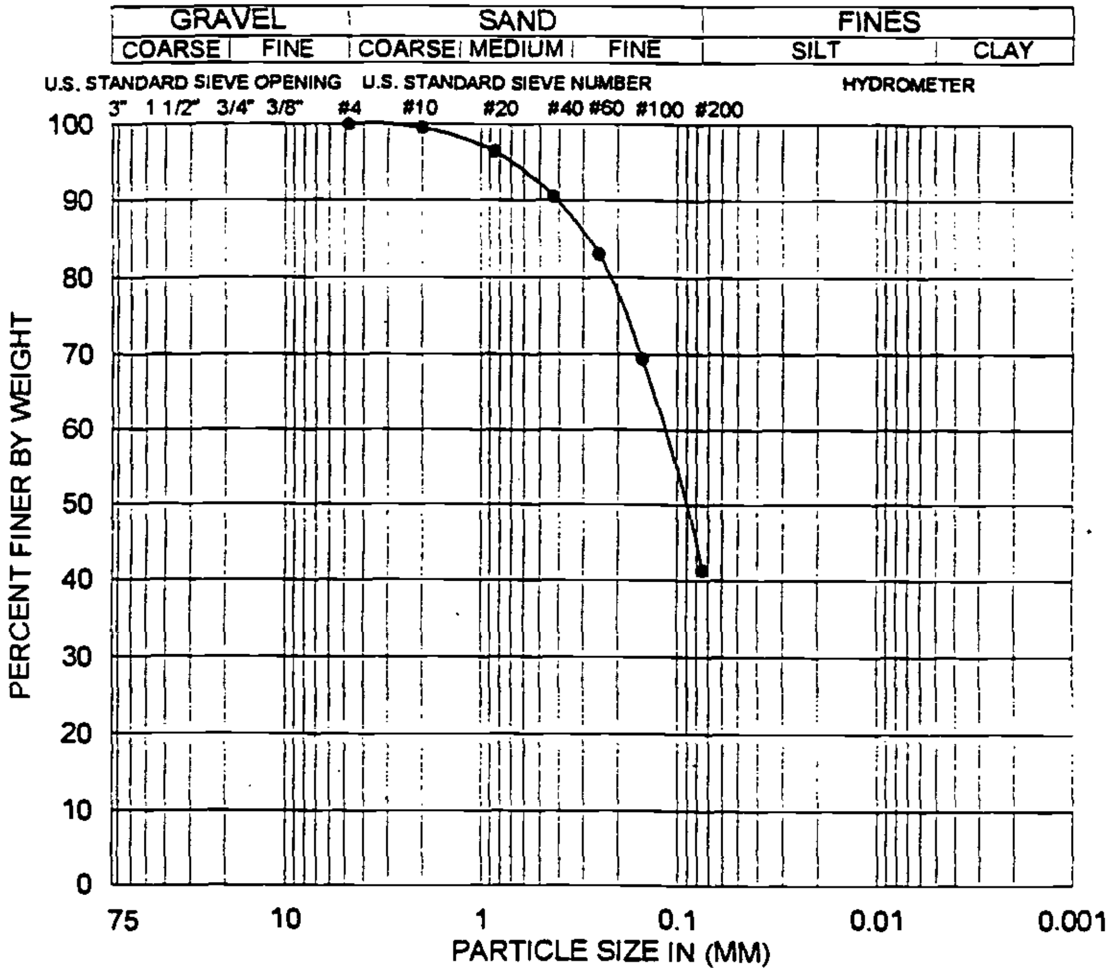
Metro Redline II

Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

12-95

Figure B-44





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-18	82.0	Bag	SM	00:59:41	N/A

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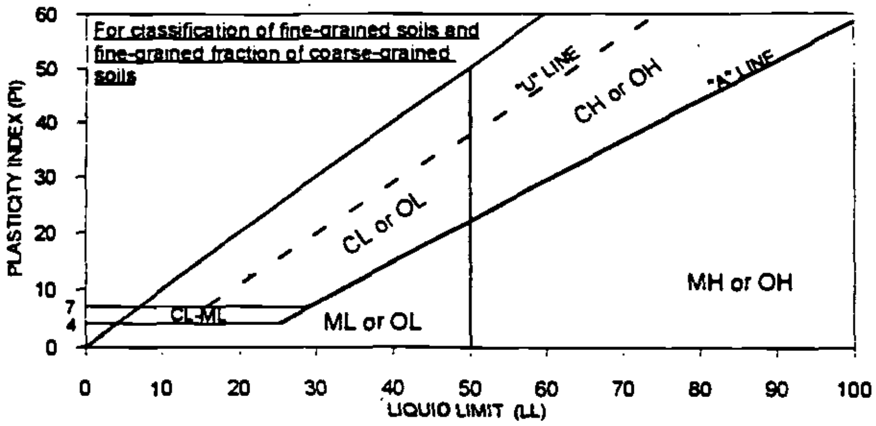
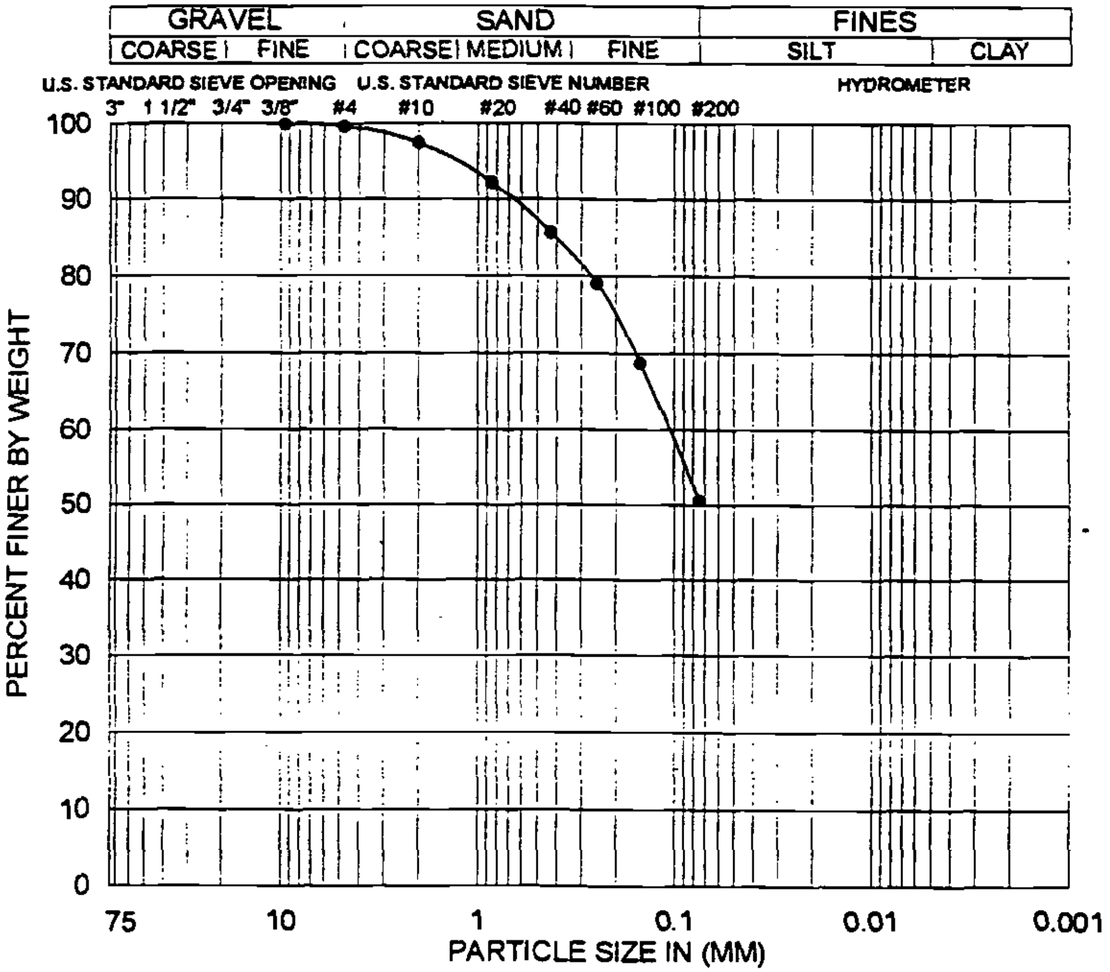
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-45





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-20	88.0	Bag	ML	00:49:51	N/A

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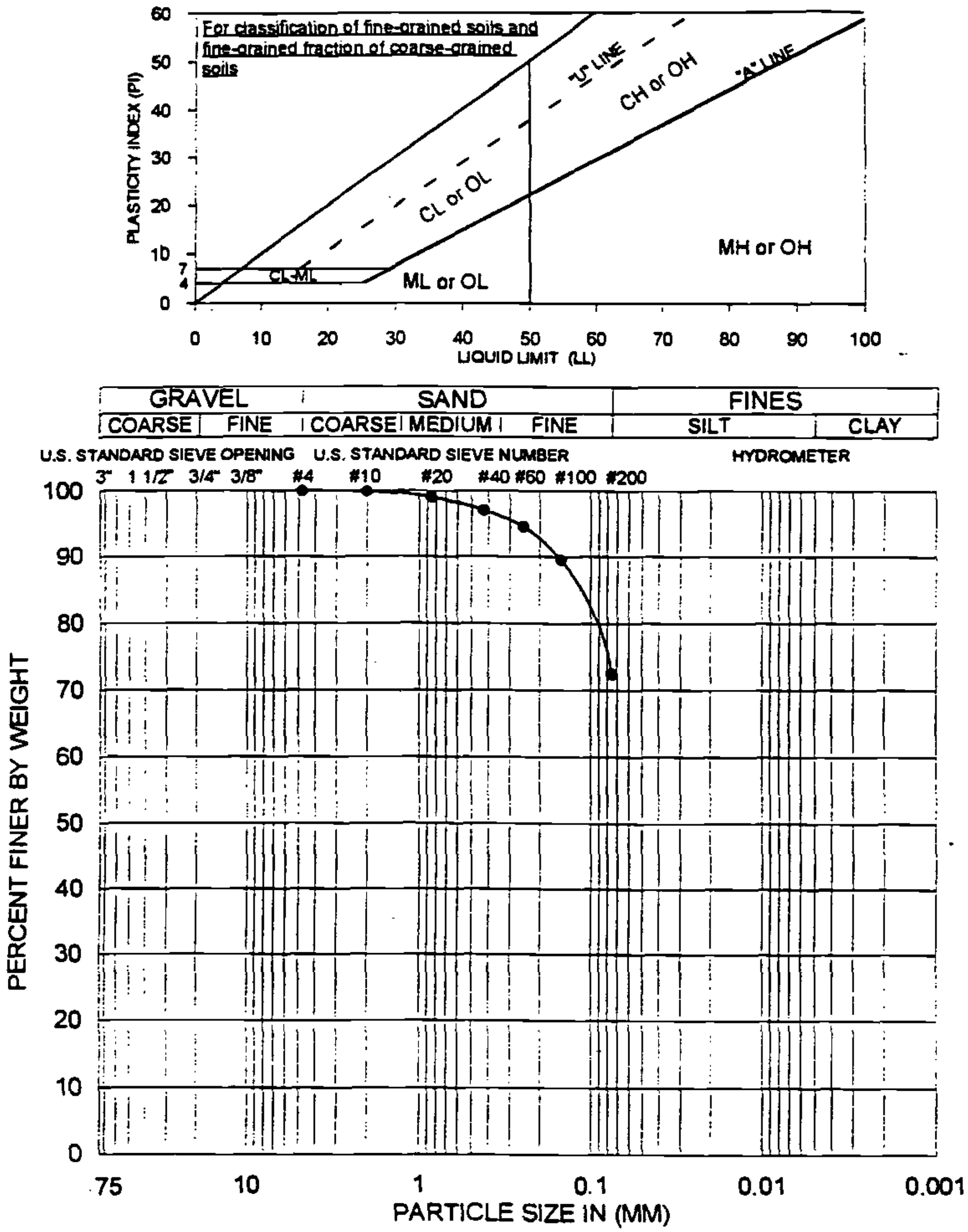
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-46



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-22	94.0	Bag	ML	00:27:73	N/A

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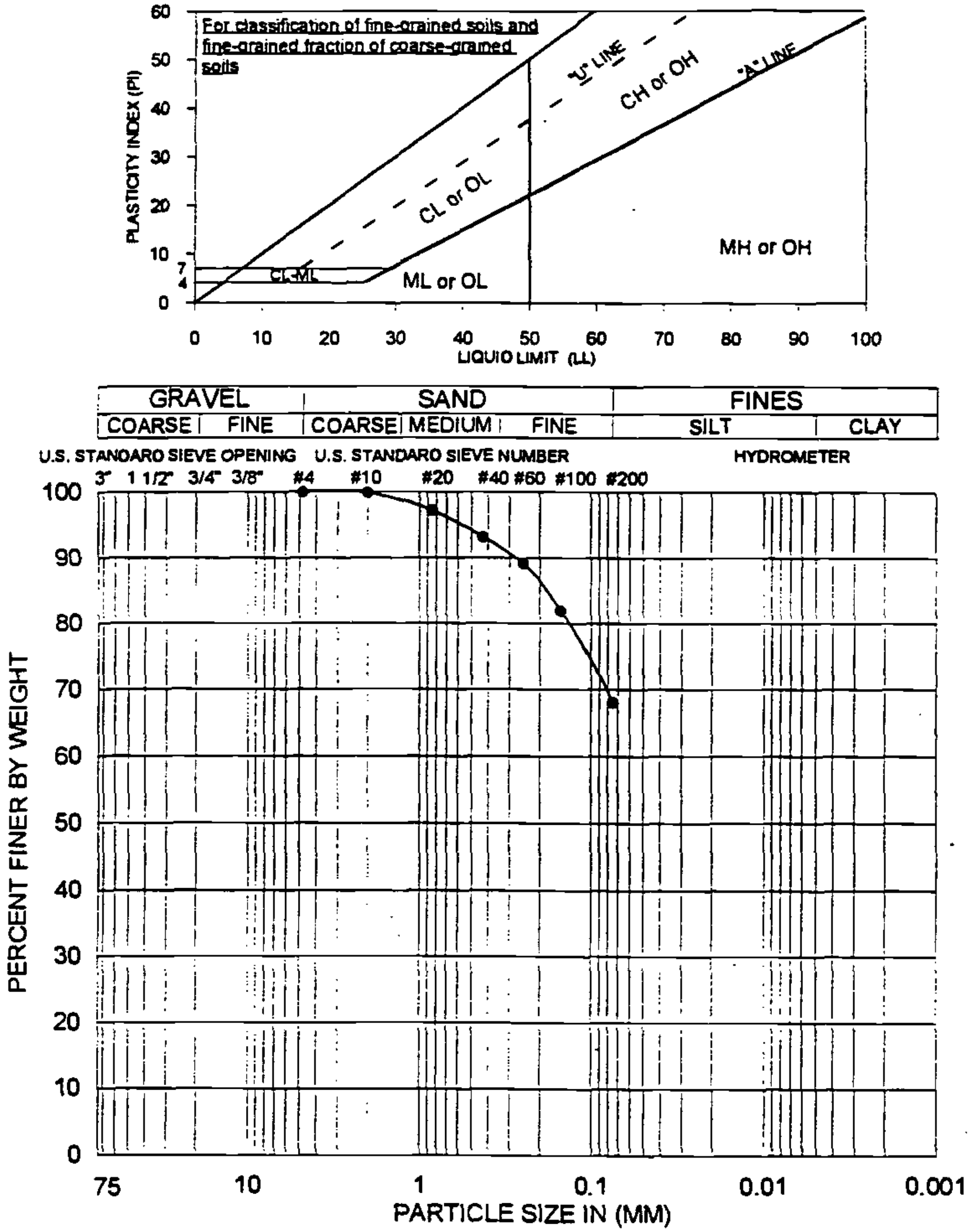
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-47



BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-24	100.0	Bag	CL	00:32:68	N/A

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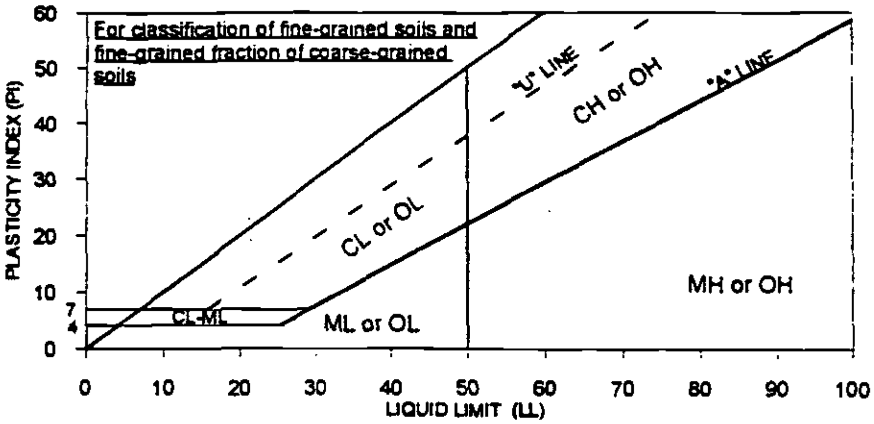
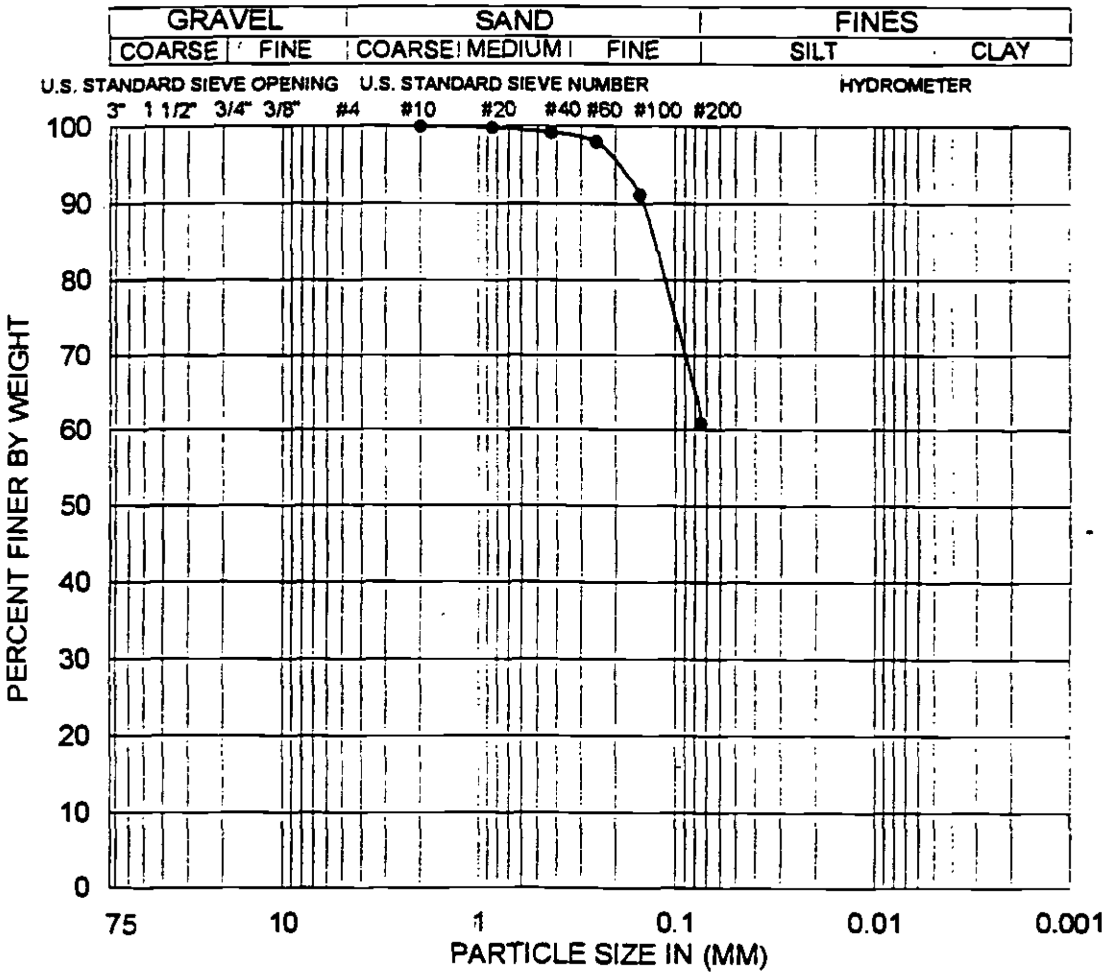
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-48





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-26	107.0	Bag	ML	00:39:61	N/A

