Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A350

BY

CONVERSE CONSULTANTS, INC.
EARTH SCIENCES ASSOCIATES
GEO/RESOURCE CONSULTANTS

MAY 1984

Funding for this Project is provided by grants to the Southern California Rapid Transit District from the United States Department of Transportation, the State of California and the Los Angeles County Transportation Commission.

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Attention: Mr. B.I. Maduke, Senior Geotechnical Engineer

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A350 prepared in accordance with our Contract No. 503 agreement dated September 30, 1983 between Converse Consultants, Inc. and Metro Rail Transit Consultants (MRTC). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A350.

Our study team appreciate the assistance provided by the MRTC staff, especially Bud Maduke. We also want to acknowledge the efforts of each member of the Converse team, in particular Fred Chen and Jim Doolittle.

Respectfully submitted,

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RMP:n

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PROFESSIONAL CERTIFICATION

This report has been prepared by CCI/ESA/GRC under the professional supervision of the principal soils engineer and engineering geologist whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions are presented, within the limits prescribed by the client, after being prepared in accordance with generally accepted professional engineering and geologic principles and practice. There is no other warranty, either express or implied.

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation and engineering analyses for the A350 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A350 Design Unit consists of the Hollywood/Cahuenga Station and Pocket Track. The structures will be constructed by cut-and-cover methods and will extend to depths of about 45 to 80 feet below the existing ground surface. This report defines the subsurface conditions and provides recommendations for design and construction purposes for facilities shown on SCRTD drawings dated 6-10-83.

1.1 CONSTRUCTION CONSIDERATIONS

Construction of the Hollywood/Cahuenga Station and Pocket Track will require shoring and lagging to support the exposed alluvial soils. Current ground water elevations increase northward and are above the planned subgrade elevation in the Pocket Track section. Dewatering along the north half of the cut-and-cover excavation will be required.

Temporary support of the excavation will be either flexible or rigid type vertical wall systems with internal bracing or external tieback systems. Caving and ravelling of the coarse-grained alluvial soils should be expected during soldier pile and/or tieback construction. Consideration should be given to alternatives which would reduce the number of tiebacks penetrating into the caving soils (such as full or partial internal bracing). Lateral pressures and other guidelines for design of temporary support systems are provided in the report.

1.2 DESIGN CONSIDERATIONS

A prime consideration for design is the presence of the Hollywood Fault zone which crosses the proposed Pocket Track structure in the vicinity of Yucca Street. The Hollywood Fault, according to geologic reports referenced in Section 5.4, is considered to be an active fault and capable of significant displacement during a maximum design earthquake event. Effects of fault movement on the structure should be carefully studied and design concepts developed to accommodate such movements if possible.

The undisturbed alluvium and Fernando Formation bedrock will adequately support the permanent reinforced concrete station structure. However, the Hollywood Fault zone creates a discontinuity in the subgrade material upon which the Pocket Track section will be supported. This condition should be carefully studied, and design concepts developed to mitigate static longitudinal elastic differential settlements along the structure.

Design lateral pressures for the permanent structure under varying earth and hydrostatic loading conditions are outlined in the text of the report.

1.3 SEISMIC CONSIDERATIONS

Liquefaction evaluation based on field correlations of SPT results and performance of granular soils indicate that liquefaction of the granular soils at the site during a maximum design earthquake has a low probability.

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD 1984 report entitled "Guidelines for Seismic Design of Underground Structures". Seismological conditions which may impact the project and the operating and maximum design earthquakes which may be anticipated in the Los Angeles area are described in the SCRTD report entitled "Seismological Investigations and Design Criteria" dated May, 1983. The 1984 report complements and supplements the 1983 report. Site specific static and dynamic properties for materials in design unit A350 are given in this report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for the A350 Design Unit which consists of the Hollywood/Cahuenga Station and Pocket Track. The work performed for this report includes borings, laboratory tests, engineering analysis, and the development of recommendations and general earthwork specifications for design and construction of the station. This Design Unit is a part of the 18.6-mile long Metro Rail Project (see Drawing 1, Vicinity Map).

Additional geotechnical information on the Metro Rail Project is included in the following reports, some of which may pertain to Design Unit A350.

- "Geotechnical Investigation Report, Metro Rail Project", Volume I - Report, and Volume II - Appendices, prepared by Converse Ward Davis Dixon, Earth Sciences Associates and Geo/Resource Consultants, submitted to RTD in November 1981. This report presents general geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experience and practices in the Los Angeles area.

- "Seismological Investigation & Design Criteria Metro Rail Project", prepared by Converse Consultants, Lindvall Richter & Associates, Earth Sciences Associates and Geo/Resource Consultants, submitted to RTD in May 1983. This report presents the results of a seismological investigation.

- "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF866, 1977), prepared by the U.S. Geological Survey in cooperation with the U.S. Department of Transportation. This publication includes a compilation of geotechnical data in the general vicinity of the proposed Metro Rail Project and this Design Unit.

- "Rapid Transit System Backbone Route", Volume IV, Book 1, 2 and 3, prepared by Kaiser Engineers, June, 1962 for the Los Angeles Metropolitan Transit Authority. This report presents the results of a Test Boring Program for the Wilshire Corridor and logs of borings.