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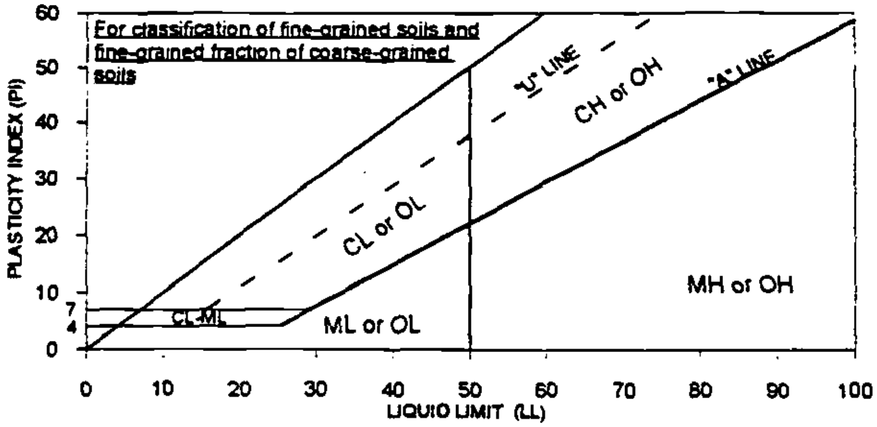
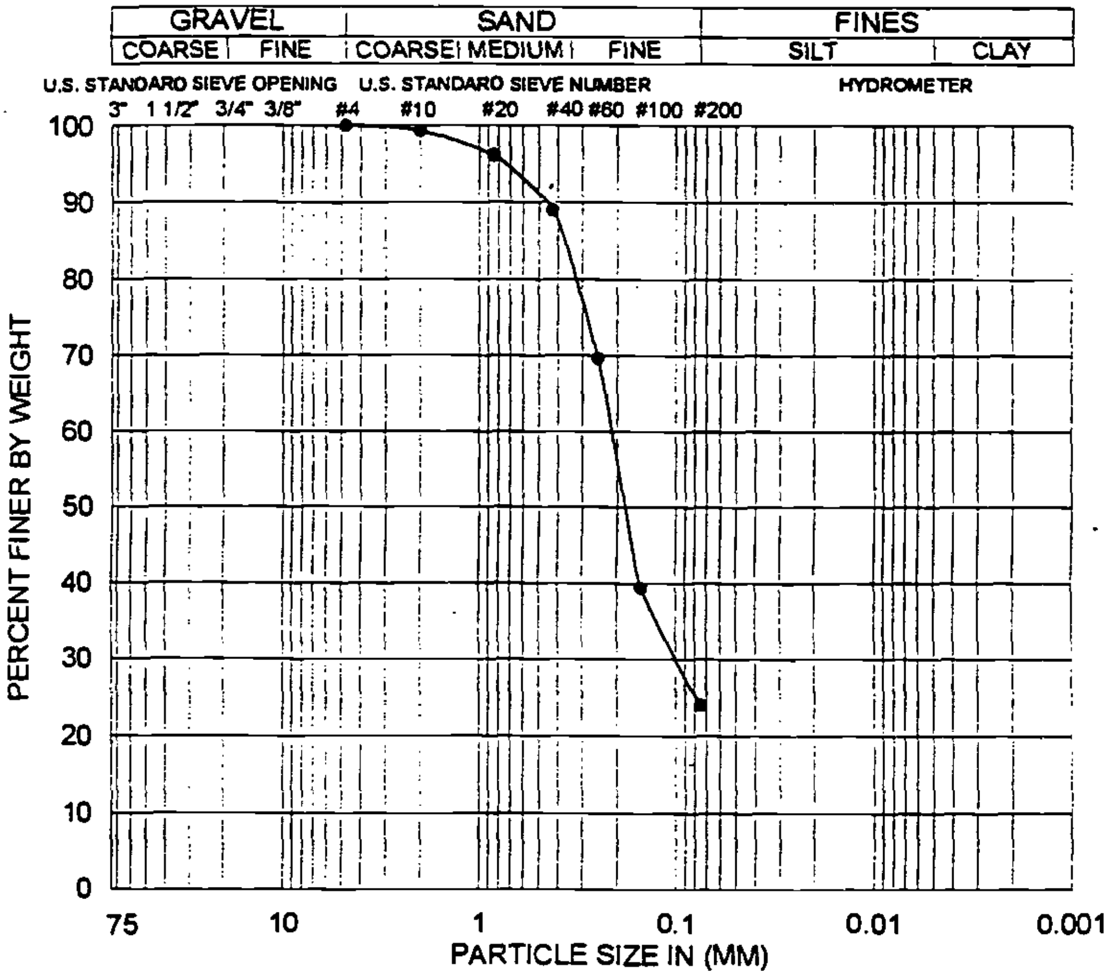
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-49





BORING NO.	SAMPLE NO.	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-28	S-28	115.0	Bag	SM	00:76:24	N/A

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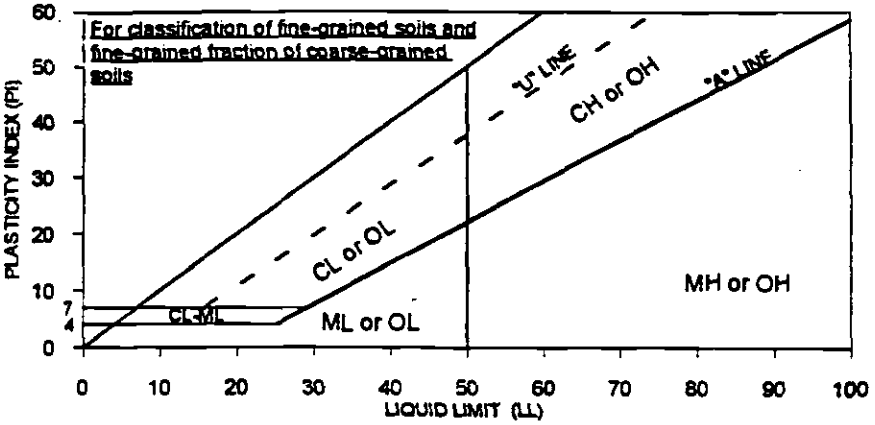
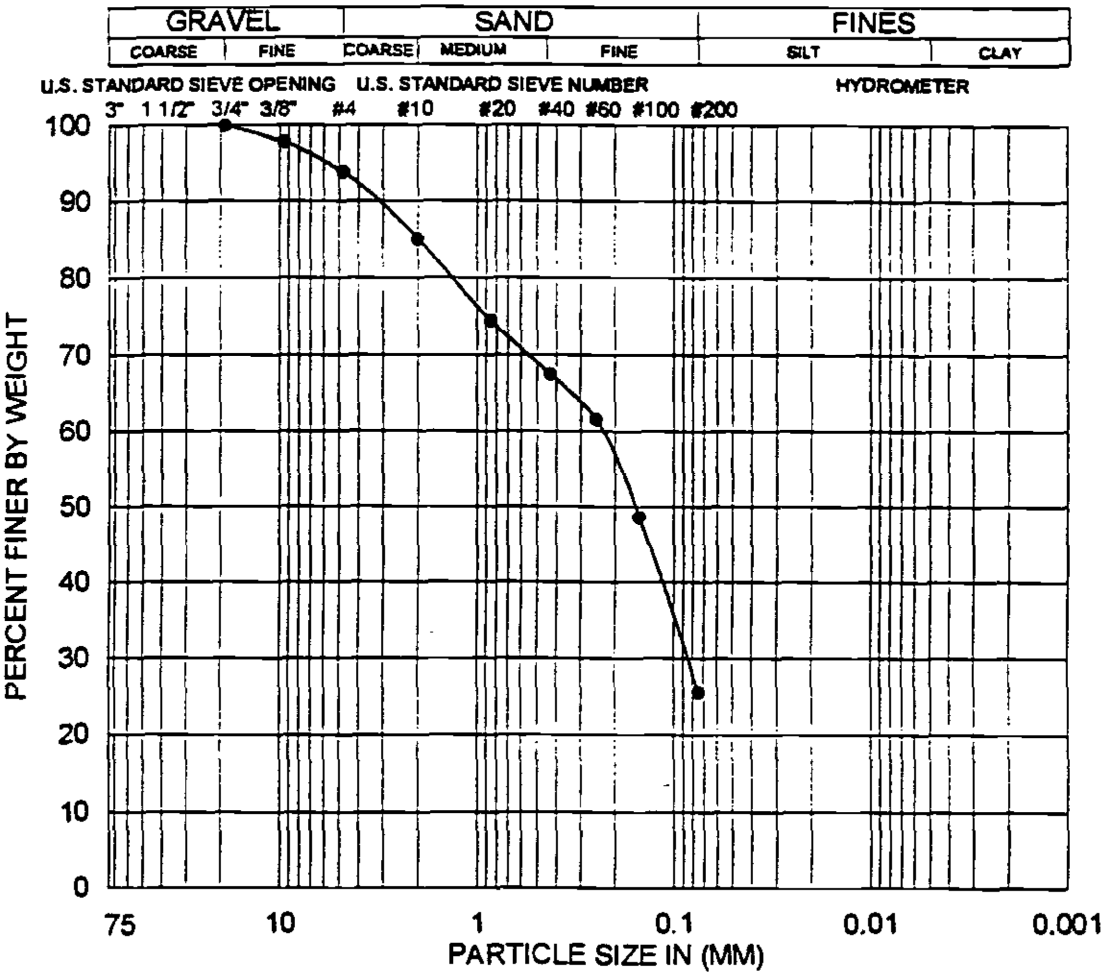
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-50





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-11	47.0	SM	6:68:26	N/A

Not enough sample to meet ASTM specification

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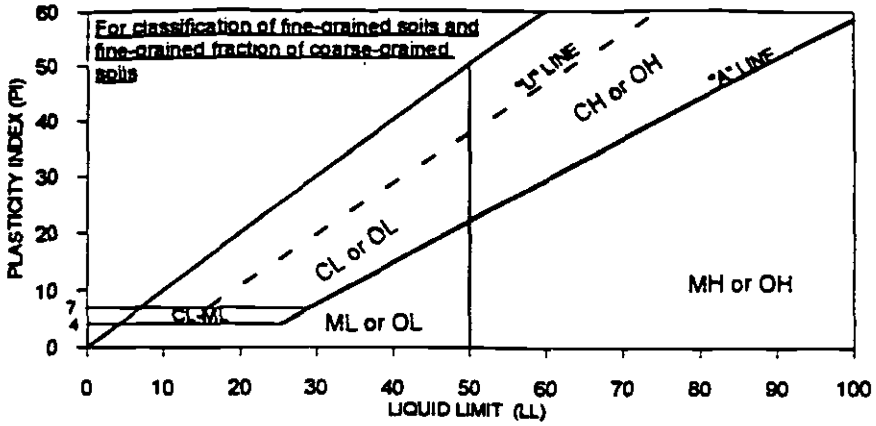
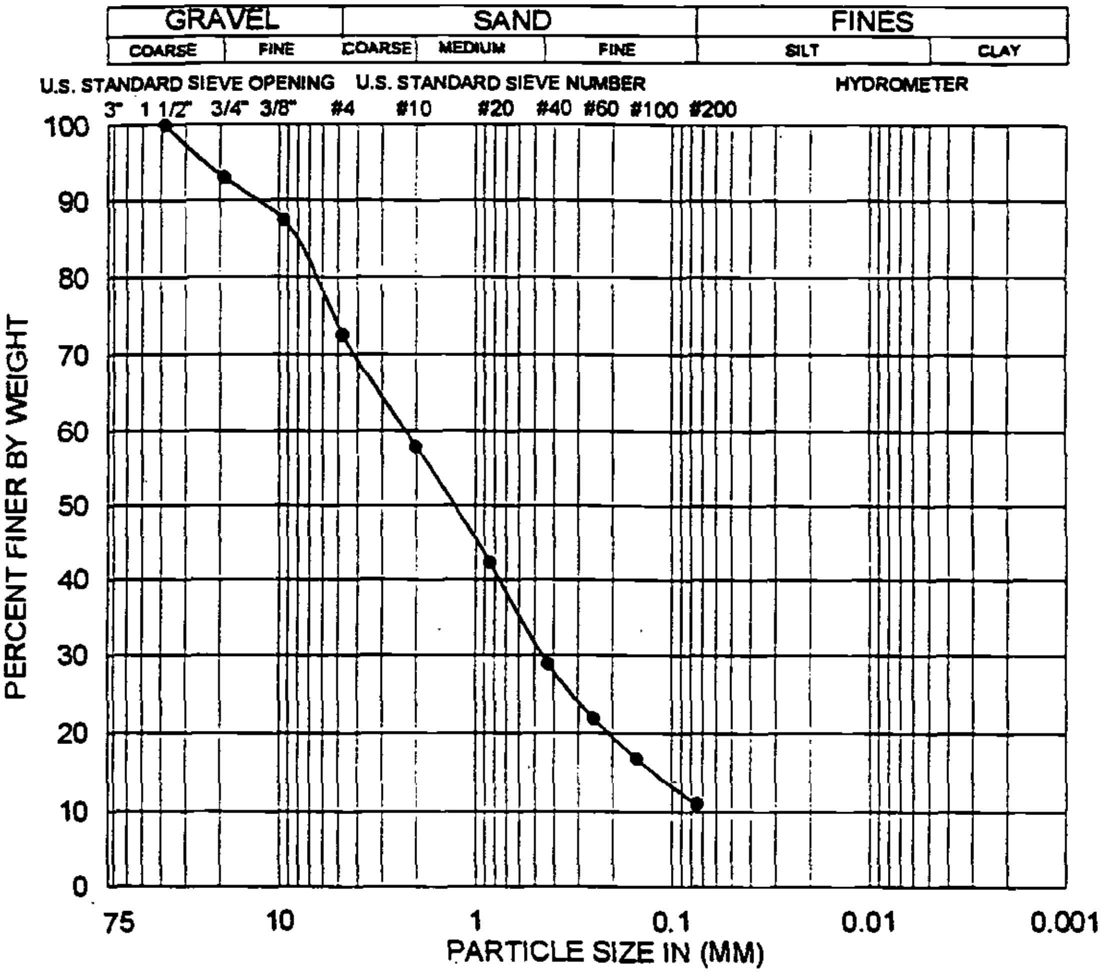
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-51





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-13	53.0	SP-SM	27:62:11	N/A

Not enough sample to meet ASTM specification

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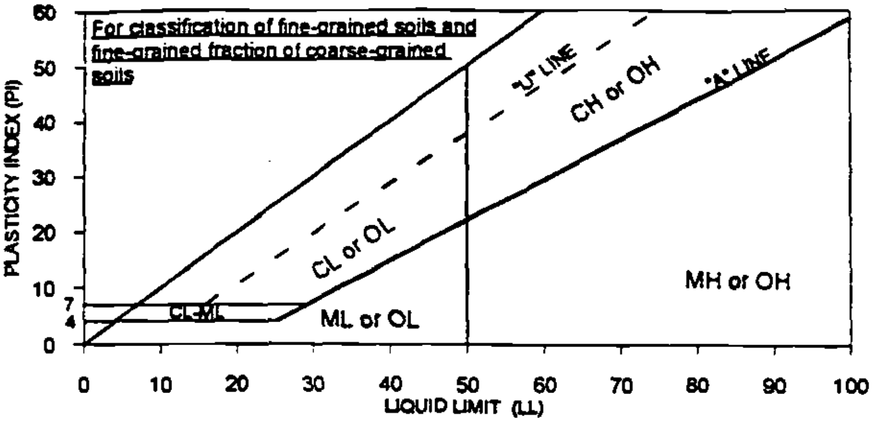
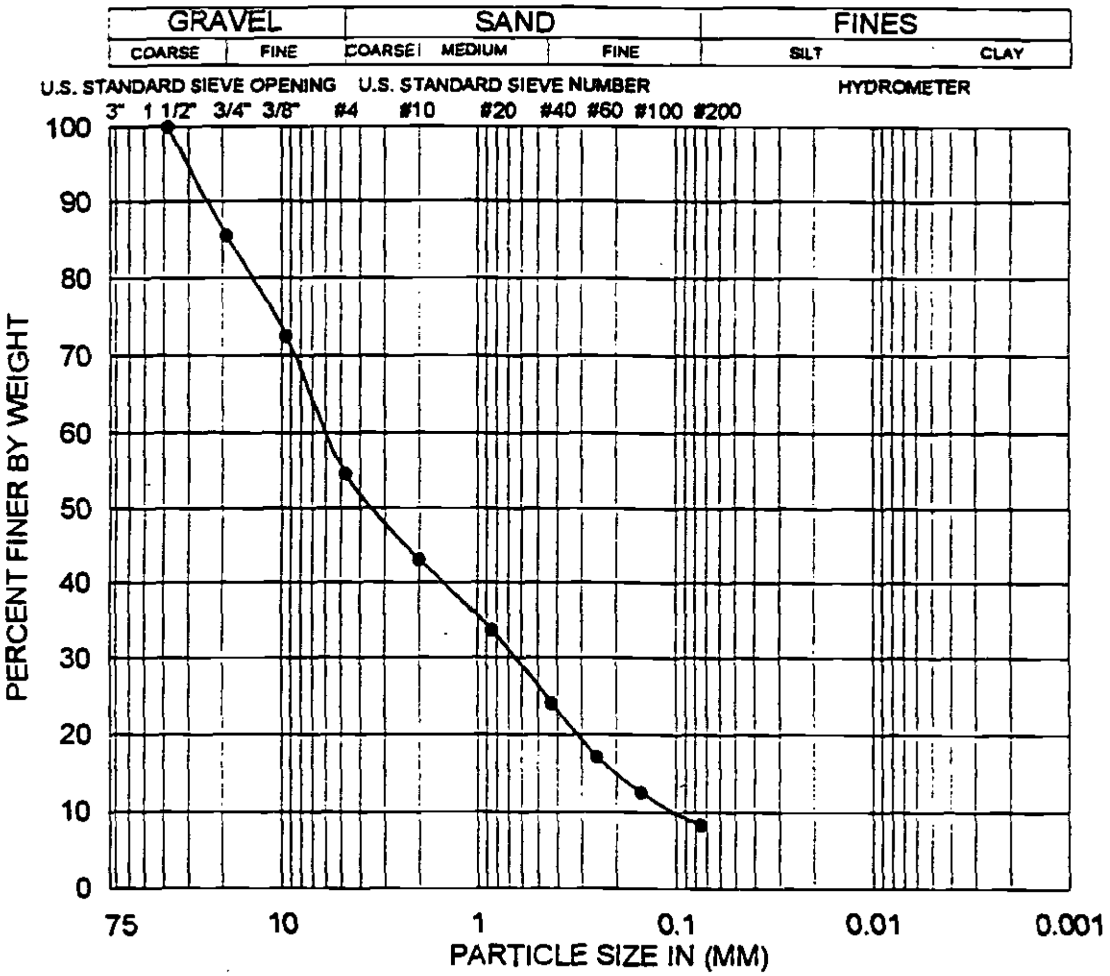
Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

12-95

Figure B-52





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-15	59.0	SP-SM	45:47:8	N/A

Not enough sample to meet ASTM specification

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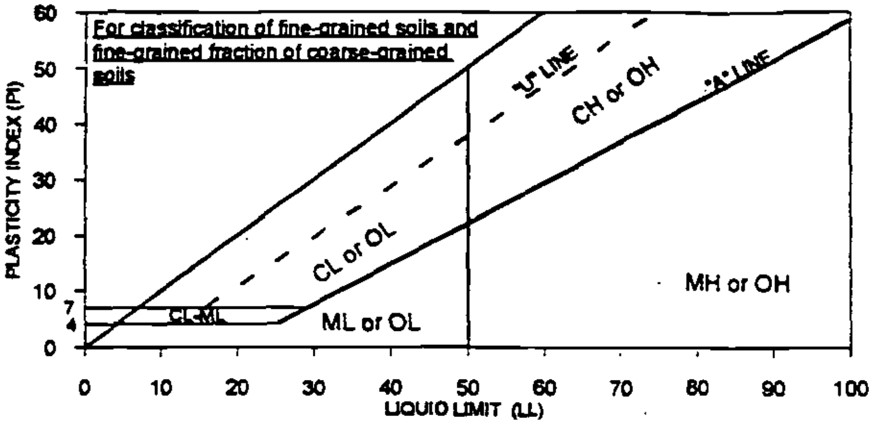
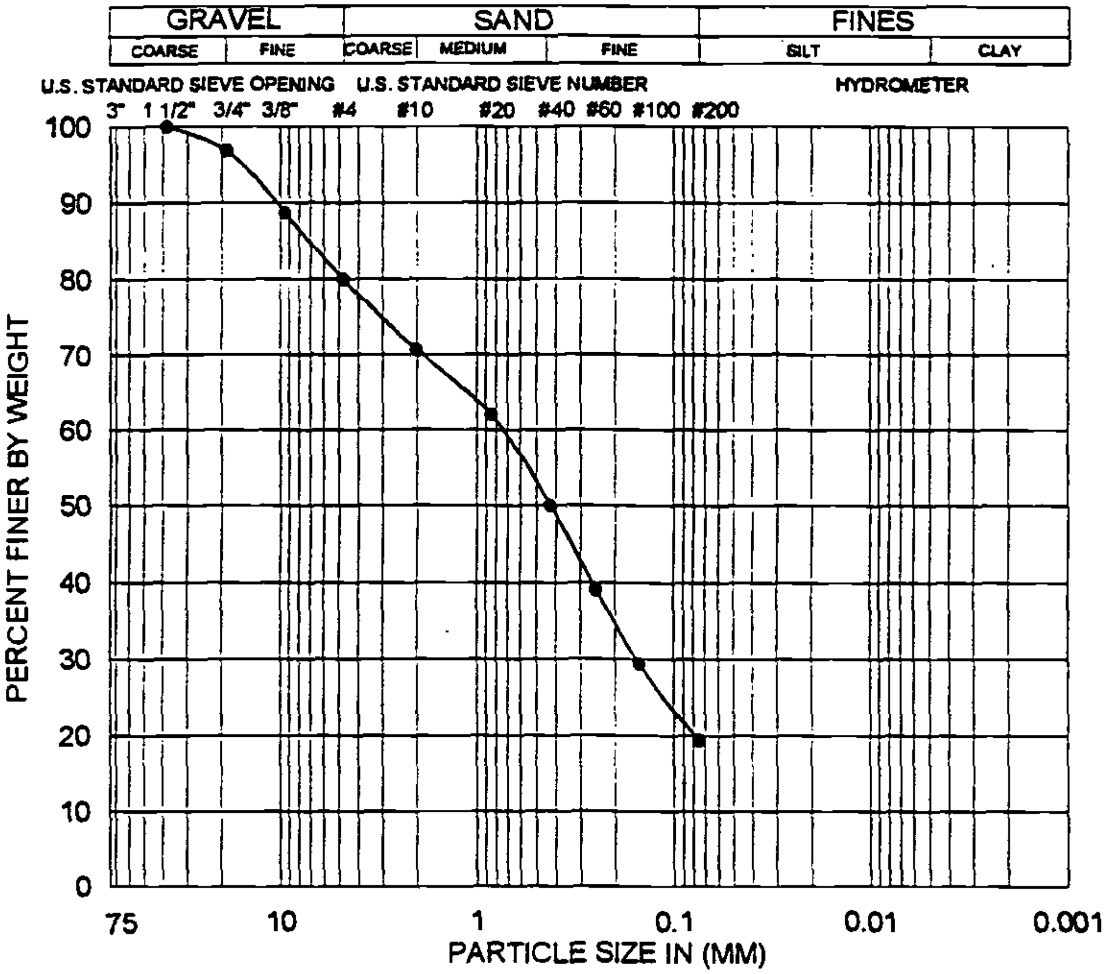
Atterberg Limits, Particle-Size Curve
(ASTM D4318, D422)