The design concepts discussed in this report are based on the "Final Report for the Development of Milestone 10, CBD to North Hollywood Line Plans, dated September 1983; and Preliminary Site Plans, and Structure Plans and Sections for Hollywood/Cahuenga Station and Pocket Track, dated March, June and July 1983.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

The Hollywood/Cahuenga Station and Pocket Track will be located off-street running north-south along the west side of Cahuenga Boulevard from a point just south of Hollywood Boulevard up to Franklin Avenue. The station area is in the commercial center of Hollywood. The development along Hollywood Boulevard is low- to medium-rise commercial with a number of theaters. A mixture of commercial and industrial buildings is located on Cahuenga Boulevard. North of Hollywood Boulevard and west of Cahuenga are high density residential areas.

The Hollywood/Cahuenga Station has been planned with two entries, one on the northwest and one on the southwest corner of Hollywood and Cahuenga. An area immediately to the south end of the station is planned for use as a bus turnaround and layover area. The pocket track will be located at the north end of the station. Both the station and the pocket track will be constructed by the cut-and-cover methods which will result in the removal of some of the existing structures facing on Cahuenga Boulevard between Hollywood Boulevard and Yucca Street.

The station is planned with a single mezzanine connecting the two station entries. Ancillary space will be provided at each end of the station, and a traction power substation will be located above the north third of the pocket track structure.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The information presented in this report is based primarily on the field and laboratory investigations performed in 1981 and 1983. This information was derived from field reconnaissance, borings, geologic reports and maps, ground water measurements, field geophysical surveys, ground water quality tests, and laboratory tests on soil and rock samples. References listed at the end of this report were utilized to complement and supplement the more recent information.

4.2 BORINGS

For the A350 investigation, 15 borings were drilled at the proposed station and pocket track structure site. Small diameter rotary wash holes 28-1 through 28-8, 28B, 29A and 29B, and the 32-inch diameter man-size auger boring, 28-C were all drilled in 1983. Rotary borings CEG-28, CEG-28A and CEG-29 were drilled in 1981 and their logs are also included. The locations of the borings are shown on Drawings 2 and 3, and the logs of the borings from the 1981 and 1983 investigations are provided in Appendix A. Ground water observation wells were installed in Borings 28, 28-A, 28-B, 28-C, 28-5, 29, 29-A and 29-B. Section 5.3 presents a summary of ground water level measurements in these wells.

None of the 1962 Kaiser Engineers borings were drilled close to the Hollywood/Cahuenga Station and Pocket Track structure area. Another source of boring information is the U.S. Geological Survey paper, "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF-866, 1977). However, the foundation investigation borings included in the USGS report were not used because they were too shallow for proper interpretation of subsurface conditions at the proposed grade of the station and pocket track.

4.3 GEOPHYSICAL MEASUREMENTS

Seismic refraction surveys were performed in the vicinity of the station and pocket track structure site during the 1981 investigation. Results of these surveys are presented in Appendix B.

Downhole and crosshole compression and shear wave velocity surveys were also performed in Boring CEG-28 which was drilled during the initial 1981 investigation. The CEG-28 boring was drilled on the east side of Cahuenga Boulevard at the A350 Station site. Appendix B summarizes the field geophysical survey procedures as well as the results of the velocity measurements.

4.4 GEOTECHNICAL LABORATORY TESTING

The laboratory program developed to test representative soil and rock samples consisted of classification tests, consolidation tests, static and dynamic triaxial compression tests, resonant column tests, unconfined compression tests, direct shear tests, and permeability tests.

Appendix C summarizes the testing procedures and presents detailed results of the 1983 program and summarizes selected results of the 1981 laboratory program.

4.5 WATER QUALITY ANALYSES

Chemical analyses were performed and selected parameters were evaluated for water samples obtained in Borings CEG-28A and CEG-29. The results of these tests are presented in Appendix D.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

The geologic sequence in the Hollywood/Cahuenga Station and Pocket Track structure site consists of Young and Old alluvium overlying bedrock of the Topanga Formation. The east-west trending Hollywood Fault crosses the alignment underneath the Pocket Track structure, and is judged to be active, according to geologic reports referenced in Section 5.4.

Drawings 2 and 4 show generalized subsurface cross sections through the proposed Hollywood/Cahuenga Station and Pocket Track structure. The subsurface profile at the Station site consists of approximately 55 to 65 feet of alluvium at the north end of the Pocket Track structure area, and over 200 feet of alluvium in the south end of the Pocket Track and Station site. The bedrock surface at this Station and Pocket Track site is discontinuous due to the offset caused by the Hollywood Fault zone.

5.2 SUBSOILS

During the field programs conducted in both 1983 and 1981, the contact between the Old and Young Alluvium was difficult to identify because the soils of these two units are physically very similar. While the Young and Old Alluvium may be geologically different, our interpretation of the field and laboratory test data suggests that they do not differ significantly from an engineering standpoint. For the purposes of this report, Young and Old Alluvium have not been differentiated and are simply referred to as Alluvium.

Specific descriptions of the soil materials encountered in the borings drilled at the Station site include:

- Fill: Sandy fill soils were encountered below surface pavements in five of the borings drilled at the site. The fill thickness ranged between 0.5 and 4 feet. The fill generally consisted of relatively clean (no debris) silty sand or silty clay which were medium dense and stiff, respectively.

- Alluvium: The alluvial soils encountered at the boring locations generally consisted of a mixture of coarse- and fine-grained soils to depths of 50 to 65 feet underlain by predominately coarse-grained soils. Near surface alluvium was predominately granular, medium dense to dense, consisting of sands, clayey sands and gravelly sands to depths up to 15 feet. The underlying soils were mixtures of clay and sand and were generally classified medium dense to dense clayey sand with some firm to stiff sandy clay. Coarse-grained alluvial soils encountered below depths of 50 to 65 feet included dense to very dense sands, gravelly sands and sandy gravel materials. This material may also contain zones of cobbles and boulders although none were encountered.

5.3 BEDROCK

Approximately the northernmost 500 feet of the proposed construction is expected to be underlain by the Topanga Formation bedrock at relatively shallow depths of about 50 to 60 feet. South of the middle trace of the Hollywood Fault (at about the Yucca Street crossing), the depth to bedrock increases abruptly to about 135 feet. South of the southern trace of the Hollywood Fault Zone, the bedrock depth increases abruptly to greater than 206 feet (maximum depth of Boring 28-B).

The Topanga Formation bedrock encountered at this site was predominately sandy siltstone, claystone and silty sandstone with localized intrusions of basalt. Borings 28-A, 29 drilled during the 1981 investigation and Borings 28-1 and 28-8 drilled in 1983 were the borings in the site vicinity to have significant penetration into the Topanga Formation bedrock. Bedrock encountered in Boring 28-A consisted of silty claystone with interbeds of sandstone. The upper 40 to 50 feet of bedrock at 28-A was very weathered and soil-like; bedding was measured to be between 60° and 84°. At Boring 29, the bedrock consisted of about 9 feet of very weathered siltstone/claystone bedrock underlain by sandy siltstone with sandstone interbeds for more than 160 feet. Bedrock encountered in Borings 28-1 and 28-8 was thinly bedded sandy siltstone to the depths of the borings. The bedrock surface slopes down gently to the south and east. Current information from a few nearby surface outcrops indicates steeply dipping bedding which incline to the north-northeast.

5.4 HOLLYWOOD FAULT

The most striking feature shown on the geologic sections is the subsurface discontinuity due to the Hollywood fault zone. The trace of the Hollywood fault zone is located between Stations 758± to 764± at Pocket Track grade, approximately 600 feet wide. This fault is judged to be active; i.e., there is evidence of displacement at or near the ground surface at least once within the past 10,000 years (Holocene time). This opinion is based on:

- Interpretation of Bouger Gravity and Density Model Profile 5 showing a vertical bedrock offset of about 400 feet along a thrust feature dipping about 50° to the north (Converse, et al, "Geotechnical Investigation Report, Volume II, Appendices", November 1981, p. II-714, and Figure No. D-5, p. II-721, prepared for SCRTD).
- Alignment of 2- to 3-meter high scarp-like features in the Hollywood, Los Feliz, Atwater area of Los Angeles; i.e., have offset very late Quaternary (including Holocene) alluvial sediments (Weber, et al, "Earthquake Hazards Associated with the Verdugo-Eagle Rock and Benedict Canyon Fault Zones, Los Angeles County, California", 1980, OFR 80-10LA, p. A-3, B-104, B-105 and B-106.
- Interpretation of Borings 28-B, 28-C, 28-2, CEG-28-A, 28-1 and 28-8 drilled by Converse for SCRTD and MRTC in 1981, 1983 and 1984.

A 1983 study to establish the date of the last movement on the Hollywood fault was inconclusive because the fault, where observed in granite bedrock, was not overlain by datable alluvium (Crook, R., Proctor, R.J. and Lindvall, C.E., "Seismicity of the Santa Monica and Hollywood Faults Determined by Trenching", February 1983, U.S. Geological Survey Contract No. 14-08-0001-20523).

The approximate 600-foot width of the fault zone is based on interpretation of alluvium and bedrock contacts from our 1981, 1983 and 1984 borings.

The seismic characteristics of the Hollywood fault are discussed in the "Seismological Investigation & Design Criteria" report of May, 1983 prepared by Converse et al for SCRTD. This report assigns a maximum design earthquake (Richter magnitude of 6.5M to the Hollywood Fault. Although there is a low probability of this event (and attendant displacement) on the Hollywood fault, during the estimated 100-year life of the facility, such a potential event requires consideration in the design of the structure.

5.5 GROUND WATER

Ground water levels in the site vicinity were measured in piezometers installed at Borings 28-A, 28-B, 28-5, 28-8, 29, 29-A, 29-B and 29-C. In addition, water levels were measured in Boring 28 and man-size auger Boring 28-C at the time they were drilled. The results of the ground water measurements are summarized in Table 5-1. Based on the results of these measurements it appears that current ground water levels slope southward across the site at an average gradient of about 4% which is steeper than the average ground surface gradient (about 2%). Current water levels vary from about elevation 380 in Borings 29-A and 29-B (at the north end of the pocket track) to about elevation 305 at the south end of the station. Drawings 2 and 5 show that these current water levels range from about 40 feet above subgrade at the north end of the site to about 30 feet below subgrade at the south end of the site.

TABLE 5-1

GROUND WATER OBSERVATION WELL DATA

			GROUND WATER ELEVATION*					
	Initial							
BORING	Reading	Date	04/28/82	02/18/83	03/02/83	12/07/83	12/20/83	03/14/84
28	310	01/12/81						
28-A	357	03/23/81	365					
28-B	329	02/18/83		329	336	351	352	351
28-C	354	10/10/83						
28-5	dry	11/19/83				318	311	309
28-8	377	02/24/84						376
29	344	03/23/81	374					
29-A	379	02/15/83			<380			<380
29-B	391	02/14/83			390			383
29-C	438	02/10/83			440		434	432

*Rounded to the nearest foot

Except for man-size auger Boring 28-C and 28-3, no gas odors or unusual ground water conditions were noted during the field exploration.