12-95

Figure B-53





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-17	65.0	SM	20:61:19	N/A

Not enough sample to meet ASTM specification

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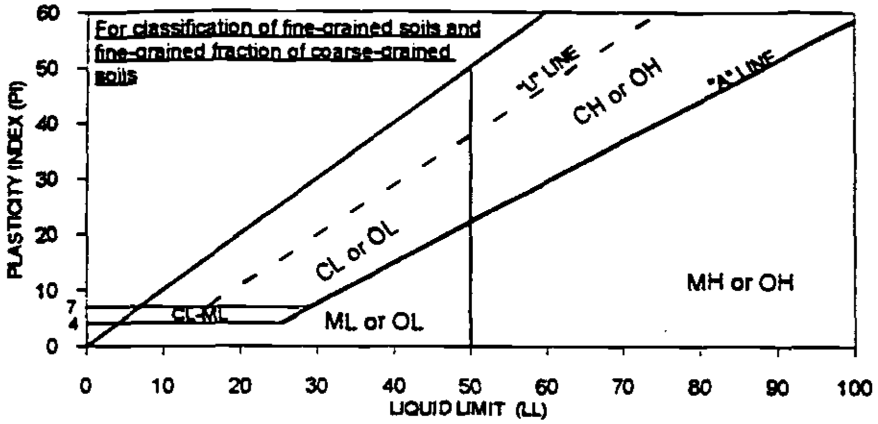
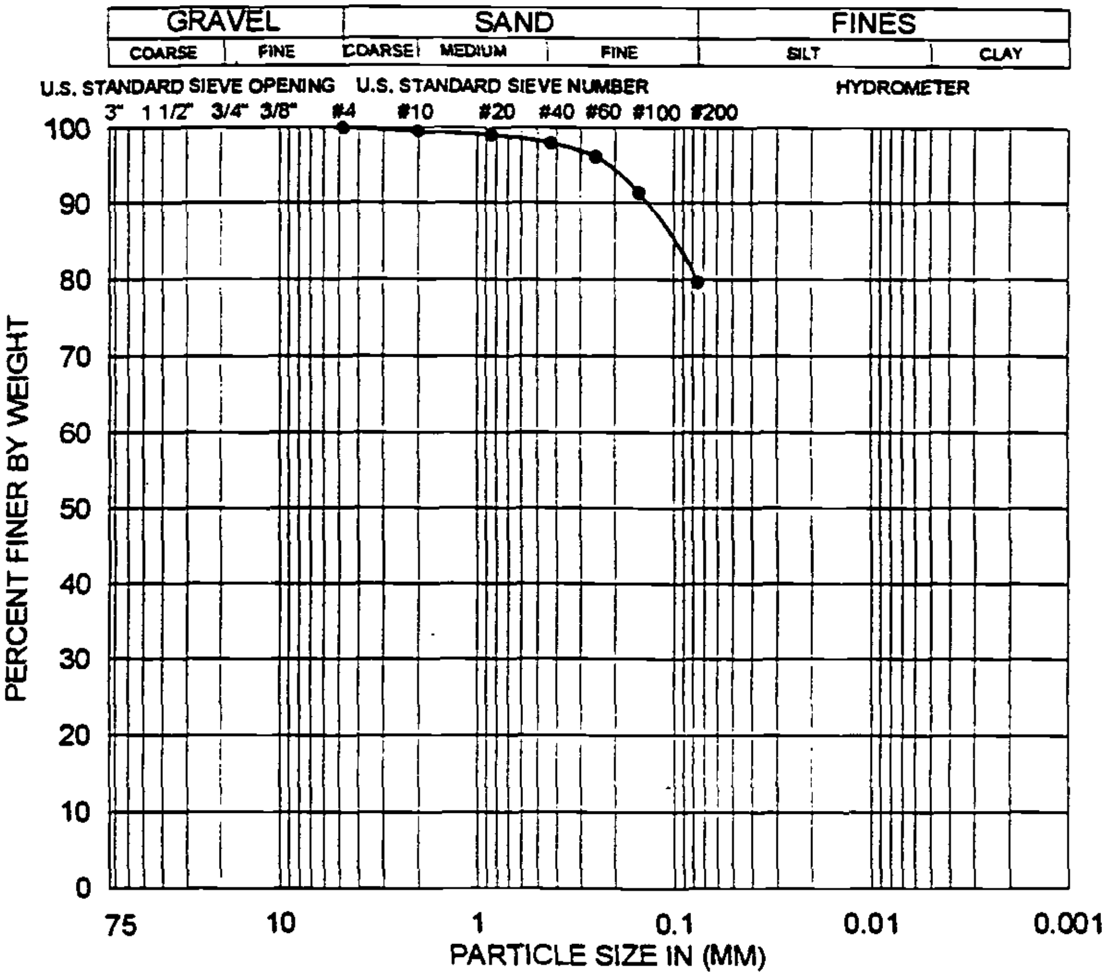
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-54





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-19	71.0	ML	0:20:80	N/A

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Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

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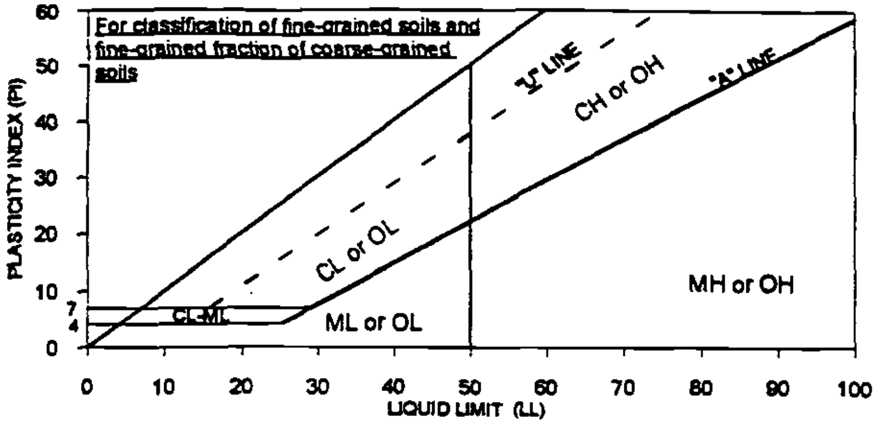
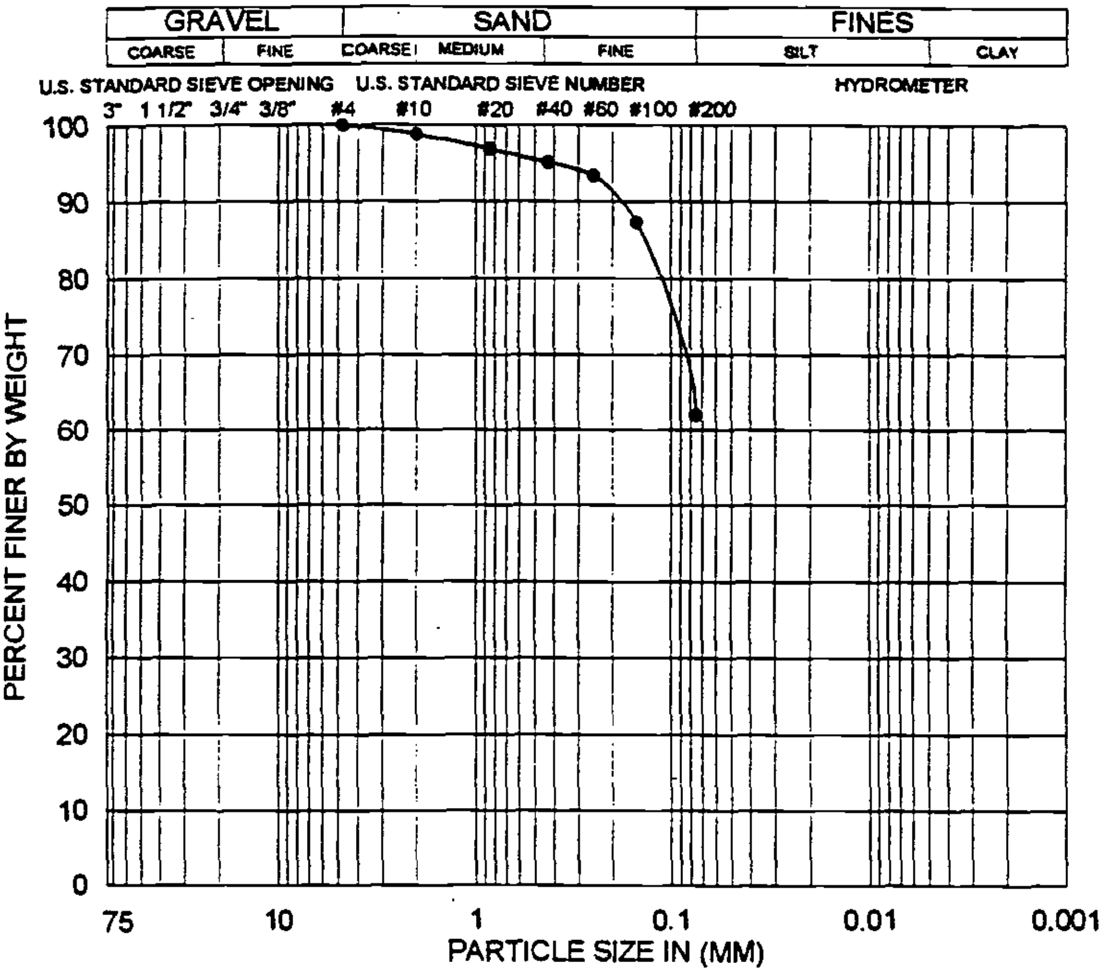
Figure B-55





BORING NO.	SAMPLE NO.	DEPTH (ft)	SOIL TYPE	GR:SA:FI (%)	LL,PL,PI
DD-62	S-21	77.0	ML	0:38:62	N/A

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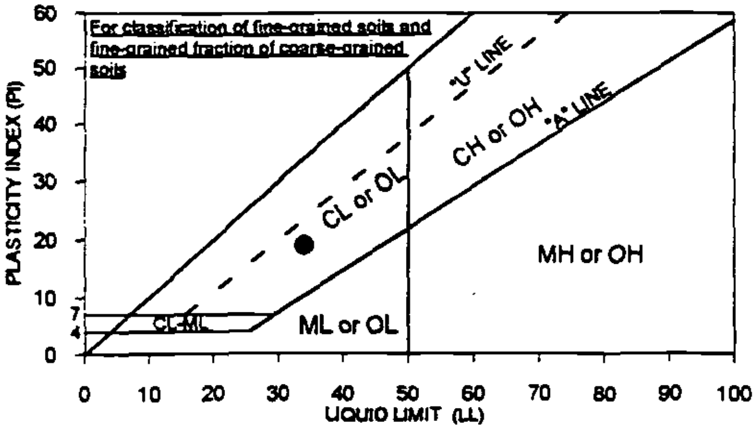
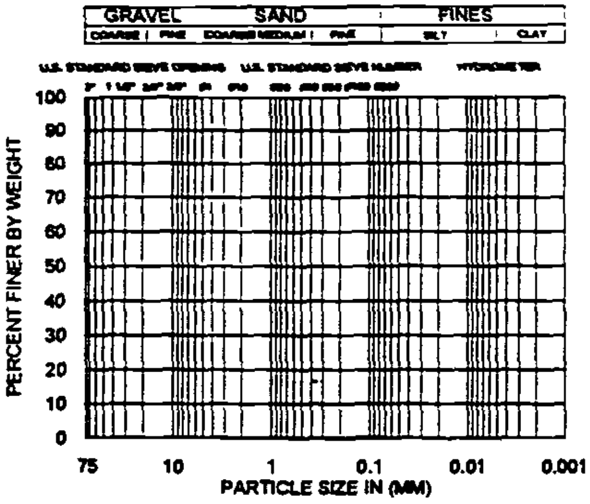
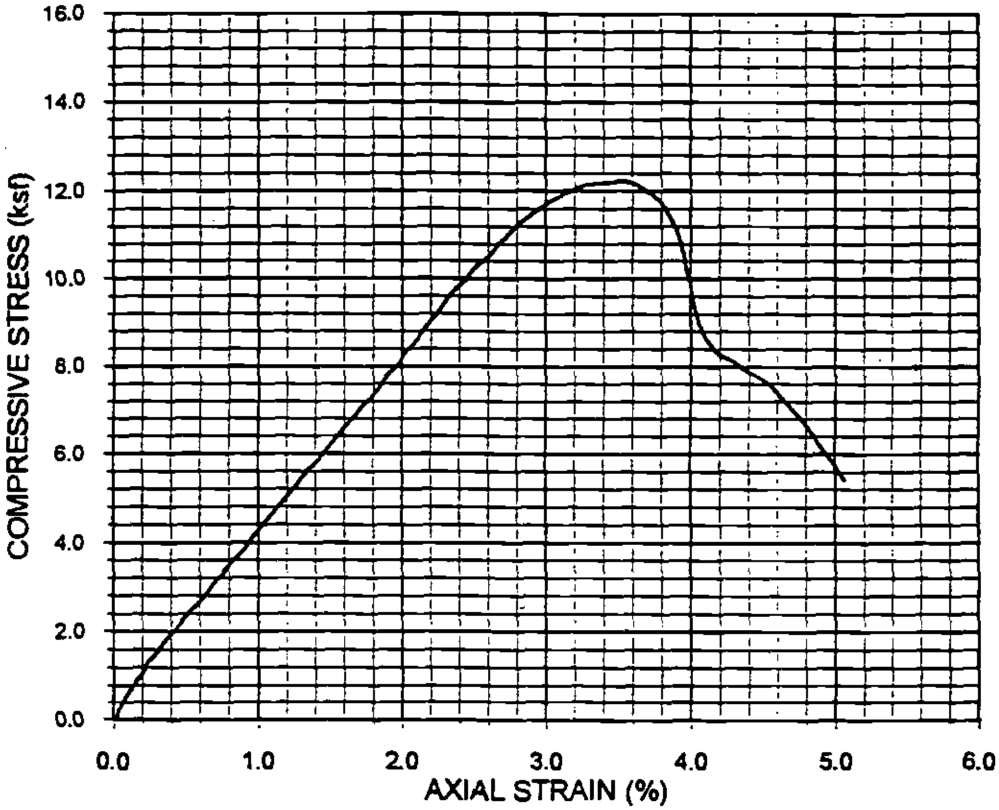
Atterberg Limits, Particle-Size Curve (ASTM D4318, D422)

12-95

Figure B-56







BORING NO.	SAMPLE NO.	DEPTH (ft)	Unconfined Compressive Strength (ksf)	Strain at Failure (%)	GR:SA:FI (%)	LL,PL,PI
SD-16C	T-27	126.4-127.0	12.224	3.52	N/A	34,15,19

Sample Description: *Light Olive Brown Sandy Clay (CL)*
Sample Type : *Undisturbed Shelby Tube*
Initial Moisture Content (%) : *17.72*
Dry Density (pcf) : *113.5*
Avg. Sample Diameter (in.) : *2.875*
Avg. Sample Height (in.) : *5.975*
Degree of Saturation (%) : *99*
Strain Rate (inch/min) : *0.04*

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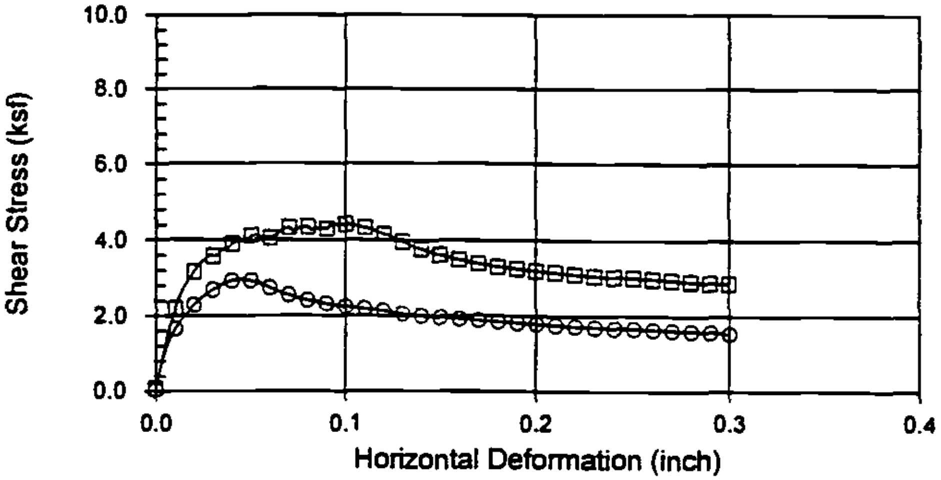
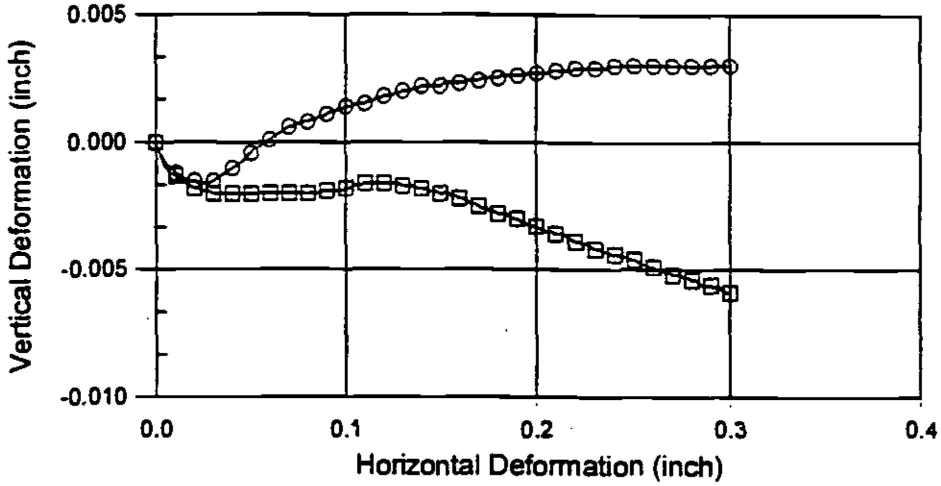
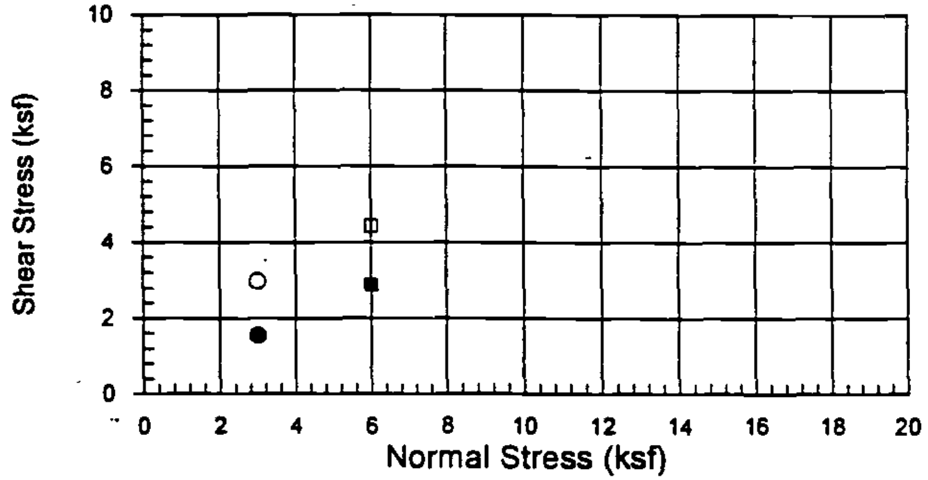
Atterberg Limits, Particle-Size Curve Unconfined Compressive Strength (ASTM D4318, D422, D2166)

12-95

Figure B-57







Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 2.980	● 1.546
6.000	□ 4.429	■ 2.878

Boring No. : SD-15
Sample No. : N/A
Depth (ft.) : 20.0
Sample Type : Undisturbed Ring
Soil Type : Yellowish Brown Lean Clay (CL)

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Direct Shear Test Results
(ASTM D3080)

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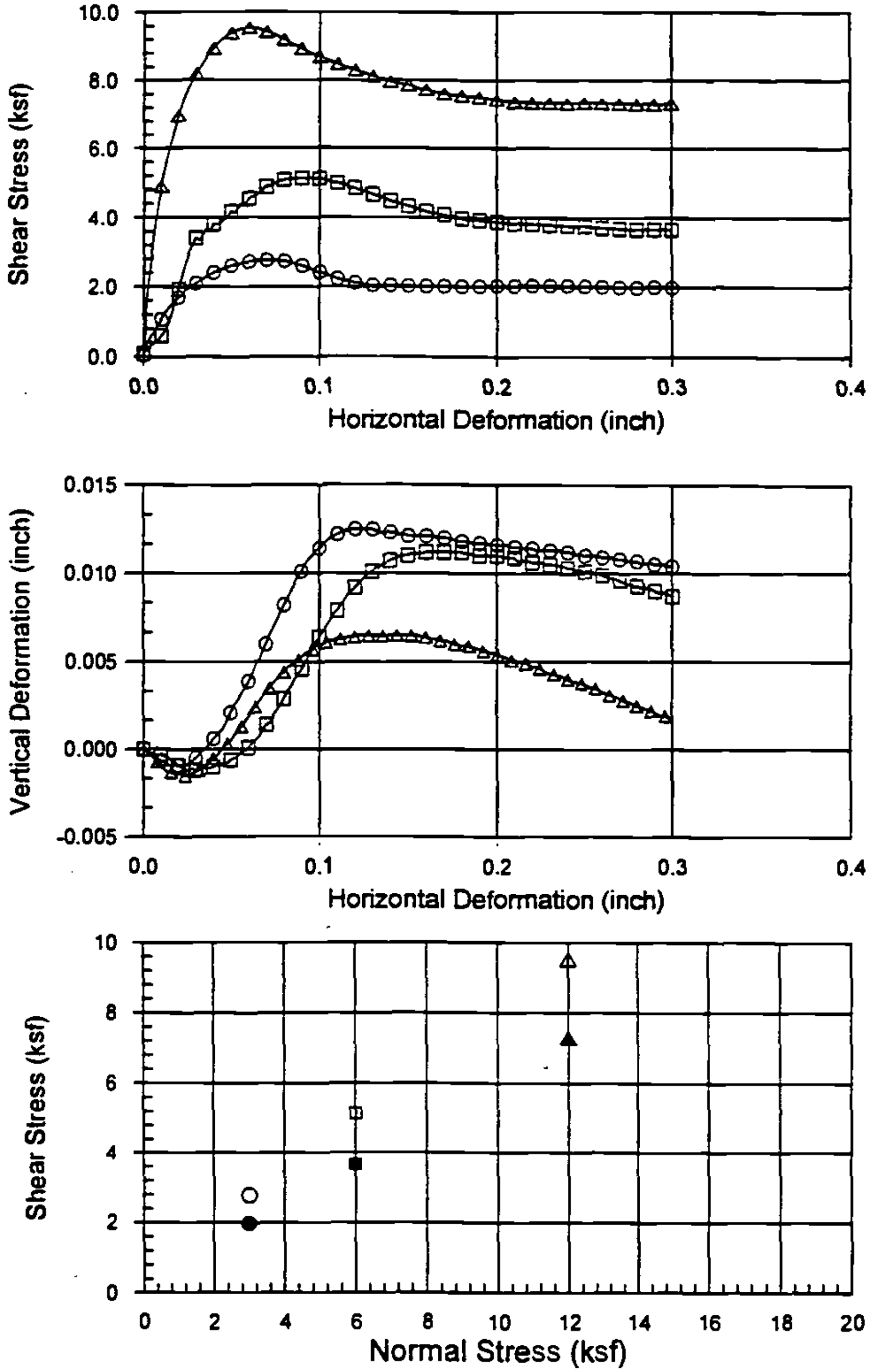
Figure B-58