Borings 28-C, 28-1, 28-8 encountered gasoline floating on top of the drilling fluid and ground water (52 feet below the ground surface at Elevation 354 feet in Boring 28-C). The gasoline concentration was 5,500 parts per million (ppm), and the general mineral analysis of Boring 28-C water indicates Total Dissolved Solids (TDS) of 1,600 ppm (see CCI Memorandum dated October 10, 1983). The refined gasoline could pose a danger during construction of the Pocket Track. The source of the gasoline is believed to be an abandoned and corroded gasoline storage tank in this general area. A strong petroleum odor was also detected in the soil sample No. C-17 in Boring 28-3 at a depth of 83 feet.

5.6 GAS

No gas analyses were made at this site; however, sulphur and/or petroleum odors from the soil and bedrock samples were noted in borings 28-C, 28-1, 28-3 and 28-8. Combustible gas may be present at the Pocket Track structure site, and caution is recommended during construction in this area.

5.7 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

For purposes of our engineering evaluation, the subsurface materials were grouped into general subsurface units. The main subsurface units affecting design of the Hollywood/Cahuenga Station include the sand/clay mixture alluvium , coarse-grained sand/gravel alluvium and Topanga Formation bedrock. Surficial fill soils encountered were considered to be too thin to have any significant effect on design.

The following presents engineering descriptions of each of the three main subsurface materials and engineering parameters assigned to these units for. our analyses (see Table 5-2).

- <u>Mixed Clay/Sand alluvium</u>: These materials were predominately classified as clayey sand with low to moderate plasticity. However, in some areas, the materials grade to sandy clay, exhibiting moderate to high plasticity, or to silty sand (no plasticity). Standard Penetration Test (SPT) results ranged from 7 to 70 in this unit but averaged about 30. Laboratory density test results indicated dry densities generally in the range of 100 to 110 pcf. Strength test results showed moderate effective strength values. Low initial densities caused positive pore pressures to be generated during shearing, resulting in relatively low undrained or total strength values. Consolidation test results indicate the near-surface materials in this unit range from moderately to highly compressible. Undrained moduli from triaxial testing were generally low but did exhibit some increase with consolidation pressure. Selected engineering design properties for the clay/sand unit are presented in Table 5-2.

TABLE 5-2

MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	FILL	MIXED SAND/CLAY ALLUVIUM	GRANULAR ALLUVIUM	TOPANGA BEDROCK
Moist Density Above Ground Water (pcf)	125	125	125	-
Saturated Density (pcf)	-	130	130	130
Effective Stress Strength				
ø' (degrees)	-	30	35	28
c' (psf)	-	0	0	0
Total Stress Strength[a]				
ø (degrees)	-	15	-	15
c (psf)	-	500	-	2000
Unconfined Compressive Strength (psf)	-	-	-	b
Permeability (cm/sec)	-	10^{-3} to 10^{-6}	10^{-1} to 10^{-4}	10^{-3} to 10^{-7}
Vertical Compression Modulus[c] (psf)	-	$150 \cdot \sigma_{v'}$[d]	$450 \cdot \sigma_{v'}$[d]	1×10^6 to 2×10^6
Poisson's Ratio (non-saturated)	-	0.40	0.35	0.35

[a] The total stress parameters should be used to determine the increase in undrained strength with depth for use in undrained strength analyses.

[b] All unconfined compressive strength bedrock test specimens failed along bedding. The average along-bedding strength was 2000 psf. However, across-bedding compressive strength is expected to be higher.

[c] Modulus values are secant modulus at 1/2% strain.

[d] $\sigma_{v'}$ is the effective overburden pressure (psf) equal to effective density times overburden depth. Moist density should be used to determine $\sigma_{v'}$ above the water table and submerged density (saturated density minus water density) should be used for the effective density of soils below the water table.

- Granular Alluvium: These materials were generally encountered at depths greater than 50 feet and were classified as dense to very dense sands, silty sands, gravelly sands and gravels. Gravel content was so high in Boring 28-6 that sampling became nearly impossible below a depth of 55 feet. Density tests on samples from this unit generally indicated dry densities ranging from about 110 to 120 pcf. Strength test results showed high effective stress strength parameters with friction angles as high as 44°. Undrained modulus values from triaxial tests exhibited rapid increase with consolidation pressure. Permeability tests performed on sand and silty sand specimens from this unit indicated permeabilities on the order of 10^{-3} to 10^{-4} cm/sec; however, the permeability of gravelly soils is considered to be 10^{-1} to 10^{-2} cm/sec. Selected engineering design properties for the coarse-grained alluvium are presented in Table 5-2.