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 2.776	● 1.978
6.000	□ 5.149	■ 3.666
12.000	△ 9.545	▲ 7.304

Boring No. : SD-15
Sample No. : N/A
Depth (ft.) : 50.0
Sample Type : Undisturbed Ring
Soil Type : Yellowish Brown Poorly Graded Sand with Silt (SP-SM)

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Direct Shear Test Results
(ASTM D3080)

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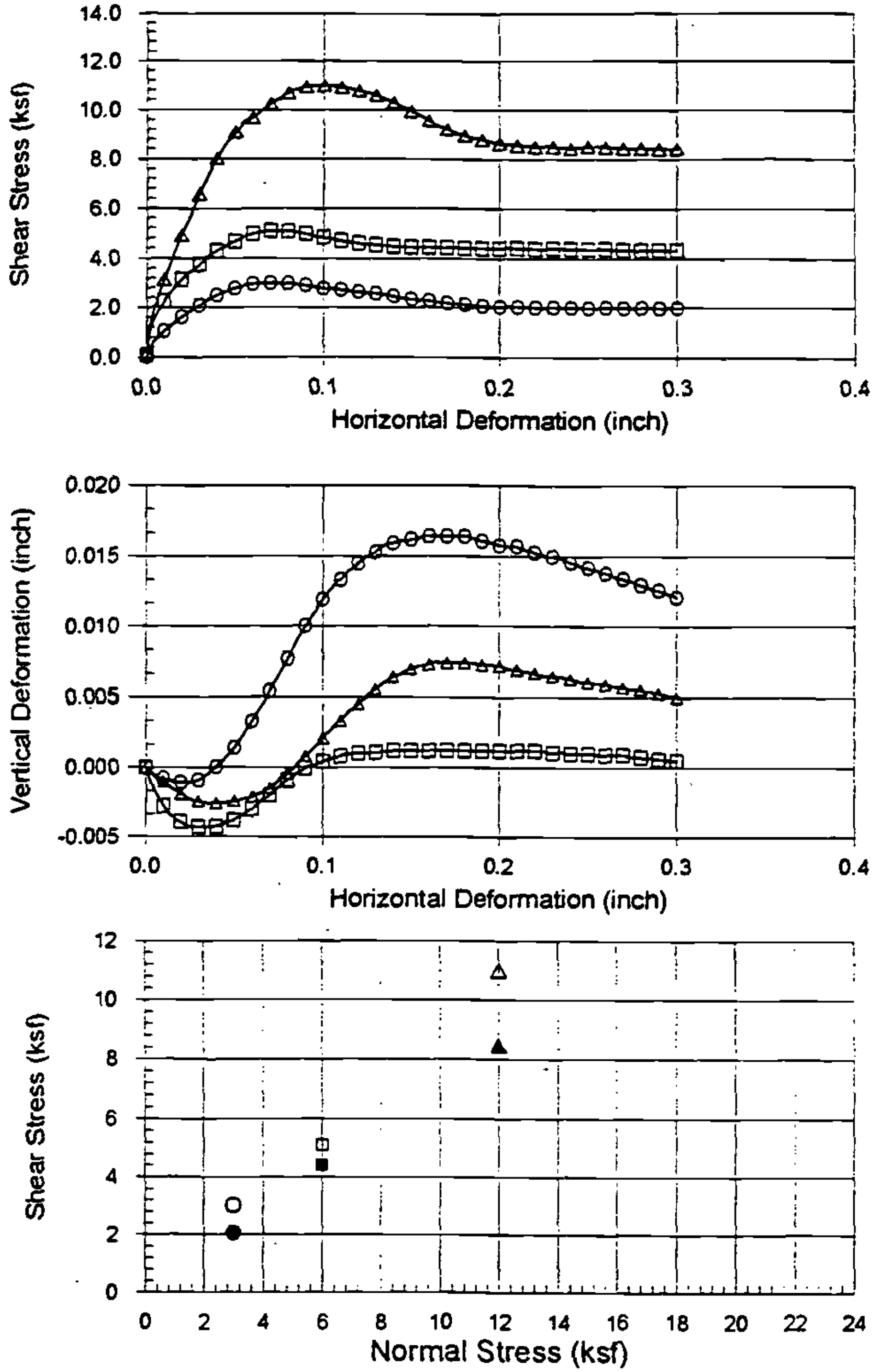
Figure B-59



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 3.022	● 2.052
6.000	□ 5.119	■ 4.402
12.000	△ 10.995	▲ 8.471

Boring No. : SD-16C
Sample No. : D-7
Depth (ft.) : 39.0
Sample Type : Undisturbed Ring
Soil Type : Dark Brown Silty Sand (SM)

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Direct Shear Test Results
(ASTM D3080)

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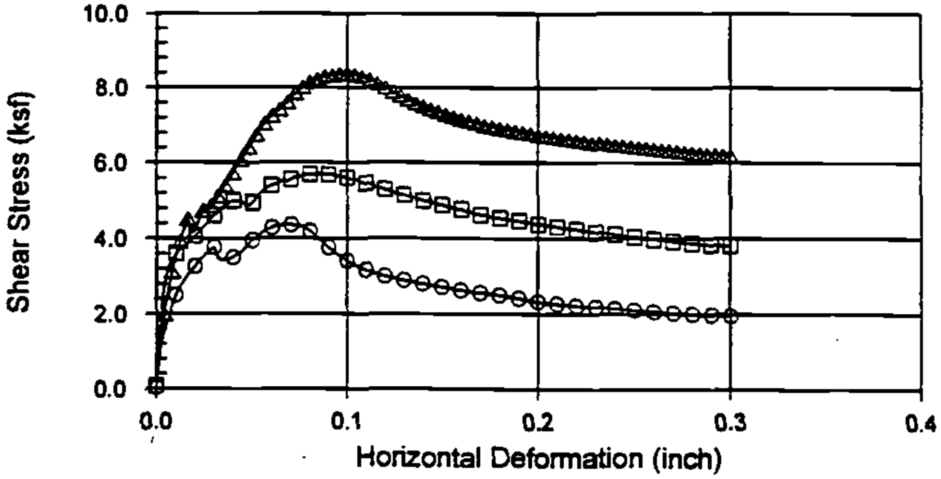
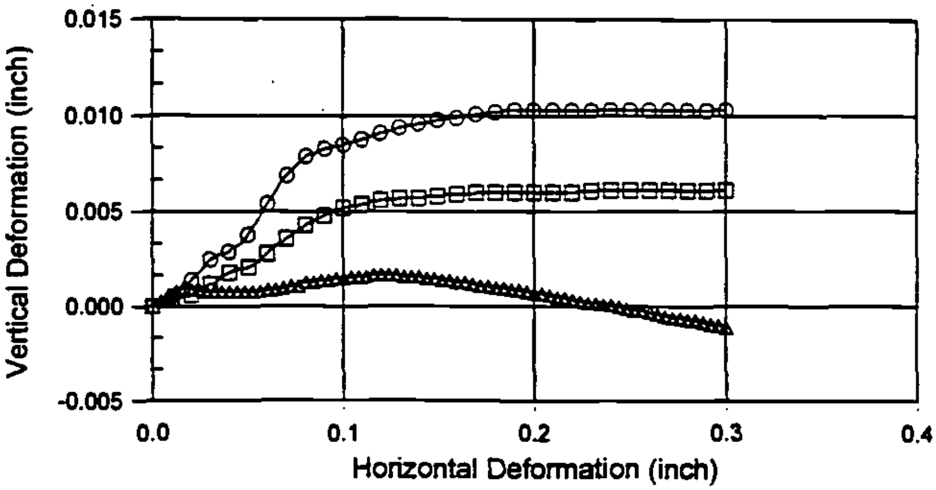
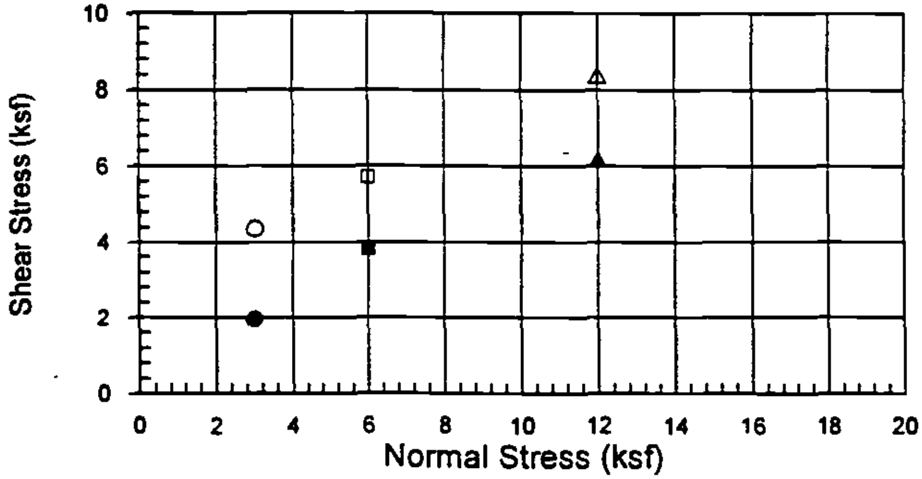
Figure B-60







Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 4.363	● 1.960
6.000	□ 5.717	■ 3.815
12.000	△ 8.386	▲ 6.178

Boring No. : SD-16C
Sample No. : D-26
Depth (ft.) : 123.0
Sample Type : Undisturbed Ring
Soil Type : Olive Brown Lean Clay (CL), stiff

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Direct Shear Test Results
(ASTM D3080)

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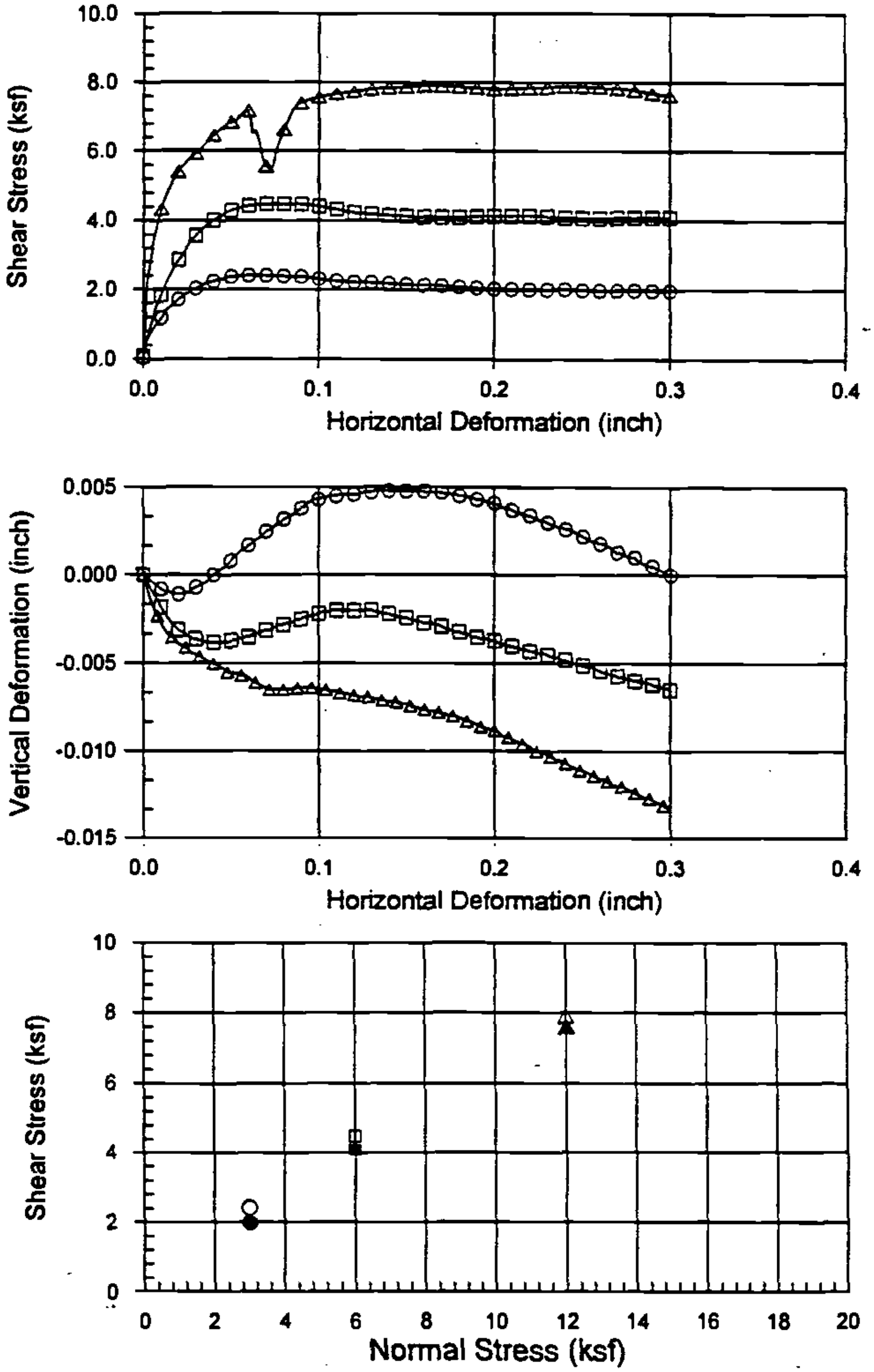
Figure B-61



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 2.415	● 1.976
6.000	□ 4.492	■ 4.094
12.000	△ 7.919	▲ 7.625

Boring No. : SD-17
Sample No. : N/A
Depth (ft.) : 45.0
Sample Type : Undisturbed Ring
Soil Type : Yellowish Brown Sandy Silt (ML)

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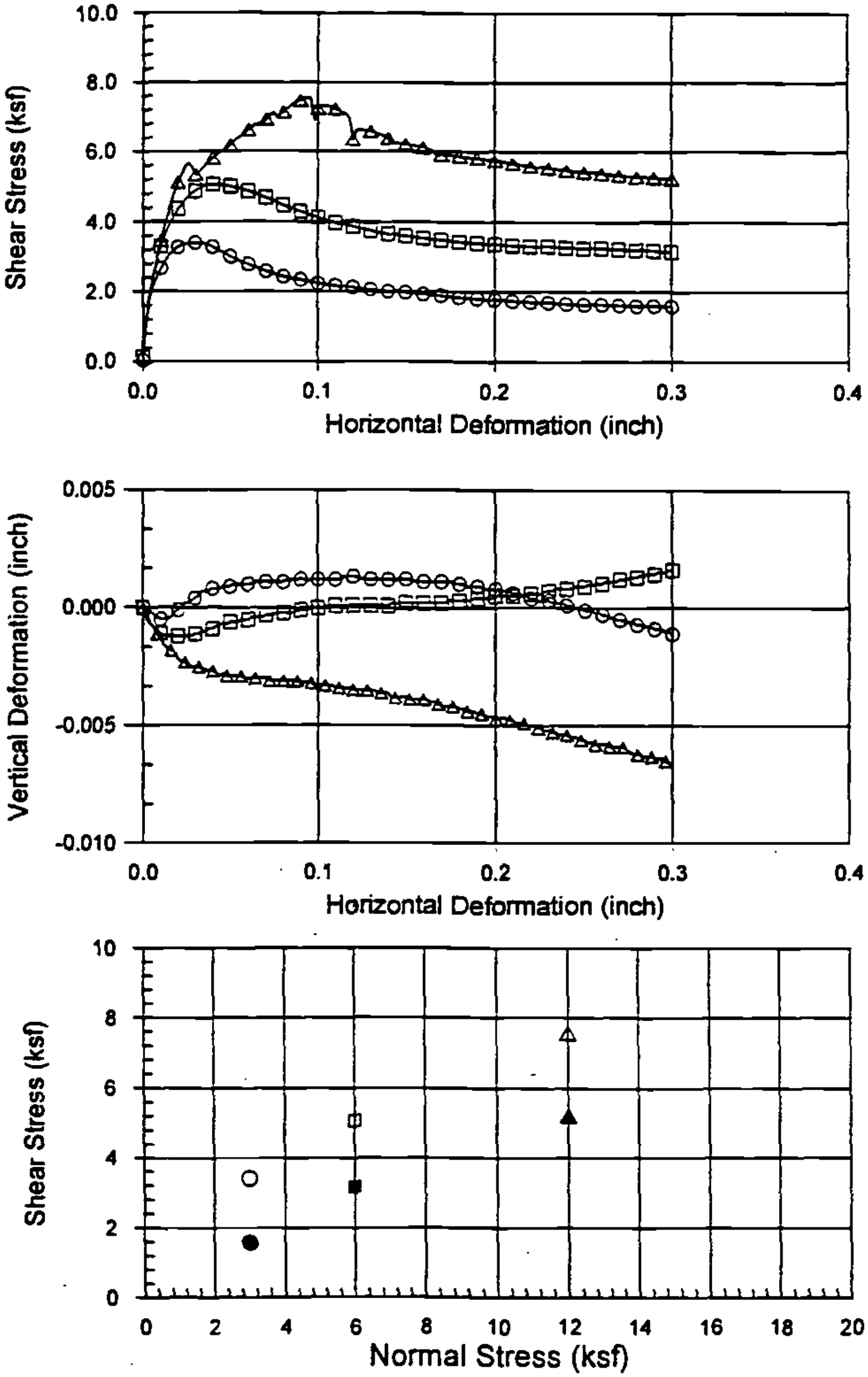
Direct Shear Test Results
(ASTM D3080)

12-95

Figure B-62



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 3.417	● 1.590
6.000	□ 5.075	■ 3.161
12.000	△ 7.566	▲ 5.230

Boring No. : SD-17
Sample No. : N/A
Depth (ft.) : 85.0
Sample Type : Undisturbed Ring
Soil Type : Brown Lean Clay (CL) with fine Sand

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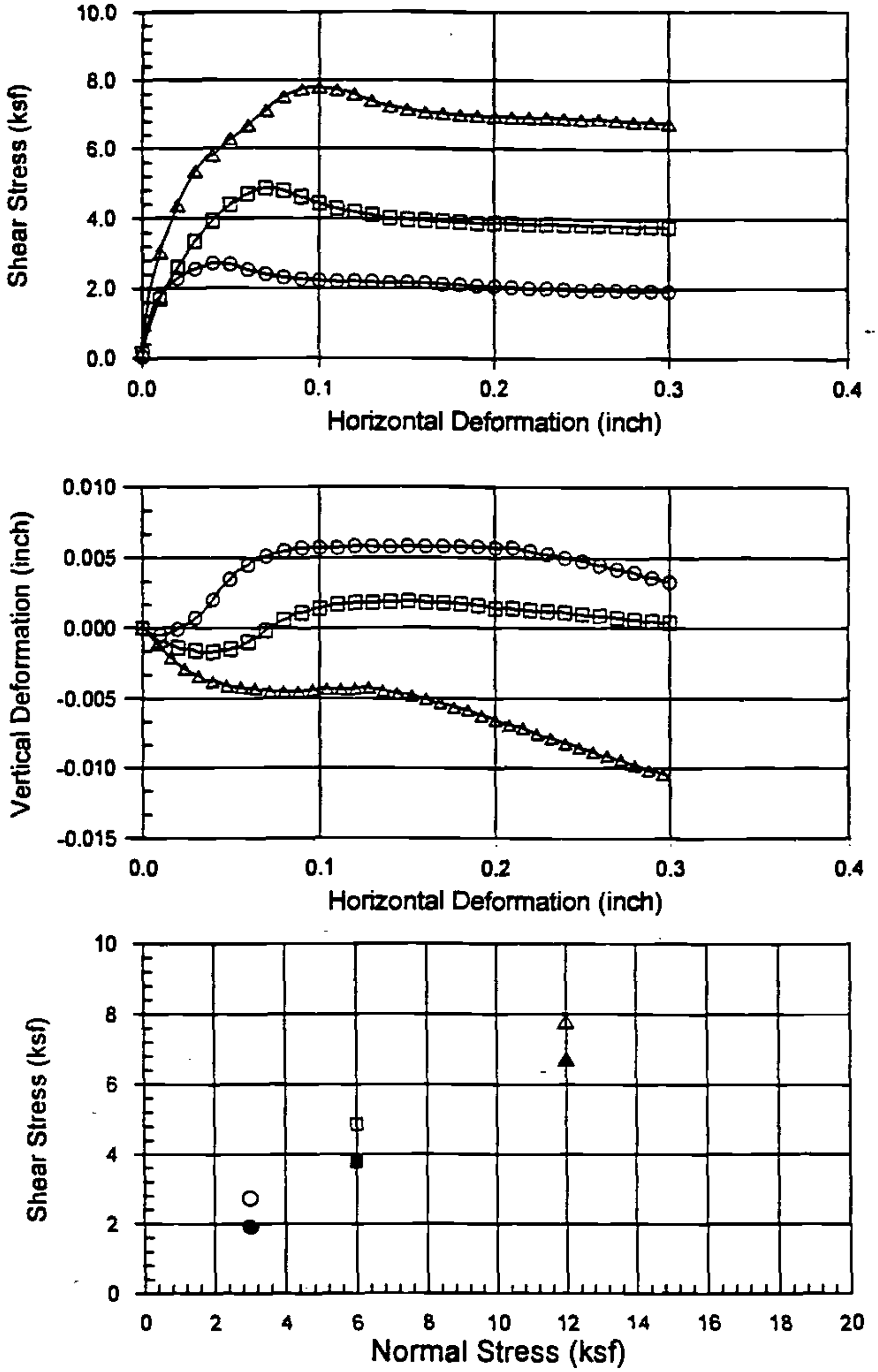
Direct Shear Test Results
(ASTM D3080)

12-95

Figure B-63



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 2.731	● 1.920
6.000	□ 4.865	■ 3.762
12.000	△ 7.802	▲ 6.745

Boring No. : SD-18C
Sample No. : D-17
Depth (ft.) : 80.0
Sample Type : Undisturbed Ring
Soil Type : Dark Yellowish Brown Sandy Silt (ML)

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Direct Shear Test Results
(ASTM D3080)

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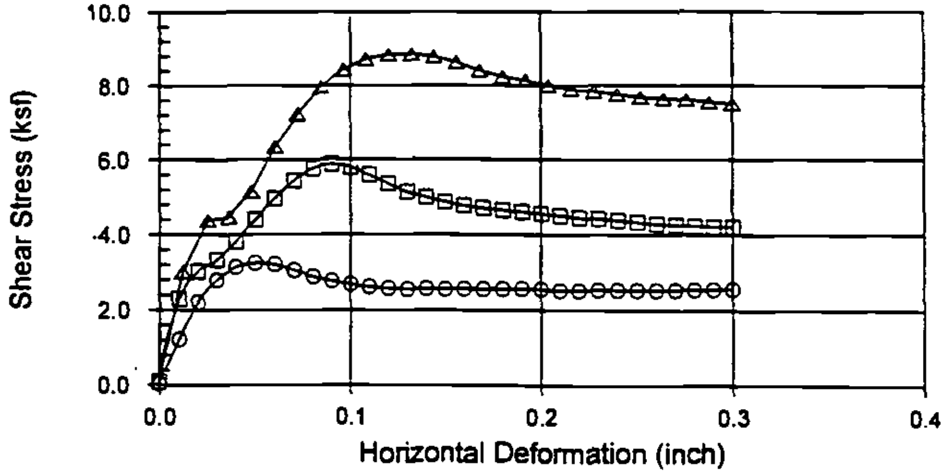
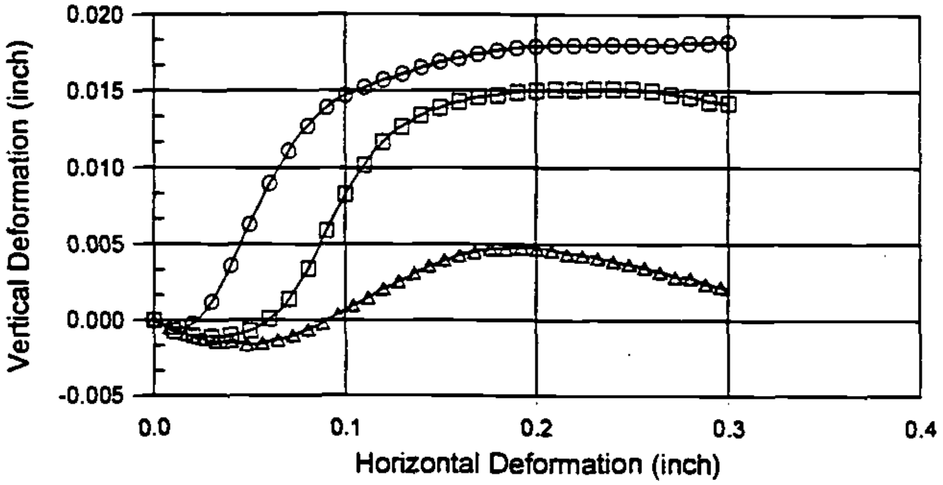
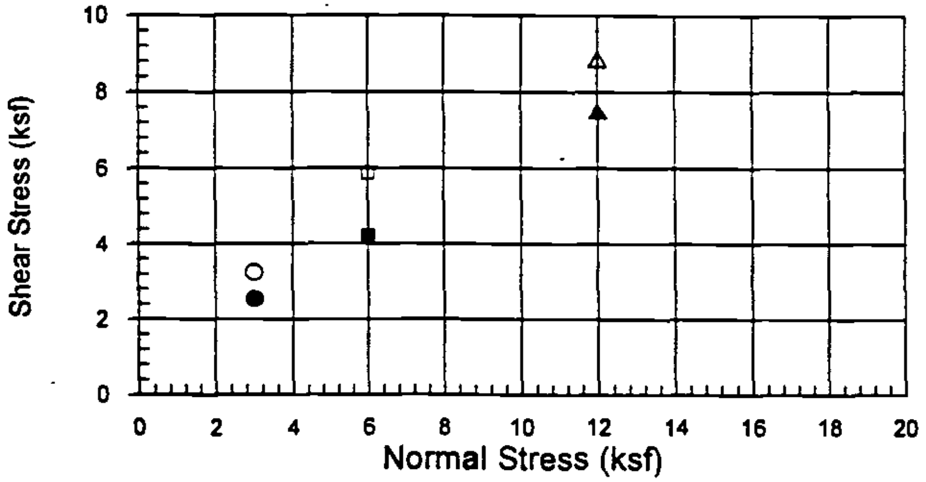
Figure B-64







Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 3.246	● 2.541
6.000	□ 5.880	■ 4.215
12.000	△ 8.870	▲ 7.512

Boring No. : DD-27S
Sample No. : D-24
Depth (ft.) : 81.0
Sample Type : Undisturbed Ring
Soil Type : Olive Brown Silty Sand (SM)

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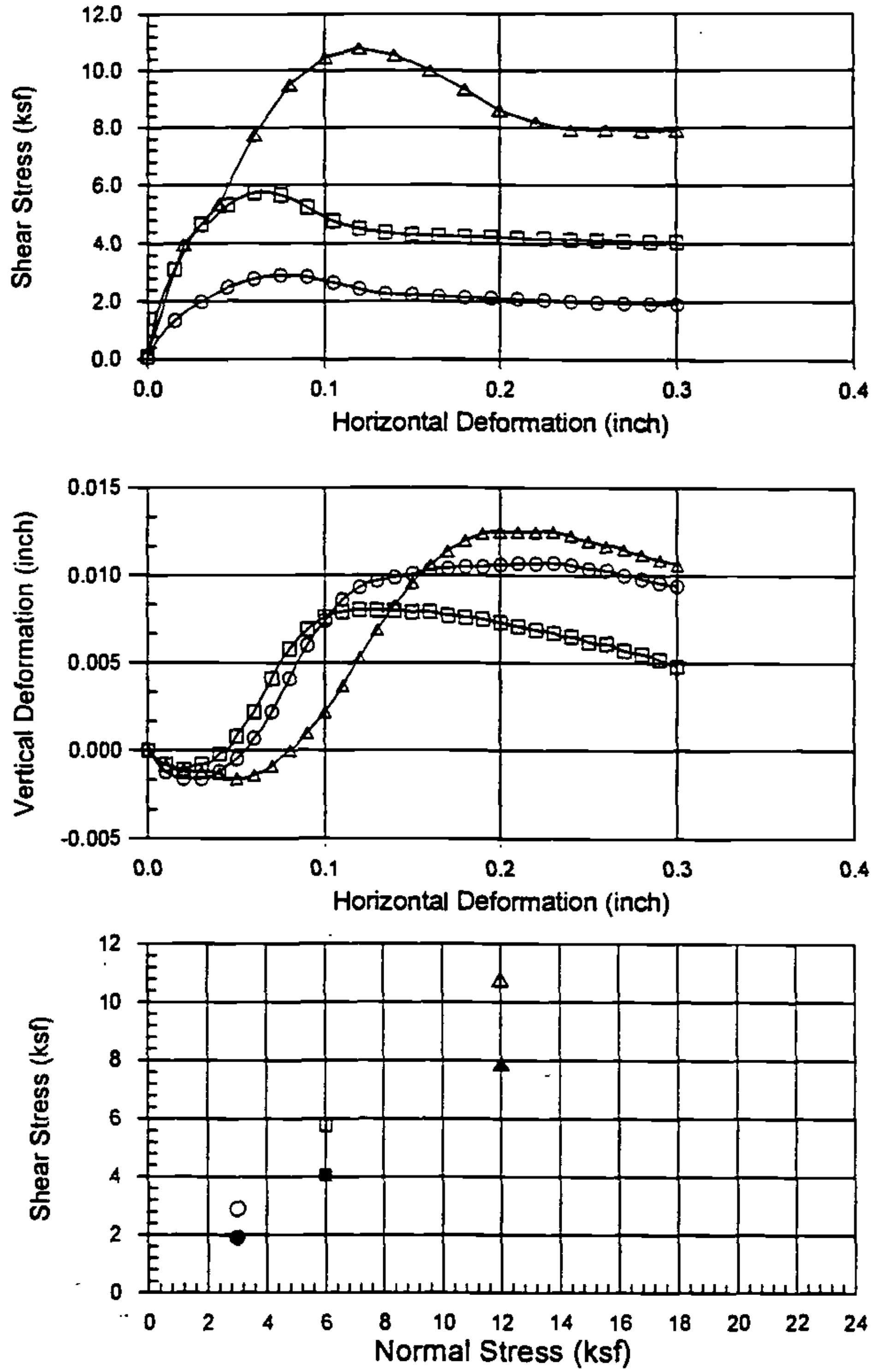
Direct Shear Test Results
(ASTM D3080)

12-95

Figure B-65



Normal Stress (ksf)	Peak Shear Stress (ksf)	Residual Shear Stress (ksf)
3.000	○ 2.909	● 1.915
6.000	□ 5.781	■ 4.062
12.000	△ 10.783	▲ 7.895

Boring No. : DD-28
Sample No. : D-19
Depth (ft.) : 85.0
Sample Type : Undisturbed Ring
Soil Type : Yellowish Brown Silty Sand (SM)

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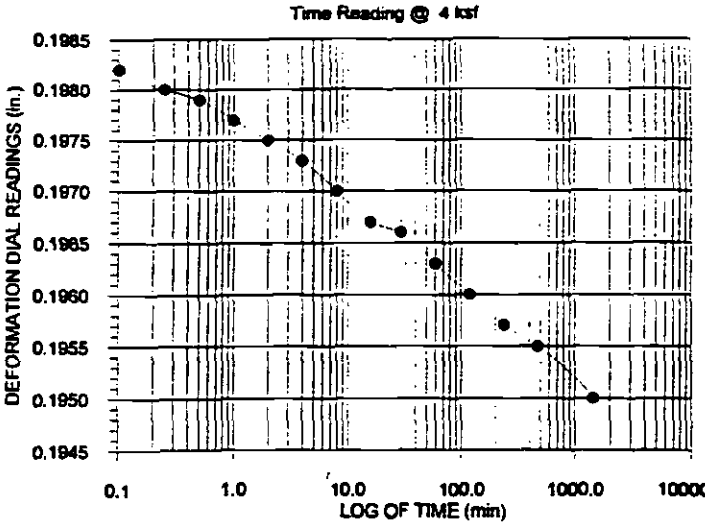
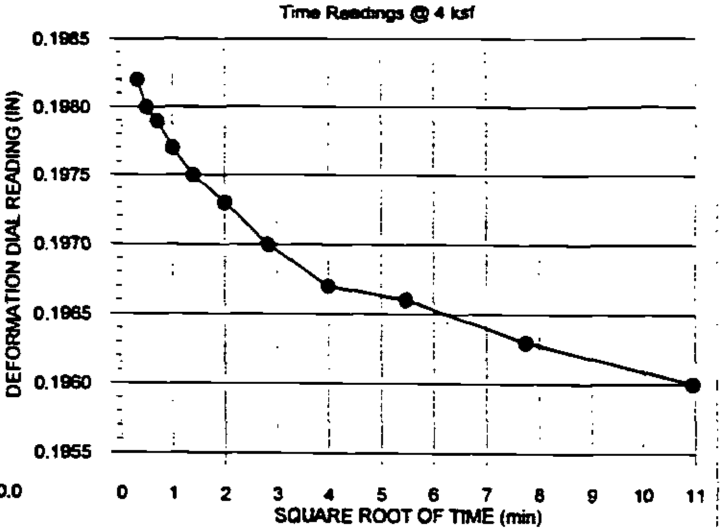
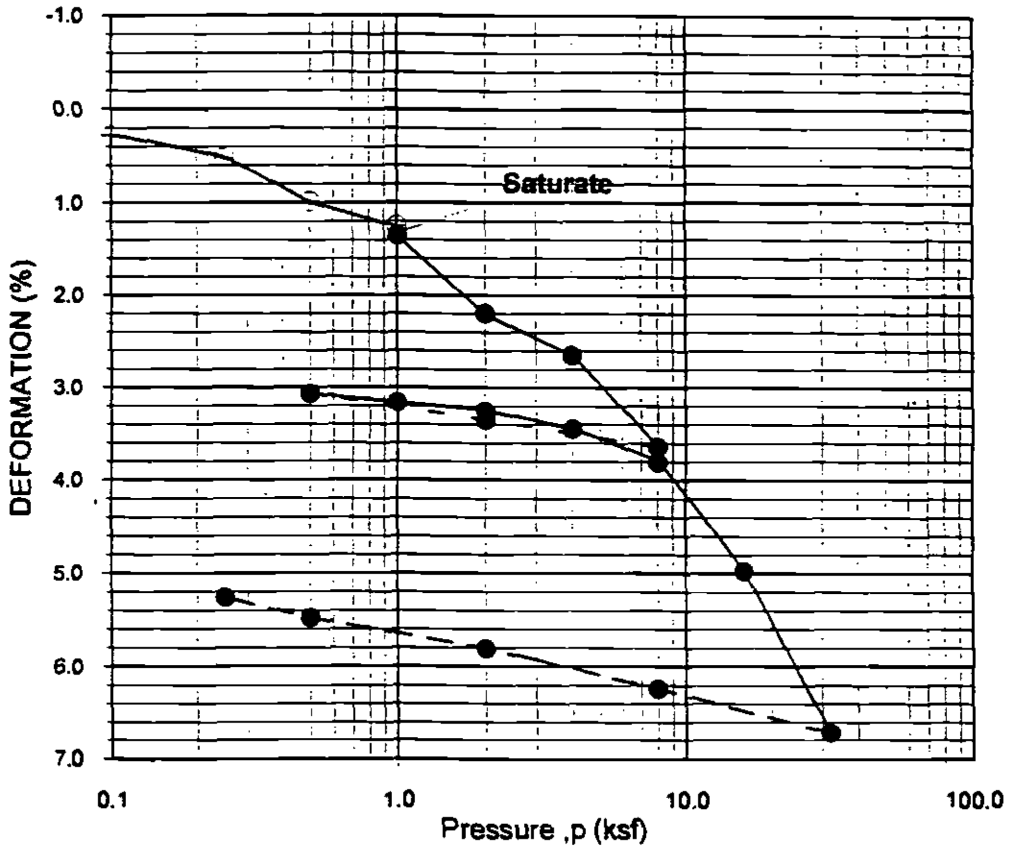
Direct Shear Test Results
(ASTM D3080)

12-95

Figure B-66







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-15	N/A	90.0	19.5 / 17.3	105.1 / 110.8	0.604 / 0.520	87 / 90

SOIL DESCRIPTION: Dark Brown Silty Sand (SM)

GeoTransit Consultants

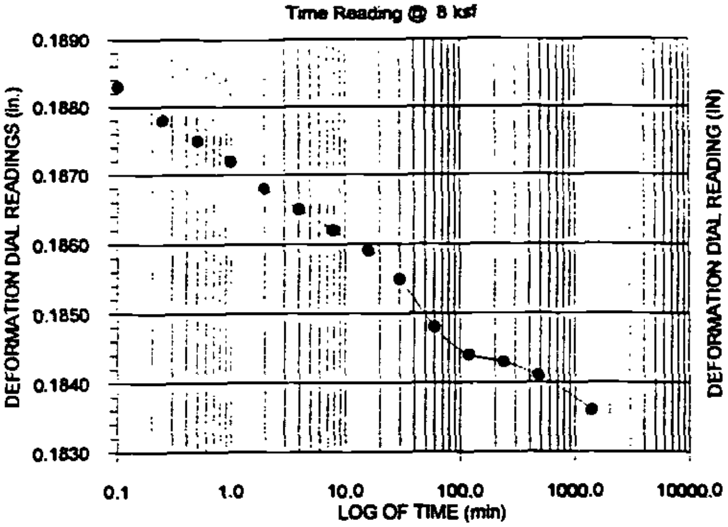
Project No.: 95-8347-14

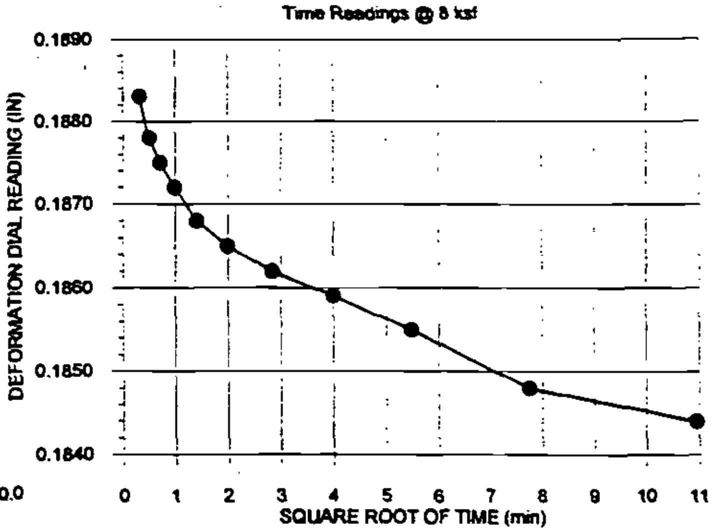
Eastside Extension
Metro Redline II

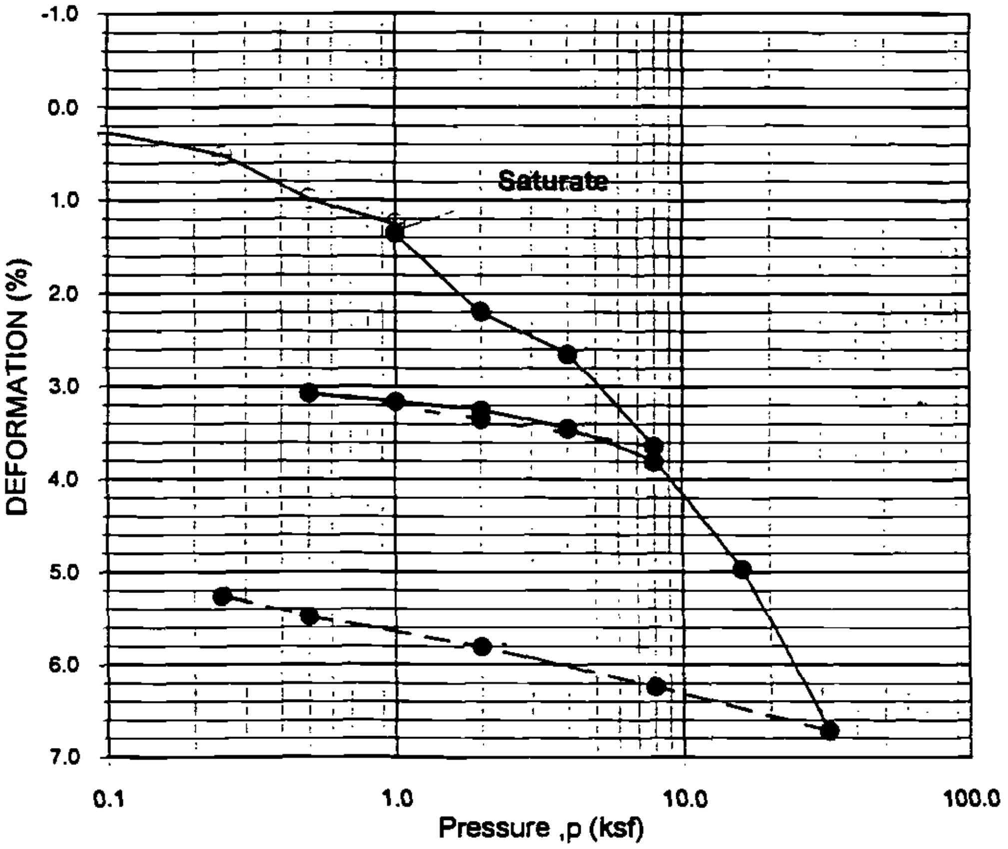
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-67a







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-15	N/A	90.0	19.5 / 17.3	105.1 / 110.8	0.604 / 0.520	87 / 90

SOIL DESCRIPTION: Dark Brown Silty Sand (SM)