- Topanga Formation Bedrock: Laboratory testing of the Topanga Formation bedrock for this study included the unconfined compression tests performed during the 1981 investigation, and the triaxial and unconfined compression tests performed in 1984 on samples obtained from Borings 28-1 and 28-8. Due to the very steep bedding of the bedrock, vertical compression tests (triaxial and unconfined compression tests) tend to fail along the bedding planes instead of across bedding. Therefore, the selected across-bedding strength parameters presented in Table 5-2 were based on consideration of the direct shear test results and strength test results of Topanga Formation from other nearby design units. Due to the tendency of compression test specimens from this site to fail along bedding, the measured modulus values were considered to be lower than in situ values. Therefore, the range of modulus values presented in Table 5-2 was also based on consideration of measured modulus values from other design units combined with engineering judgement. Permeability tests performed on the thinly bedded material indicated that the siltstone beds to have low permeability (10^{-5} cm/sec), the claystone interbeds to have very low permeability (10^{-6} to 10^{-7}), and the sandstone interbeds to have significantly higher permeability (10^{-3} cm/sec).

Section 6.0

Geotechnical Evaluation and Design Criteria for Station and Pocket Track

6.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA FOR STATION AND POCKET TRACK

6.1 GENERAL EVALUATION

Construction of the Hollywood/Cahuenga Station and Pocket Track will involve an excavation through alluvial soils and bedrock to depths of 45 to 80 feet below the existing ground surface. The excavation will require shoring and lagging to support the exposed alluvial soils. Current ground water elevations increase northward and are above the planned subgrade elevation in the Pocket Track section. Dewatering will, therefore, be required to lower ground water levels up to about 40 feet at the north end of the Pocket Track section.

A primary consideration for design is the presence of the Hollywood Fault zone which crosses the proposed Pocket Track structure in the vicinity of Yucca Street. The Hollywood Fault is considered to be an active fault and capable of significant displacement during a maximum design earthquake event. The approximate location of the Hollywood Fault zone is shown on Drawings 2 and 4. However, it should be noted that the actual location and width of the fault zone and/or "zone of disruption" may extend beyond limits shown on Drawings 2 and 4.

The Hollywood Fault zone also creates a discontinuity in the subgrade materials upon which the Pocket Track section will be supported (see Drawing 5). Within the fault zone, the subgrade materials will vary from bedrock to deep alluvium. The differences in elastic properties of the alluvium and bedrock may cause differential settlements. Geotechnical solutions to this condition are limited to attempting to "smooth" the transition between materials by partial overexcavation of the bedrock. Design concepts should consider the potential for differential settlements which may approach 2 inches or more across this discontinuity.

The presence of the shallow bedrock north of the fault should not have a significant impact on shoring design. Installation of soldier piles and/or tieback anchors into the bedrock should not be unusually difficult, except where cemented sandstone or basalt intrusions are encountered.

Caving and ravelling of the coarse-grained alluvial soils should be expected during soldier pile and/or tieback construction. Consideration should be given to alternatives which would reduce the number of tiebacks penetrating into the caving soils (such as full or partial internal bracing).

The following subsections present more detailed evaluations and recommendations for design and construction of the Hollywood/Cahuenga Station and Pocket Track.

6.2 EXCAVATION DEWATERING

6.2.1 General

Dewatering will generally be required for construction of the Pocket Track section. However, no dewatering is anticipated for construction of the station at the southern portion of the site based on the reported water levels. Current ground water levels range from 0 to 40 feet above the proposed subgrade of the northern pocket track section and 40 to 60 feet below the ground surface (see Drawings 2 and 5).

Much of the pocket track will be in the Topanga Formation bedrock. We expect that the dewatering system will not be able to draw water levels down significantly below the bedrock surface. Therefore, dewatering in this section would require internal sumps as well as a perimeter pumping system since some water would escape perimeter pumps and flow into the excavation along the bedrock surface. A possible dewatering system might consist of the following:

- deep wells and/or ejector wells placed around the perimeter of the excavation. Where the bedrock surface is above the planned subgrade, the wells should penetrate to the bedrock surface. Where the subgrade is underlain by alluvium, the wells should penetrate below the subgrade.
- supplementary ditch drains and sumps would be added within the bedrock portion of the excavation to control flow into the excavation along the bedrock surface.

6.2.2 Criteria for Dewatering Systems

It is understood that the contractor will be responsible for designing, installing, and operating a suitable construction dewatering system subject to review and acceptance by the Metro Rail Construction Manager. The system should satisfy the following criteria:

- The dewatering system should be installed and in operation for a sufficient period prior to the excavation reaching the level of static ground water level to adequately drawdown the ground water table. This period is a function of the maximum pumping capacity installed.
- The system should maintain the ground water levels low enough to prevent piping of the alluvial soils into the excavation. Inflow quantities should be reduced to levels which can be handled by a drain/sump system and allow excavation and construction to proceed.
- Wells must be designed and developed to eliminate loss of ground from piping of soils from around the wells. The well operations should be constantly monitored for evidence of piping.
- The system should maintain water levels low enough to assure the stability of the bottom of the excavation at all times during construction.
- The system should be operated continuously. Emergency power and backup pumps should be required to ensure continual excavation dewatering.

6.2.3 Induced Subsidence

Submerged alluvial deposits varying in thickness up to a maximum of about 20 feet are expected to be dewatered during construction. Potential settlements due to dewatering were calculated based on the assumption that the materials below the water table were granular and similar to those encountered in the borings. In addition, it was assumed that the dewatered soils overlie Topanga bedrock. These calculations indicate that total surface settlement would be less than 1/4 inch for up to 20 feet of drawdown. Differential settlements across adjacent structures should be less than 1/8 inch. Some of the settlement caused by dewatering would rebound after dewatering is terminated and water levels reach equilibrium.