GeoTransit Consultants

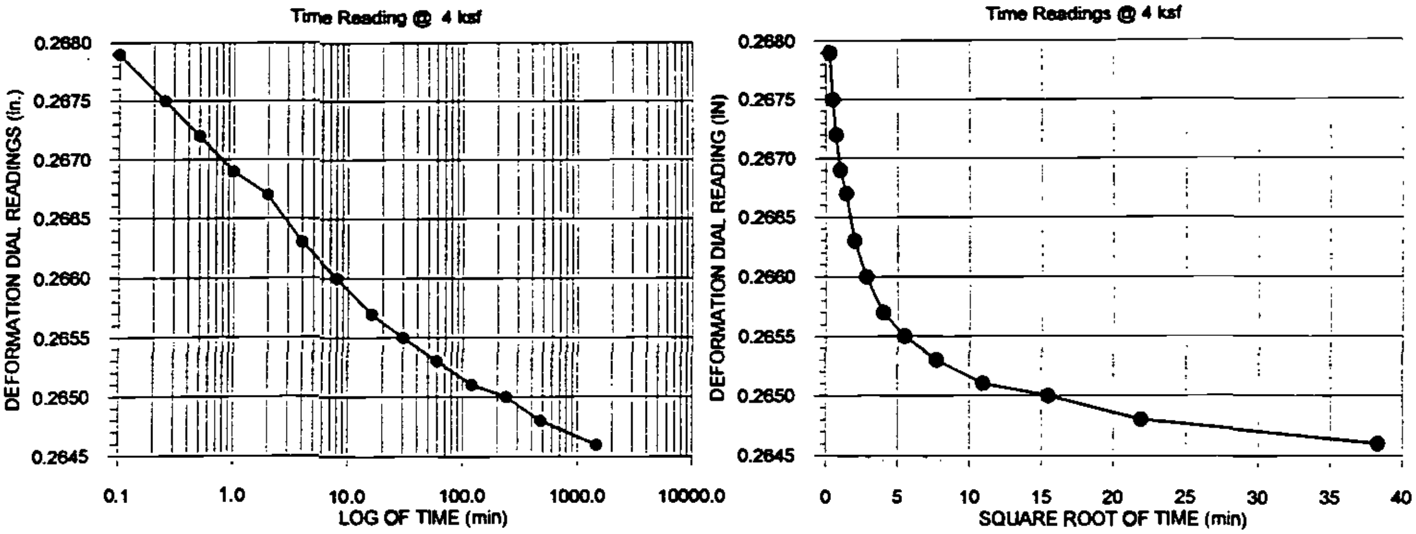
Project No.: 95-8347-14

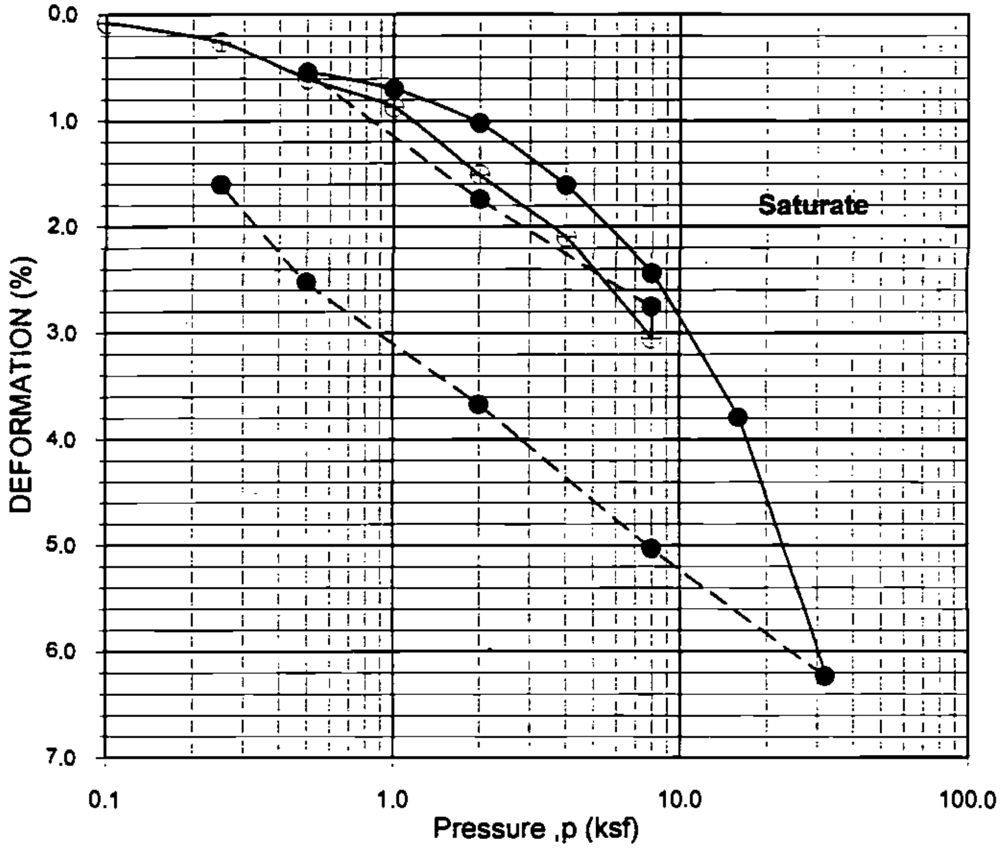
Eastside Extension
Metro Redline II

One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-67b





BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-16C	D-13	68.5	22.3 / 23.6	103.0 / 104.6	0.637 / 0.611	94 / 100

SOIL DESCRIPTION: Dark Olive Fat Clay (CH)

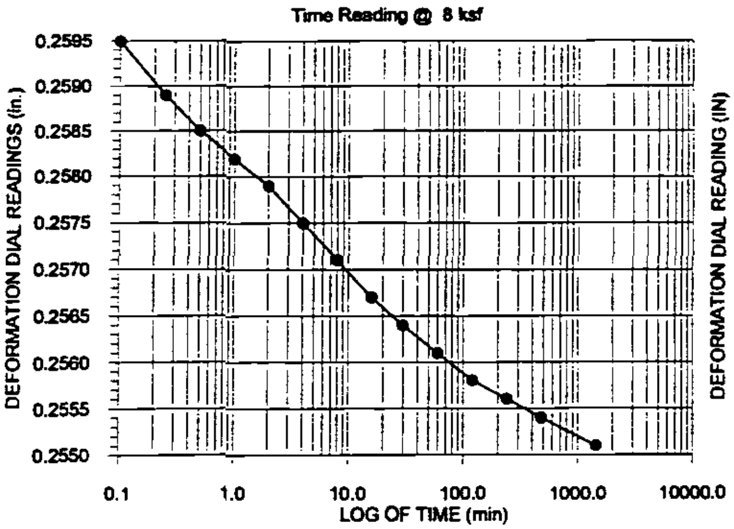
GeoTransit Consultants

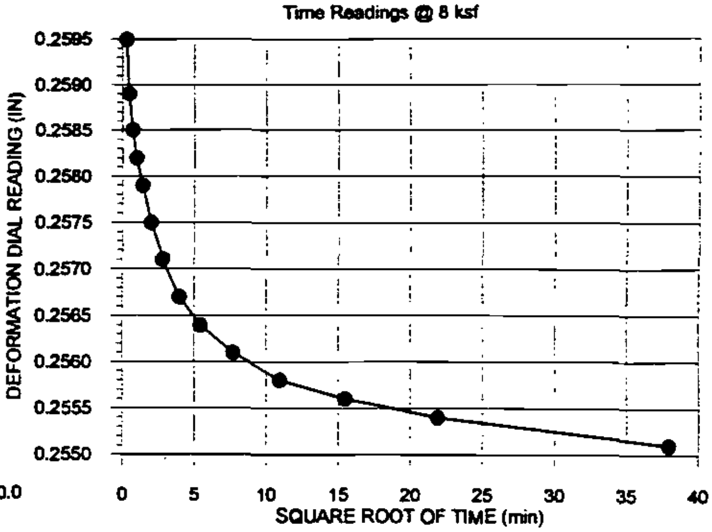
Project No. 95-8347-14
Eastside Extension
Metro Redline II

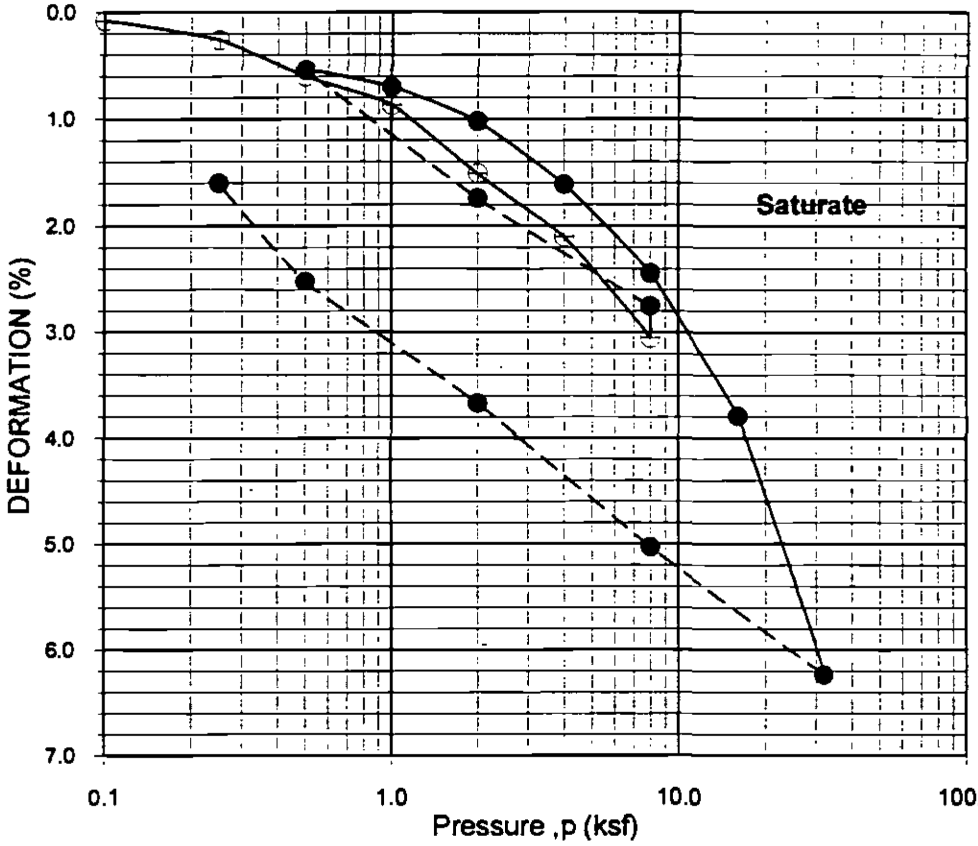
ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES OF SOILS
(ASTM D 2435)

12-95

Figure B-68a







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-16C	D-13	68.5	22.3 / 23.6	103.0 / 104.6	0.637 / 0.611	94 / 100

SOIL DESCRIPTION: Dark Olive Fat Clay (CH)

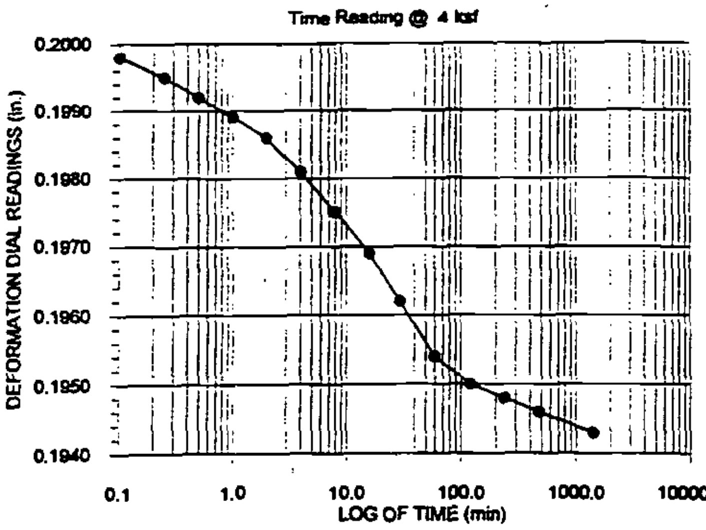
GeoTransit Consultants

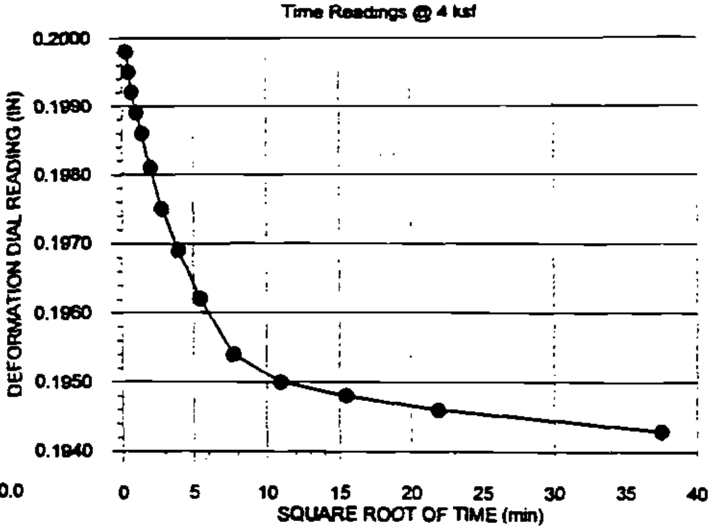
Project No. 95-8347-14
Eastside Extension
Metro Redline II

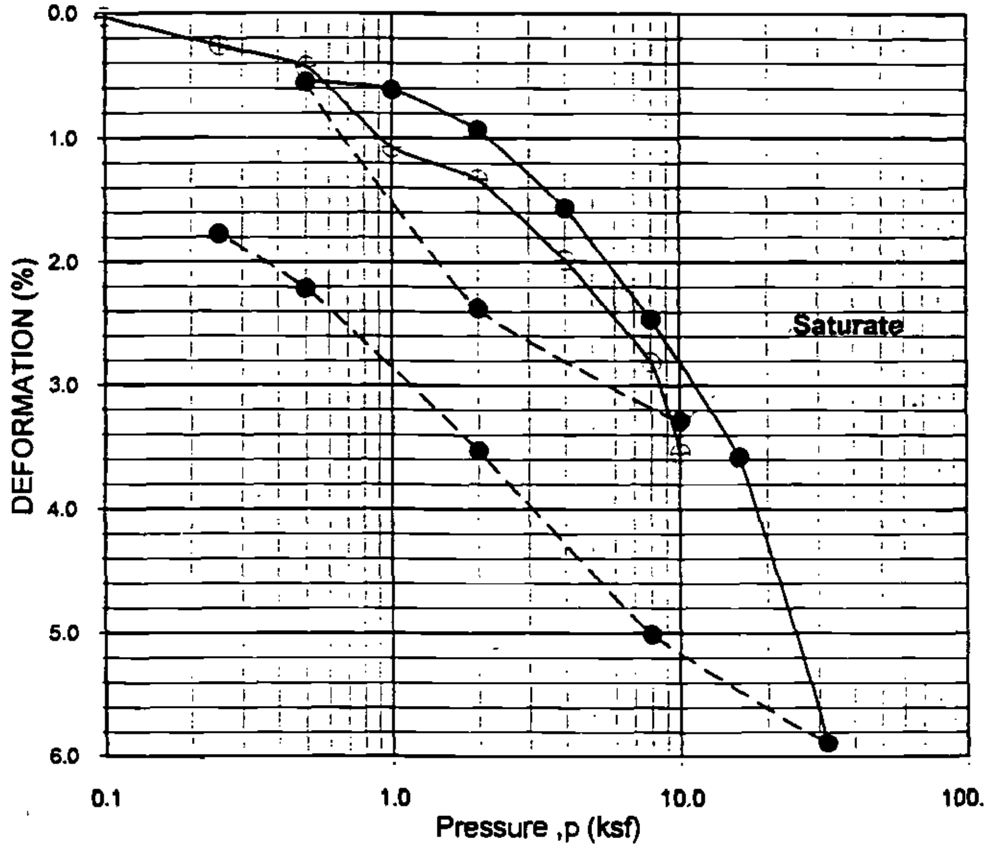
ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES OF SOILS
(ASTM D 2435)

12-95

Figure B-68b







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-16C	D-18	89.0	20.0 / 19.9	108.0 / 109.8	0.561 / 0.534	96 / 100

SOIL DESCRIPTION: Dark Brown Lean Clay (CL)

GeoTransit Consultants

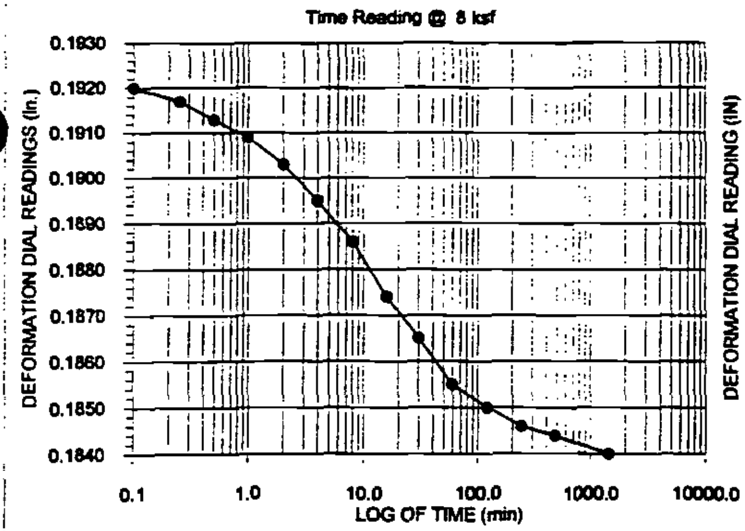
Project No.: 95-8347-14

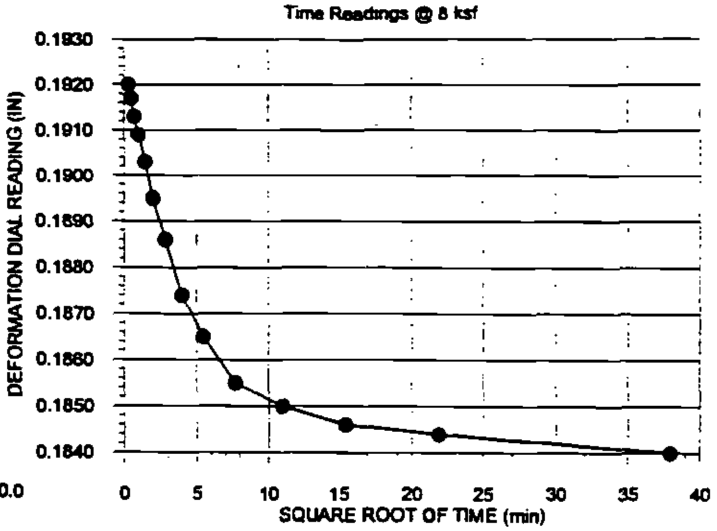
Eastside Extension
Metro Redline II

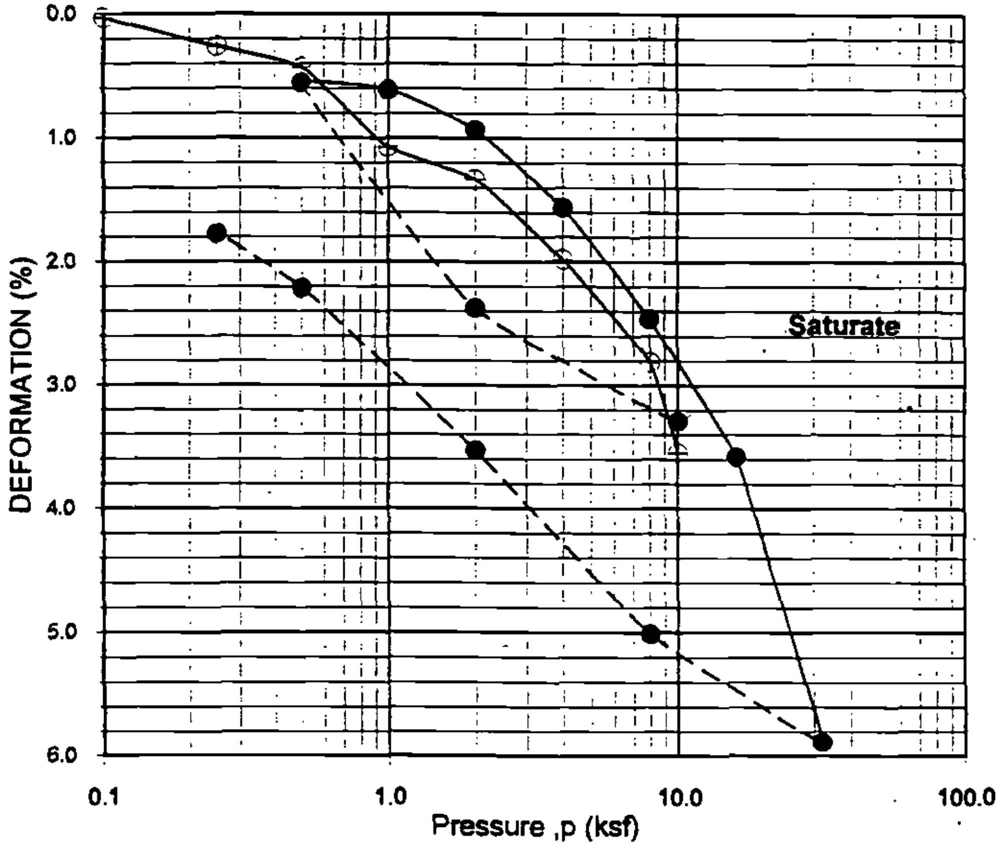
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-69a







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-16C	D-18	89.0	20.0 / 19.9	108.0 / 109.8	0.561 / 0.534	96 / 100

SOIL DESCRIPTION: Dark Brown Lean Clay (CL)