6.3 UNDERPINNING

The need to underpin and the appropriate type of underpinning for specific buildings located adjacent to the proposed excavation depends on many factors related to both engineering and economics. Thus each structure needs to be evaluated separately. To aid the designers in evaluating underpinning requirements, this section presents general underpinning guidelines based on engineering considerations as shown on Figure 6-1.

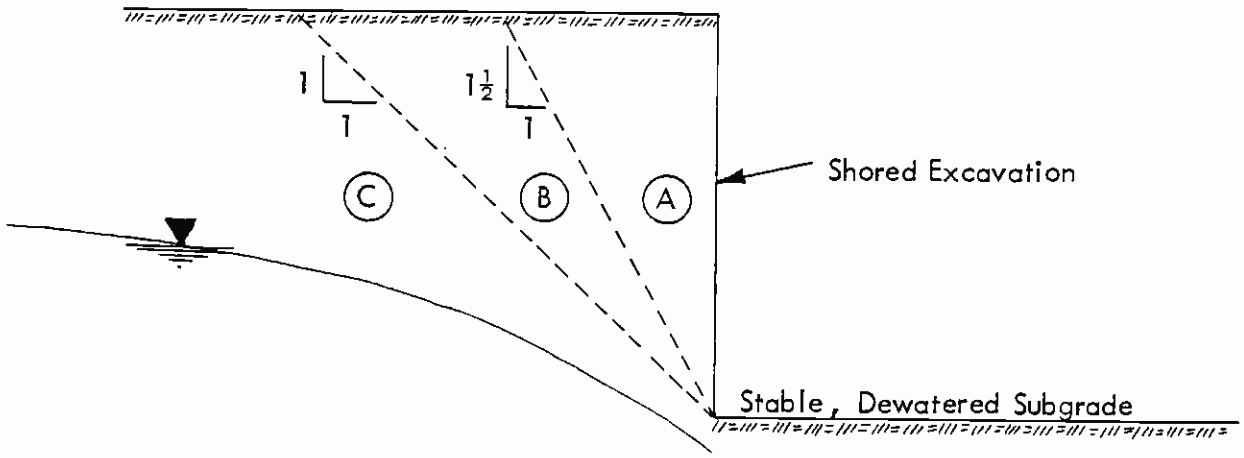
The proposed location of the station and Pocket Track shown on Drawing 3 generally provides setbacks of 70 to 90 feet from the larger structures in the area such as the Hollywood Pacific Theater, Hollywood Security Building, and Mayfair Apartments (see Drawings 3 and 4). However, the possibility of deterioration of wood piles due to ground water lowering should be checked for these larger structures. There are some minor structures which are closer to the proposed station. These include two minor commercial structures, west of the station on Hollywood Boulevard, which appear to be within about 25 feet of the excavation; two residential structures, at the north end of the Pocket Track, which are within about 60 feet; and one residence which is about 25 feet from the excavation. Considering the relatively minor size and importance of the nearest structures, it is not expected that the section designer will recommend underpinning at this site. Therefore, no further discussions and recommendations on underpinning are considered warranted at this time.

6.4 TEMPORARY EXCAVATIONS

6.4.1 General

The required A350 station and pocket track excavation will extend some 45 to 80 feet below the existing ground surface and up to 40 feet below the water table. Several methods for supporting vertical excavations may be employed. These methods include soldier piles with lagging, sheet piles, and slurry wall construction. Bracing systems are generally limited to soil/rock anchor tiebacks or internal bracing. We understand that the excavation support system will be chosen and designed by the contractor in accordance with specified criteria and subject to the review and acceptance by the Metro Rail Construction Manager.

Conditions encountered at the site will cause some difficulty in installation of any type of shoring system. Difficult drilling and caving of the sand/gravel alluvium was experienced during exploration in all borings. Man-size auger Boring 28C encountered ravelling and sloughing from a shallow gravelly layer (12 to 16 feet) and below the water level (52 feet). Rotary wash Boring



NOTES: 1.) These guidelines consider displacements related to shoring movement for stable ground. Other conditions would require special evaluation.

2.) For structure foundations bearing in zones A, B, or C the following guidelines are presented:

Ⓐ Special Provisions Required for Important Structures:

Underpinning or construction of conservative shoring system (designed to support lateral loads from building foundations with acceptably small ground movements) must be considered.

Ⓑ Generally No Special Provisions Required:

Properly designed shoring system generally adequate without underpinning unless underlain by poor soils or adjacent to especially sensitive structures. Settlements due to dewatering must also be considered.

Ⓒ No Special Provisions

Ground displacements due to shoring are negligible however settlements due to dewatering must be considered.

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UNDERPINNING GUIDELINES - ADJACENT TO SHORING

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Figure No.
6-1

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28-6 encountered caving conditions below about 52 feet, and the hole eventually caved back from 82 feet to 58 feet and was abandoned. Casing was required to a depth of 31 feet in Boring 28-8. Therefore, caving should be anticipated in excavations which penetrate the granular alluvium. In addition, excavations which penetrate the bedrock may encounter hard cemented sandstone zones or intrusions of basalt.

Both slurry wall and soldier pile systems are considered feasible, but their construction will likely encounter problems with caving as discussed above. Driven sheet pile shoring does not appear feasible at this site due to the presence of dense gravelly soils and the possibility of encountering cemented bedrock or basalt intrusions.

Internal bracing would appear to be preferable over tiebacks from the installation standpoint due to the potential for caving in the granular alluvium. Consideration may be given to a combination of tieback support in the upper portion of the shoring and internal bracing in the lower portions (where tiebacks would penetrate the granular alluvium).

The need for a stiff shoring system (such as a slurry wall) does not appear to be necessary at this site since no major structures appear to fall within the zone of influence of the excavation.

Considering the above-discussed items and local construction practice, we believe that a conventional soldier pile and lagging shoring system with tiebacks and/or internal bracing is the most likely shoring system to be used at this site. The following discussions and recommendations are, therefore, directed to a conventional soldier pile wall system. However, other shoring systems can be considered by the contractor, and further recommendations can be provided for their design if required.

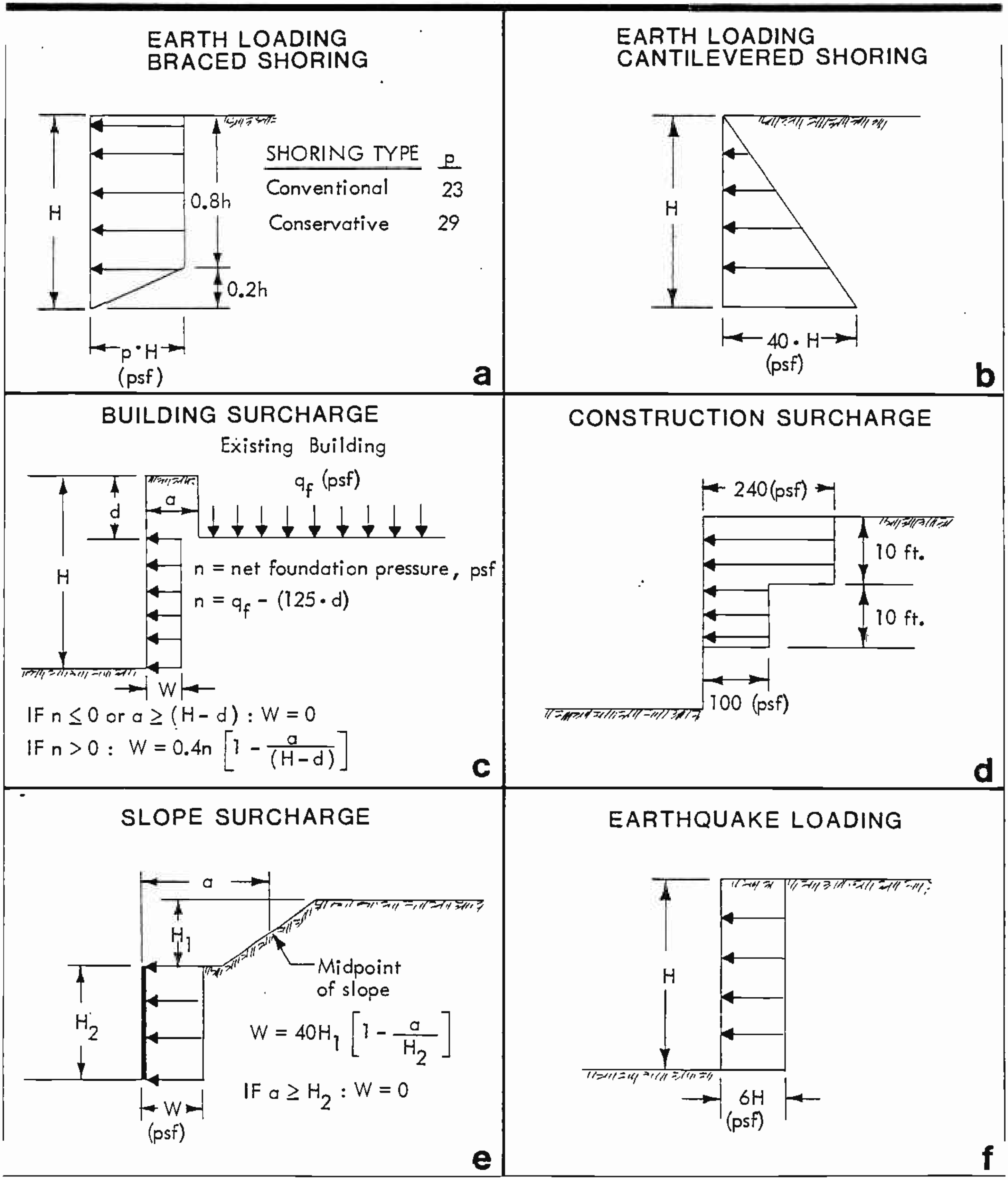
6.4.2 Shoring Design Criteria

This section provides design criteria for both conventional and conservative soldier pile shoring systems consisting of soldier piles and wooden lagging supported by tiebacks and/or internal bracing. The soldier piles are assumed to consist of steel W or H-sections installed in predrilled circular shafts. It is assumed that the drilled shaft will be filled with concrete. Thus, for computing the allowable soil loads, the piles were assumed to have circular concrete sections.

Appendix E.1 summarizes the design shoring pressures for nine shoring systems in the Los Angeles vicinity. There are no known data on field measurements of actual lateral soil pressures for shored excavations in the Los Angeles area and, therefore, the design pressures of Appendix E.1 have not been directly verified. However, performance of shoring walls designed based on local practice has generally been good.