GeoTransit Consultants

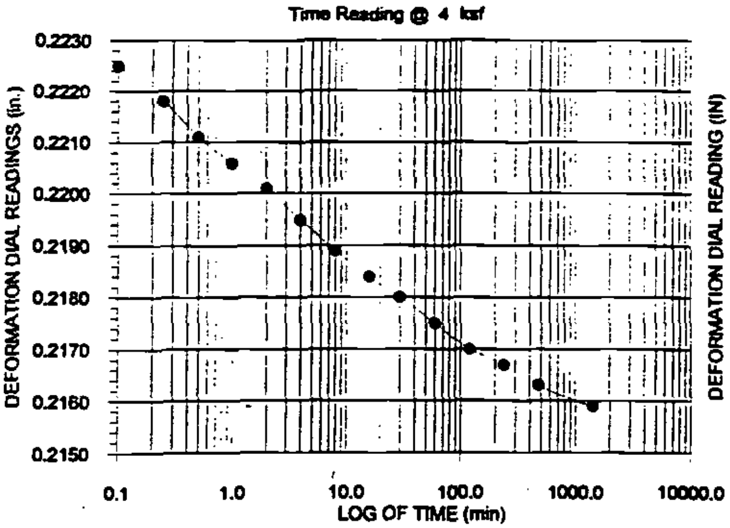
Project No.: 95-8347-14

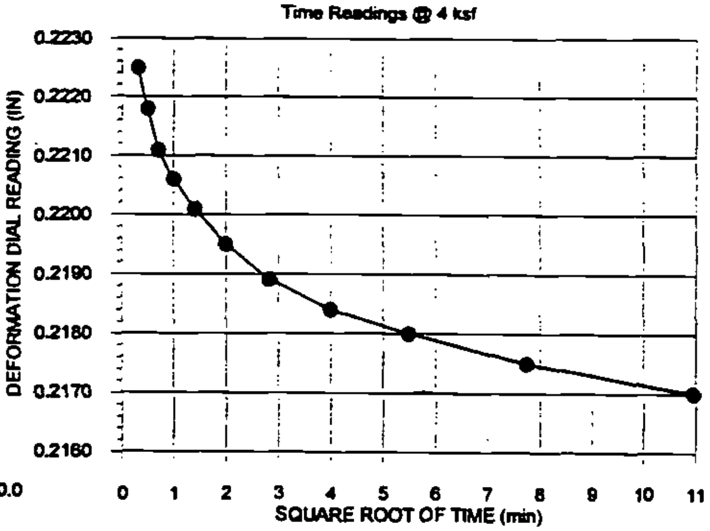
Eastside Extension
Metro Redline II

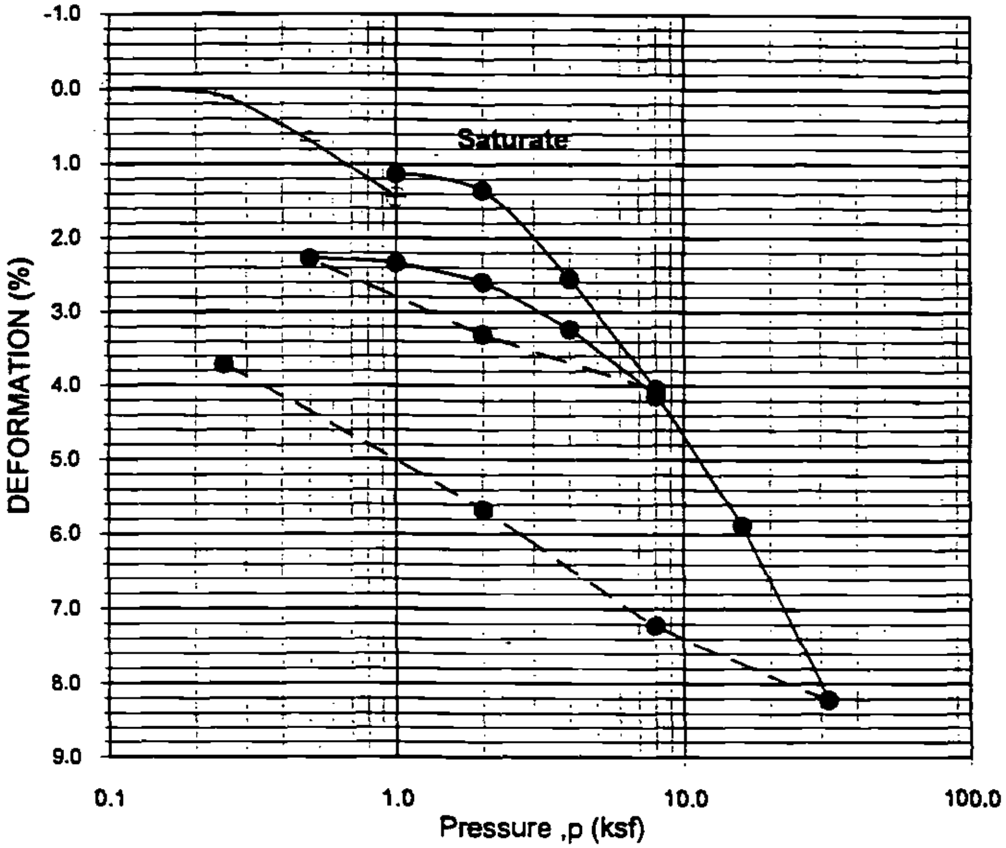
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-69b







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-17		85.0	18.8 / 20.6	108.5 / 113.7	0.554 / 0.496	92 / 100

SOIL DESCRIPTION: Dark Olive Brown Lean Clay (CL)

GeoTransit Consultants

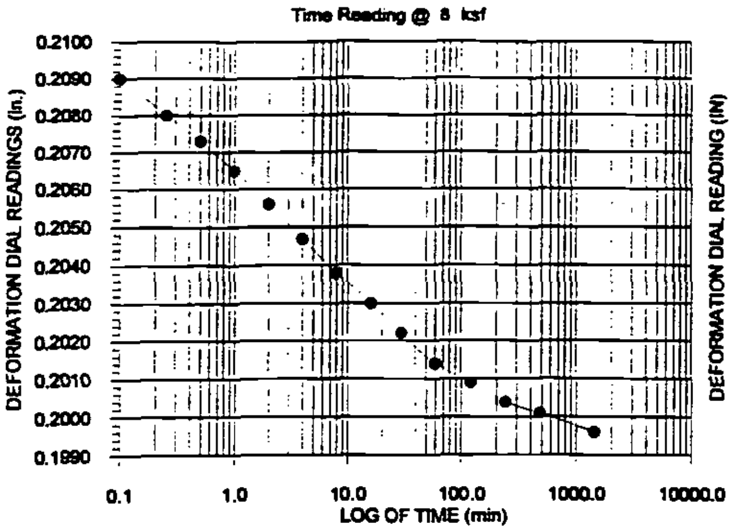
Project No.: 95-8347-14

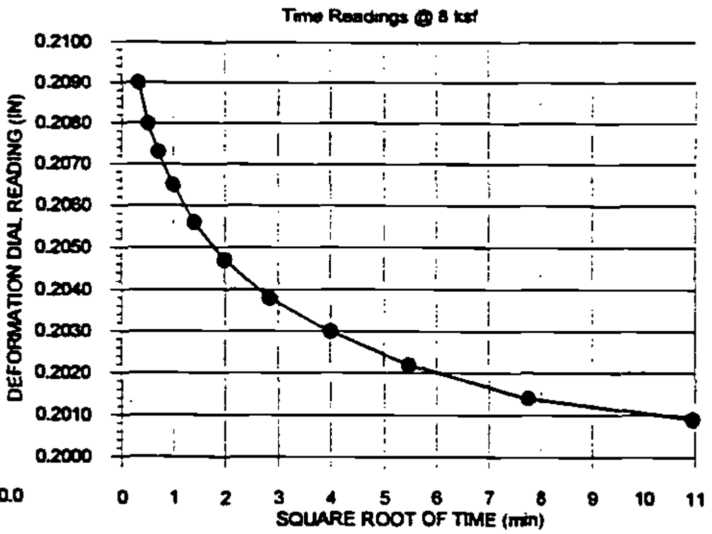
Eastside Extension
Metro Redline II

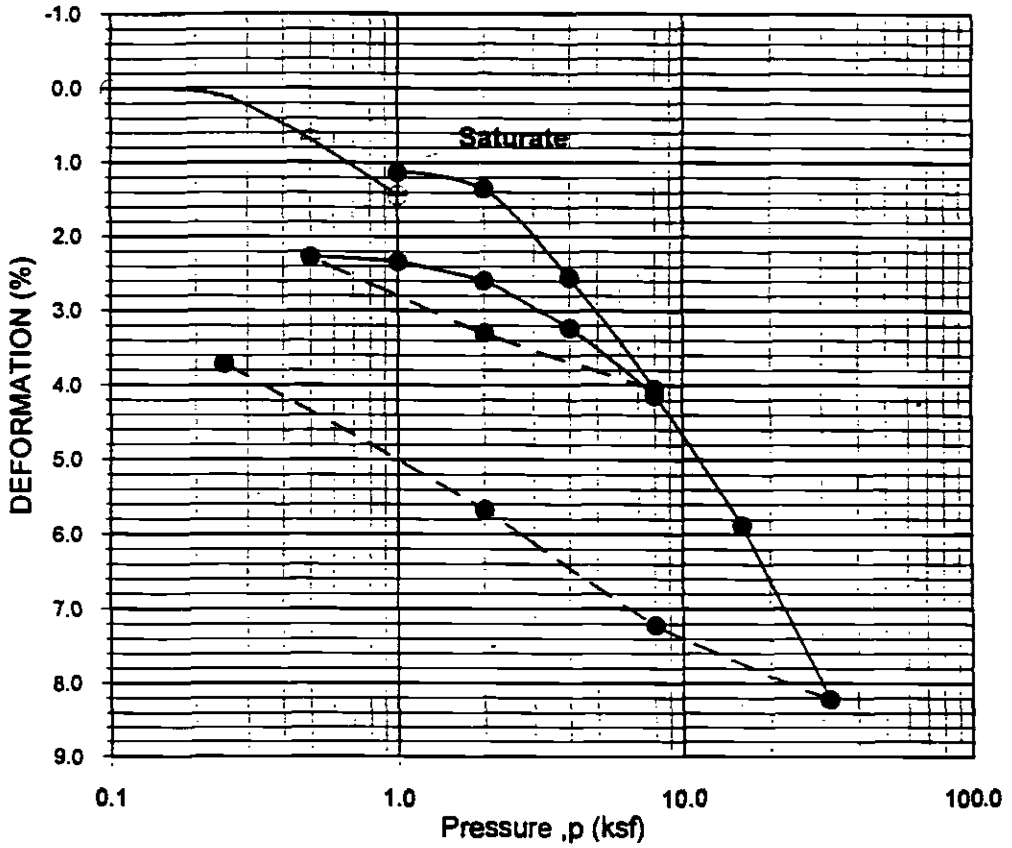
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-70a







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-17		85.0	18.8 / 20.6	108.5 / 113.7	0.554 / 0.496	92 / 100

SOIL DESCRIPTION: Dark Olive Brown Lean Clay (CL)

GeoTransit Consultants

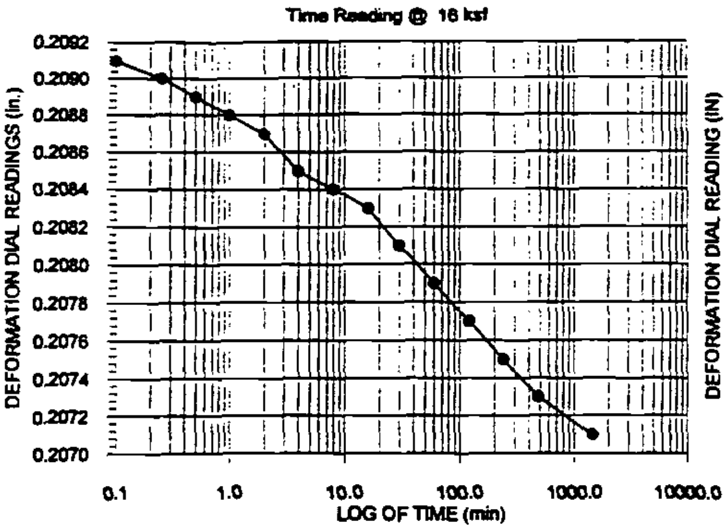
Project No.: 95-8347-14

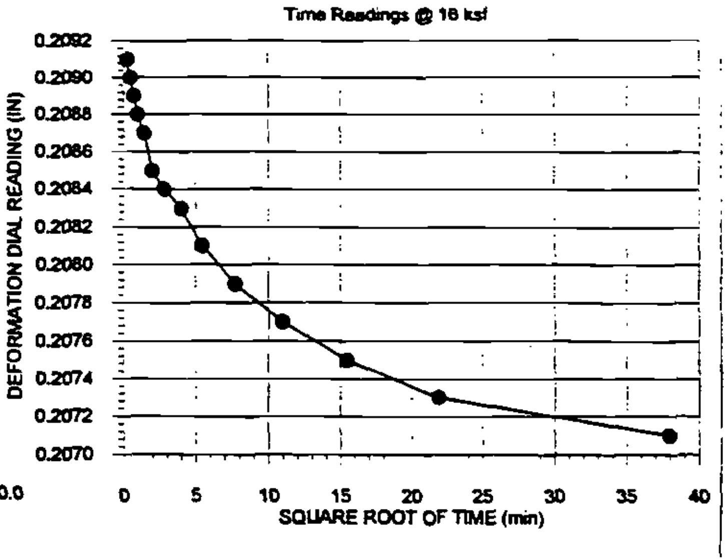
Eastside Extension
Metro Redline II

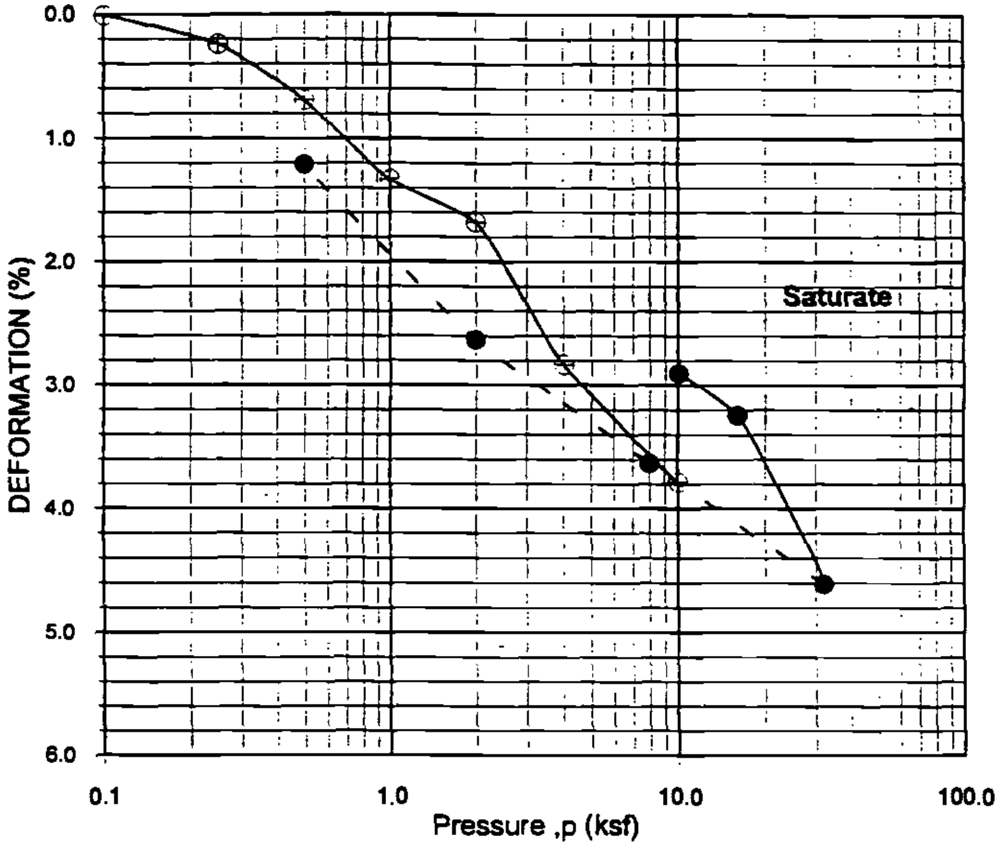
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-70b


Time Reading @ 16 ksf


Time Readings @ 16 ksf



BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-17	21	105.0	20.2 / 21.6	105.7 / 106.4	0.595 / 0.575	92 / 100

SOIL DESCRIPTION: Dark Olive Lean Clay (CL)

GeoTransit Consultants

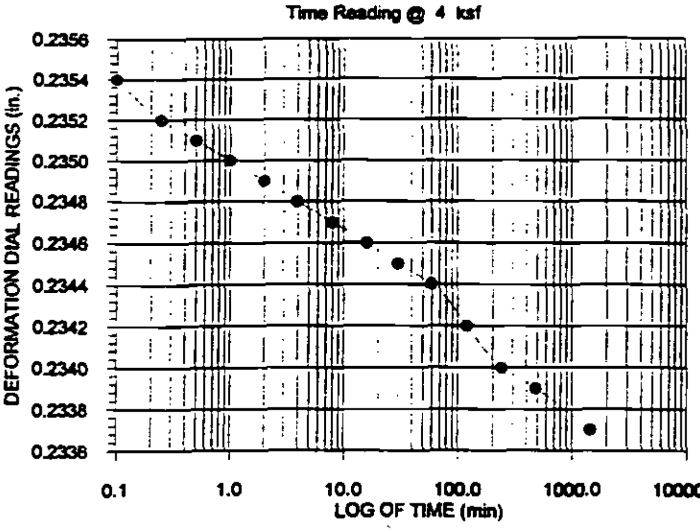
Project No.: 95-8347-14

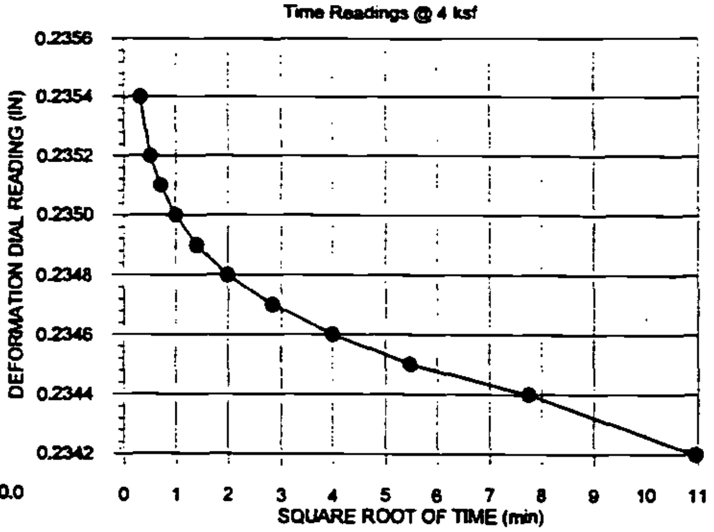
Eastside Extension
Metro Redline II

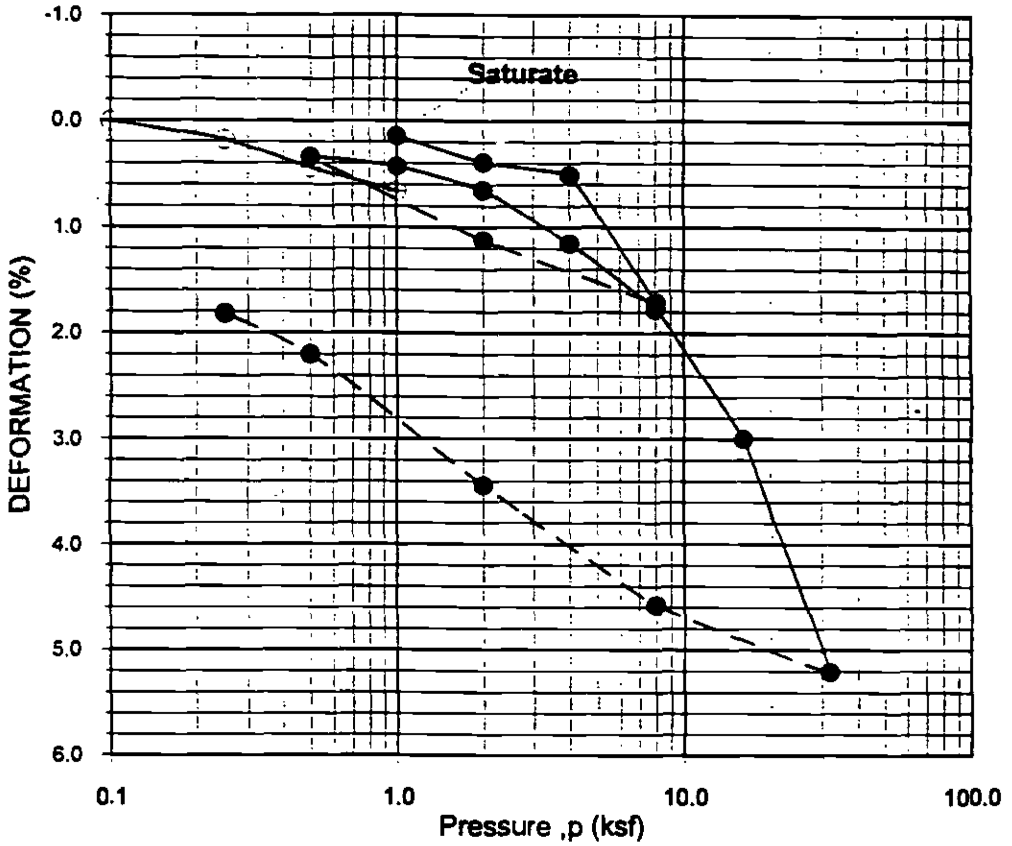
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-73


Time Reading @ 4 ksf


Time Readings @ 4 ksf



BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-18 C	D-21	94.0	18.6 / 20.1	108.8 / 109.9	0.549 / 0.521	91 / 100

SOIL DESCRIPTION: Yellowish Brown Lean Clay (CL)

GeoTransit Consultants

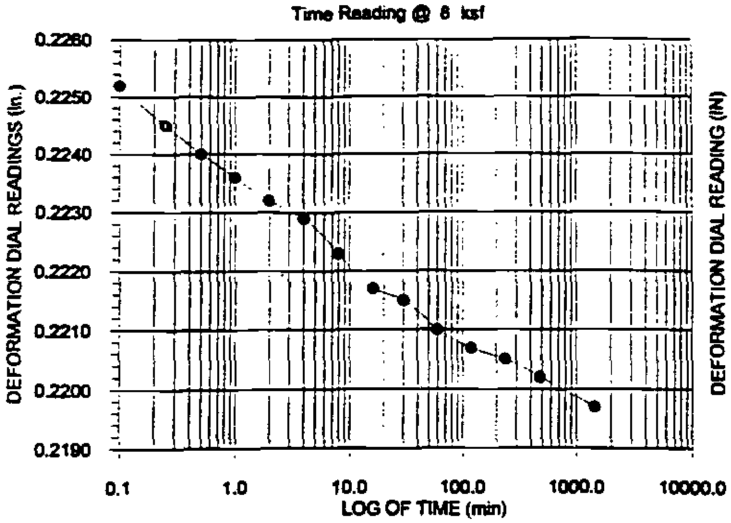
Project No.: 95-8347-14

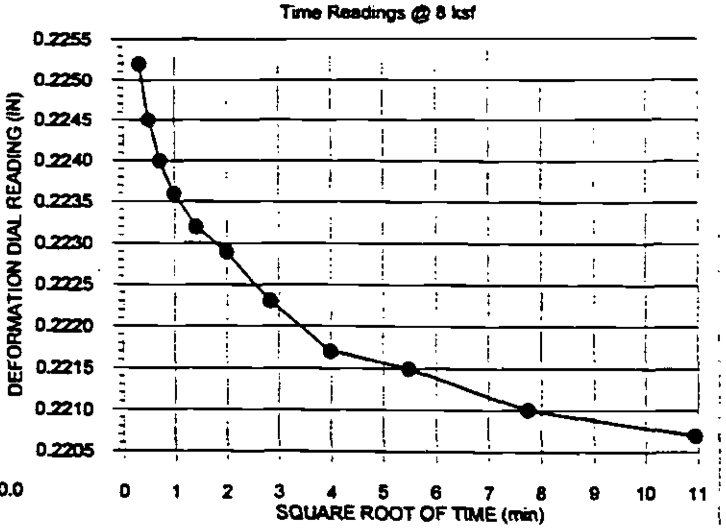
Eastside Extension
Metro Redline II

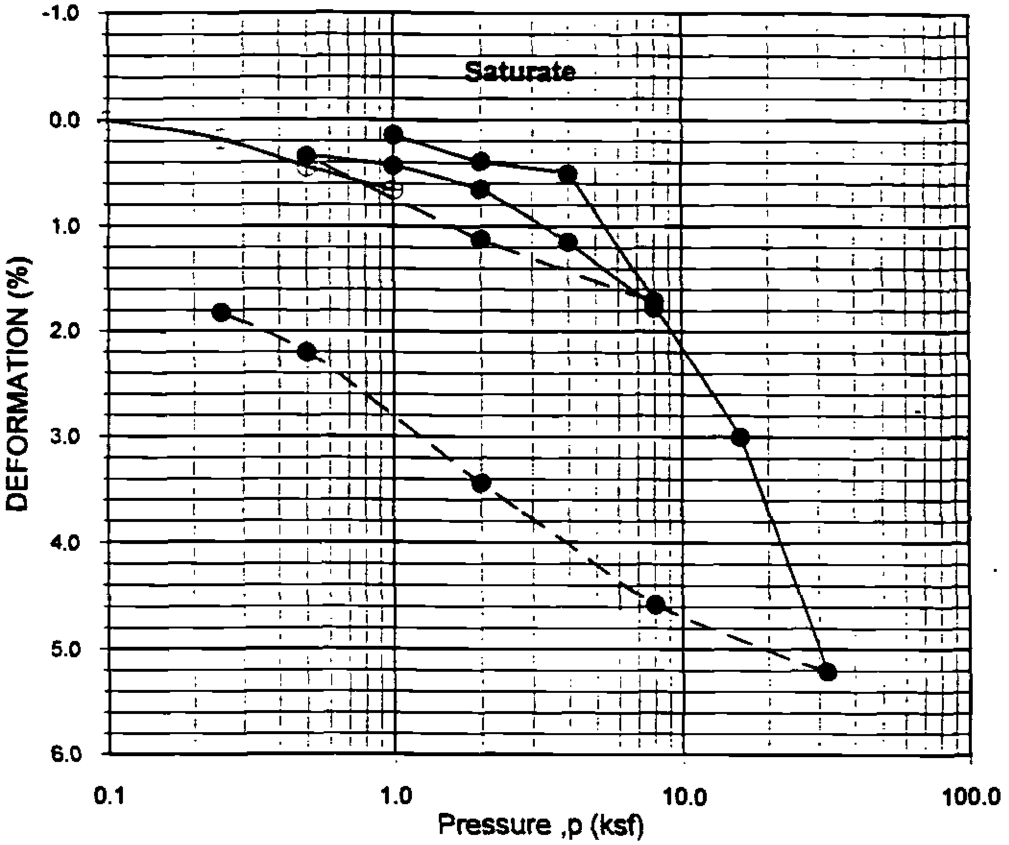
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-72a







BORING NO.	SAMPLE NO.	DEPTH (ft)	MOISTURE CONTENT (%) Initial / Final	DRY DENSITY (pcf) Initial / Final	VOID RATIO Initial / Final	DEGREE OF SATURATION (%) Initial / Final
SD-18 C	D-21	94.0	18.6 / 20.1	108.8 / 109.9	0.549 / 0.521	91 / 100

SOIL DESCRIPTION: Yellowish Brown Lean Clay (CL)

GeoTransit Consultants

Project No.: 95-8347-14

Eastside Extension
Metro Redline II

One-Dimensional Consolidation Properties of Soils (ASTM D 2435)

12-95

Figure B-72b

Laboratory Test Results

Measurement of Collapse Potential of Soils

(ASTM D5333)

PROJECT NAME:Metro Rail Redline – Eastside Extension (I, II,III) EARTH TECH NO.: 96–370–90006
CLIENT PROJECT NO.: 95–8347–24 CLIENT: EARTH TECH, INC. – Irvine
SUMMARIZED BY: sp DATE: February 22,1996 CHECKED BY: DATE: 2-23-96

BORING NO.: DD–62 Sample No. : D–4
Dry Density (pcf): 101.5 DEPTH(ft.): 20.0
Soil Description : Yellowish Brown Silty Sand (SM)
Initial Moisture (%): 19.4 Final Moisture (%) : 23.2
Initial Length (in.): 1.0000 Final Dry Density (pcf) : 102.0
Initial Dial Reading: 0.2569 Initial Void Ratio: 0.661
Diameter(in): 2.425 Specific Gravity : (assumed) 2.70
Initial Saturation (%) 79.2 Final Saturation (%) : 95.8

PRESSURE (p) (ksf)	FINAL READING (in)	APPARENT THICKNESS (in)	APPARATUS COMPLIANCE (%)	SWELL (+) SETTLEMENT (–) % OF SAMPLE THICKNESS	VOID RATIO	CORRECTED SWELL (+) SETTLEMENT (–) (%)
0.10	0.2567	1.0000	0.00	0.00	0.661	0.00
0.25	0.2547	0.9980	0.03	–0.20	0.658	–0.23
0.50	0.2514	0.9947	0.06	–0.53	0.653	–0.59
1.00	0.2449	0.9882	0.10	–1.18	0.643	–1.28
2.50	0.2325	0.9758	0.22	–2.42	0.625	–2.64
H2O	0.2308	0.9741	0.22	–2.59	0.622	–2.81

Swell / Collapse Potential, percent **–0.17**

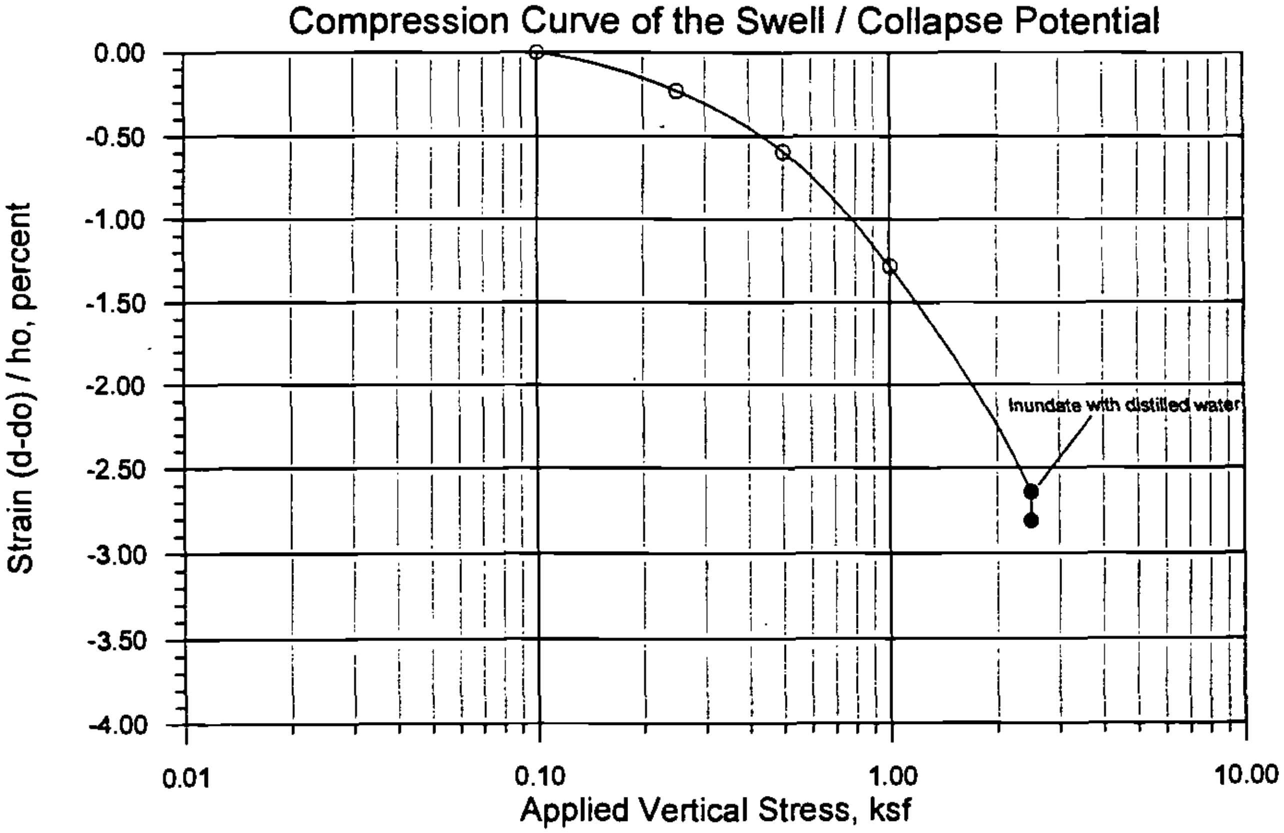


Figure B–73a

BORING NO.: DD-62 **Sample No. :** D-4

DEPTH(ft.): 20.0

Calculation

Sample Diameter(in):	2.425
Sample Thickness(in.):	1.000
Weight of Sample + ring(gm):	192.05
Weight of Ring(gm):	45.16
Height after consolidation(in.):	0.9719
Before Test	
Weight of Wet Sample+Cont.(gm):	111.23
Weight of Dry Sample+Cont.(gm):	102.53
Weight of Container(gm):	57.68
After Test	
Weight of Wet Sample+Cont.+ Ring (gm):	269.14
Weight of Dry Sample+Cont.+ Ring (gm):	241.31
Weight of Container(gm):	75.97
Vertical Rdg.(in): Initial	0.2569
Vertical Rdg.(in): Final	0.2308
Specific Gravity:	2.70
Water Density(pcf):	62.43
Wet density(pcf)	121.2

Final Saturation (%)	95.8
Final Dry Density (pcf)	102.0

TIME READING

Elapsed Time	Dial Rdgs.
0.10	0.2325
0.25	0.2324
0.50	0.2324
1	0.2324
2	0.2324
4	0.2324
8	0.2324
16	0.2324
30	0.2323
60	0.2322
120	0.2320
240	0.2318
1464	0.2311
3054	0.2308

0.100	0.000	
0.250	−0.230	
0.500	−0.590	
1.000	−1.280	
2.500	−2.640	
2.500		−2.810

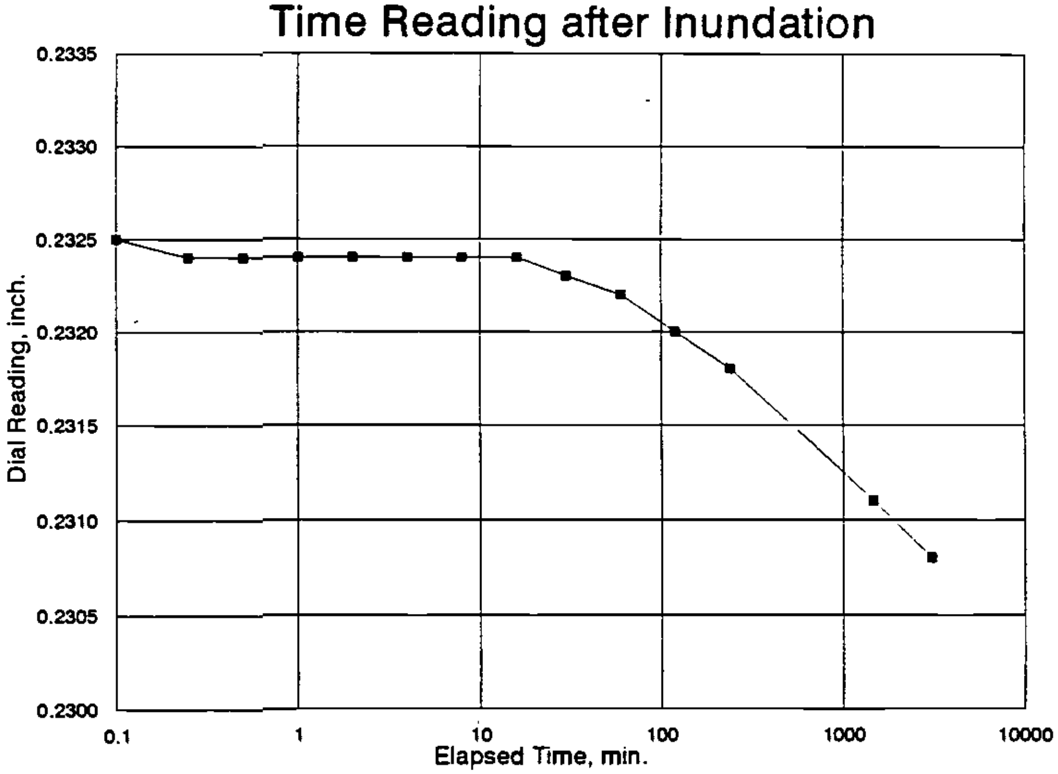


TABLE B-8 SATURATED HYDRAULIC CONDUCTIVITY

(TRIAXIAL CELL)
TEST PROCEDURE NO. ASTM D 5084/EPA 9100
FALLING HEAD METHOD

Project Name: Eastside Ext. - Metro Redline (II)
Project No: 95-8347-14
Boring No.: SD-15
Depth(ft): 70.0
Sample No.: SD-15
Sample Type: Drive
Sample Description: Yell Brwn, Sand (SP) with Grave

Cell Pressure: 121.8 psi
Bottom Pressure (Pb): 93.8 psi
Top Pressure (Pt): 91.8 psi
Consolidation Pressure: 30.0 psi
Burette Area (influent) (Ai): 0.401 sq. in.
Burette Area (effluent) (Ao): 0.396 sq. in.
Vol. Change During Consol.: 0.203 cu. in.

Initial Sample Height: 2.4783 in
Initial Area of Sample: 4.6696 sq. in.
Final Sample Ht.* (L): 2.4639 in
Final Sample Area (A)*: 4.6147 sq. in.
Tested by: Klat/PAA Date: 09/08/95
Input Checked by: LF Date: 09/11/95

* After Consolidation

Date	Time (min)	Incremental Elapsed Time (t) (min)	Temp. °C	Water Height Influent Burette (hi) (cm)	Water Height Effluent Burette (ho) (cm)	Uncorrected Hydraulic Conductivity(k) (cm/sec)	Corrected Conductivity at 20 °C (cm/sec)	Inflow Rate/ Outflow Rate	Remarks	
08-Sep-95	09:29 AM	0		27.5	2.5				Initial Reading	
08-Sep-95	09:29 AM	0.5	24.0	23.2	6.8	4.8E-04	4.4E-04	1.01	Average Last 4 rdgs=	4.0E-04
08-Sep-95	09:30 AM	0.5	24.0	19.6	10.6	4.3E-04	4.0E-04	0.96	Upper Limit=	5.0E-04
08-Sep-95	09:30 AM	0.5	24.0	16.2	14.3	4.4E-04	4.0E-04	0.93	Lower Limit=	3.0E-04
08-Sep-95	09:31 AM	0.5	24.0	12.8	17.6	4.3E-04	3.9E-04	1.04		
08-Sep-95	09:31 AM	0.5	24.0	9.5	20.9	4.5E-04	4.1E-04	1.01		
08-Sep-95	09:32 AM	0.5	24.0	6.5	23.9	4.3E-04	3.9E-04	1.01		

$k = Ai.Ao.L.\ln(h1/h2)/(A.t.(Ai+Ao))$ where h1, h2= ((Pb−Pt)/Y+(hi−ho) at t0−(change in hi + change in ho) at t1 and t2 Y = Water Density

TABLE B-9 SATURATED HYDRAULIC CONDUCTIVITY

(TRIAXIAL CELL)
TEST PROCEDURE NO. ASTM D 5084/EPA 9100
FALLING HEAD METHOD

Project Name:	Eastside Ext.-Metro Redline (II)	Cell Pressure:	132.5 psi	Initial Sample Height:	3.0083 in	
Project No:	95-8347-14	Bottom Pressure (Pb):	74.5 psi	Initial Area of Sample:	4.6186 sq. in.	
Boring No.:	DD-28	Top Pressure (Pt):	72.4 psi	Final Sample Ht.* (L):	2.9701 in	
Depth(ft):	73.0	Consolidation Pressure:	60.0 psi	Final Sample Area (A)*:	4.4997 sq. in.	
Sample No.:	D-15	Burette Area (influent) (Ai):	0.361 sq. in.	Tested by:	TM	Date: 10/18/95
Sample Type:	Drive	Burette Area (effluent) (Ao):	0.374 sq. in.	Input Checked by:		Date: 10/23/95
Sample Description:	Yellow Brown, Silt,(ML) Stiff	Vol. Change During Consol.:	0.530 cu. in.			

* After Consolidation

Date	Time (min)	Incremental Elapsed Time (t) (min)	Temp. °C	Water Height Influent Burette (hi) (cm)	Water Height Effluent Burette (ho) (cm)	Uncorrected Hydraulic Conductivity(k) (cm/sec)	Corrected Conductivity at 20 °C (cm/sec)	Inflow Rate/ Outflow Rate	Remarks	
20-Oct-95	08:44 AM	0		27.5	2.2				Initial Reading	
20-Oct-95	09:01 AM	17	22.9	26.2	3.5	4.6E-06	4.3E-06	0.97	Average Last 4 rdgs=	5.0E-06
20-Oct-95	09:14 AM	13	22.9	25.1	4.6	5.2E-06	4.9E-06	0.97	Upper Limit=	6.2E-06
20-Oct-95	09:27 AM	13	22.9	23.9	5.8	5.7E-06	5.4E-06	0.97	Lower Limit=	3.7E-06
20-Oct-95	09:54 AM	27	23.0	21.9	7.7	4.6E-06	4.3E-06	1.02		
20-Oct-95	10:07 AM	13	23.0	21.0	8.6	4.5E-06	4.2E-06	0.97		
20-Oct-95	10:22 AM	15	23.1	19.9	9.7	4.8E-06	4.5E-06	0.97		
20-Oct-95	10:39 AM	17	23.1	18.7	10.8	4.5E-06	4.2E-06	1.05		
20-Oct-95	10:53 AM	14	23.1	17.7	11.8	4.8E-06	4.5E-06	0.97		
20-Oct-95	11:27 AM	34	23.2	14.9	14.5	5.6E-06	5.2E-06	1.00		
20-Oct-95	11:43 AM	16	23.2	13.5	15.8	6.0E-06	5.6E-06	1.04		
20-Oct-95	11:57 AM	14	23.2	12.5	16.8	5.2E-06	4.8E-06	0.97		

k=Ai.Ao.L.ln(h1/h2)/(A.t.(Ai+Ao)) where h1, h2= ((Pb-Pt)/Y+(hi-ho) at t0-(change in hi + change in ho) at t1 and t2 Y = Water Density

APPENDIX C
EARTHWORK GUIDELINES

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Earthwork and site preparation activities will encompass clearing and grubbing, subgrade preparation for pavements and structural support, excavation for foundations, utility line relocations, and fill placement. The following guidelines are provided for earthwork associated with the First/Lorena station with crossover.

Clearing and Grubbing

Fill material, trash and any vegetation should be removed from the site area to be graded. Subsurface conditions at the site may differ from those observed in the borings and, therefore, a qualified geotechnical engineer should inspect and approve the prepared graded area prior to the placement of fill.

Excavations

Minor excavations at the site will include those for utility lines and near-surface foundations. Major excavations will be for the main station housing the rail facilities. Recommendations for major temporary excavations are presented in Section 5.5 of this report. Temporary shallow excavations to a depth of 5 feet can be expected to remain stable with 1H:1V (one horizontal to one vertical) slopes for short periods. Temporary excavations deeper than 5 feet should be shored or sloped back at 1-1/2H:1V. Recommendations for sloped excavations deeper than 5 feet are presented in Section 5.5 of this report. Due to the granular nature of the alluvium, raveling and running conditions should be anticipated.

Fill Materials and Placement

Fill materials may consist of excavated onsite granular soils or approved clean granular soils free of expansive materials, debris, organic contaminants or rock fragments larger than 6 inches.

The following specifications are recommended for fill placement.

- All areas that are to receive compacted fill shall be observed and approved by a geotechnical engineer
- Prior to fill placement, exposed soil surfaces shall be scarified to a depth of 6 inches, moisture conditioned as necessary and compacted to at least 90 percent of the maximum dry density as determined by procedure ASTM D1557

- All fill materials shall be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to a moisture content 1 to 3 percent above optimum moisture content and mechanically compacted to at least 90 percent of the maximum dry density as determined by ASTM procedure D1557. Each lift should be compacted before the next lift is placed.

- A geotechnical engineer or an experienced soils technician shall observe fill placement and compaction and conduct in-place field density tests on the compacted fill to verify the adequacy of compaction.

In deep fill areas or fill areas, and structure foundation subgrade areas, the compaction requirement shall be increased to 95 percent of the maximum dry density (ASTM D1557), as directed by a qualified geotechnical engineer.

Subgrade Preparation

Subgrade for shallow footings shall be overexcavated to a minimum depth equal to the footing width and replaced with properly compacted fill. Overexcavation shall extend a minimum distance equal to half the width or 2 feet, whichever is larger, beyond the perimeter of the footing. Fill shall be compacted to at least 95 percent of the maximum dry density as determined by ASTM Procedure D-1557.

In the absence of specific information on loads, dimensions and locations of structures, Figure 5-6 is provided as a general guideline for foundation support needs. Structure specific foundation recommendations should be provided when such information becomes available.

The mat foundation shall be underlain by a minimum 2-foot-thick dense granular material with an in situ density of at least 95 percent of maximum dry density as per ASTM D-1557. In granular alluvium this may be achieved by proof rolling the excavated subgrade with a heavy vibratory roller or by overexcavating and compacting in layers, the 2-foot zone below the foundations. In fine grained alluvium, the exposed subgrade shall be overexcavated at least 2 feet below the design grade and replaced with fill compacted to project specifications. If the mat is directly placed on the native materials, rock fragments larger than 6 inches should not be allowed in the exposed subgrade. The compacted fill blanket should extend a minimum of 5 feet beyond the foundation perimeter.

Overexcavations and susbgrade preparation shall be observed by the Geotechnical Engineer. Based on exposed field conditions, overexcavation depths may be increased by the Geotechnical Engineer.

Site Drainage

Adequate positive drainage should be provided away from the surface structures to prevent water from ponding and to reduce water percolation into subsoils. If there is any granular backfill directly underneath the area of surface drainage, the backfill should be covered or capped with at least 18 inches of relatively impervious compacted fill. Fine grained alluvium excavated at the site will be suitable for this purpose.