Specific shoring design criteria include:

- Design Wall Pressure: Figures 6-2a and 6-2b present the recommended lateral earth pressure on the temporary shoring walls. Design lateral pressures for both conventional and conservative shoring systems are presented in Figure 6-2a. Figure 6-2e also includes the case of partial slope cuts. Appendix D.2 provides technical support for the recommended



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No
6-2

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seismic pressures of Figure 6-2f. The full loading diagram above the bottom of excavation should be used to determine the design loads on tieback anchors and the required depth of embedment of the soldier piles. For computing design stresses in the soldier piles, the computed values can be multiplied by 0.8. For sizing lagging, the earth pressures can be reduced by a factor of 0.5.

- <u>Depth of Pile Embedment</u>: The embedment depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to satisfy both the lateral and vertical loads under static and dynamic loading conditions for both soil and bedrock materials.

  The required depth of embedment to satisfy vertical loading should be computed based on allowable vertical loads shown on Figures 6-3 and 6-4. Figure 6-3 should be used for piles penetrating alluvium. Figure 6-4 should be used for piles penetrating into Topanga Formation bedrock.

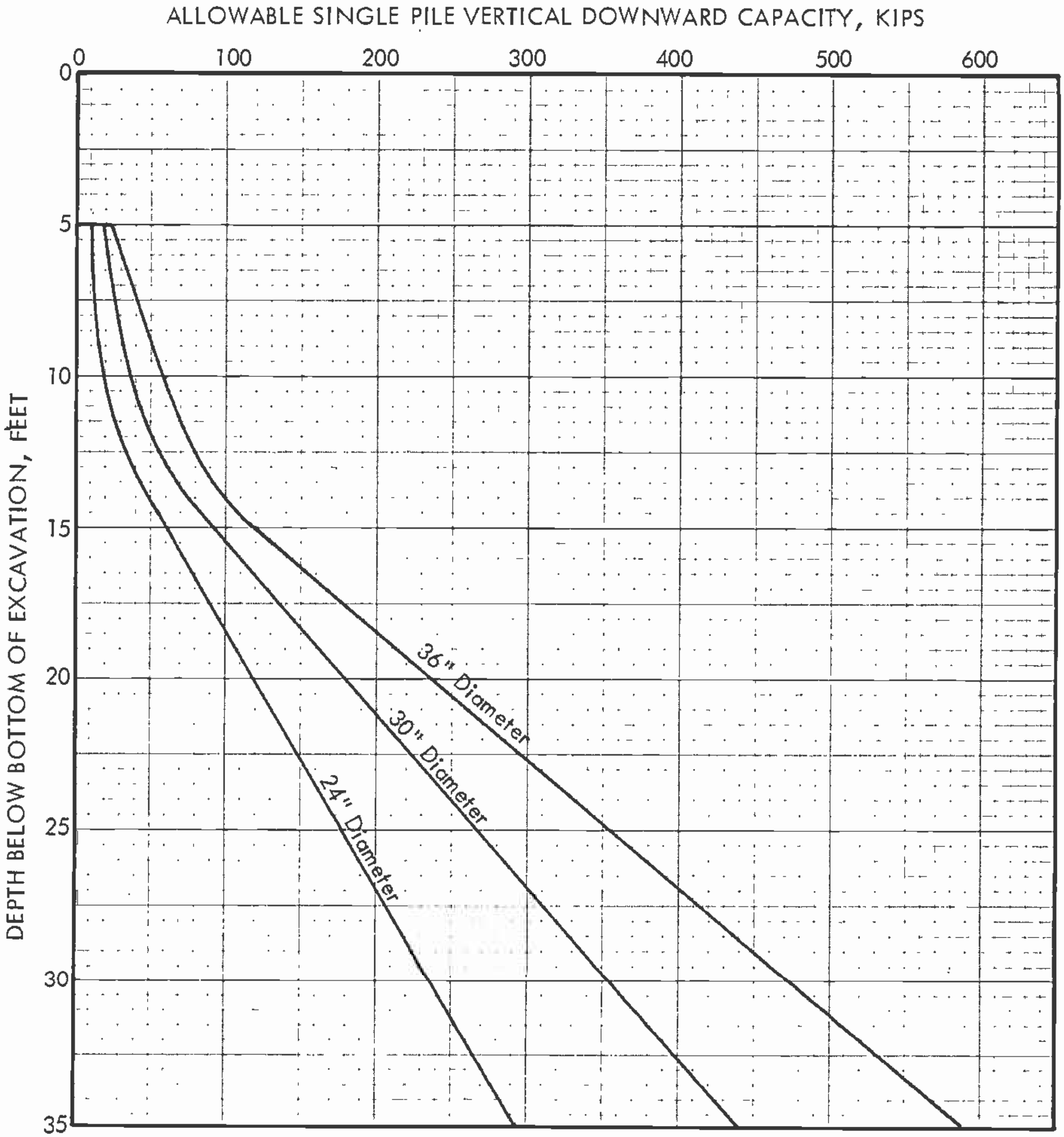
  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-2 minus the support from tiebacks and/or internal bracing. The required depth of embedment to satisfy lateral loads should be computed based on the net allowable passive resistance (total passive resistance of the soldier pile minus the active earth pressure below the excavation). Due to arching effects, it is recommended that the effective pile diameter be assumed equal to 1.5 pile diameters or half of the pile spacing, whichever is less. Figures 6-5 and 6-6 indicate the recommended method to compute net passive resistance. Figure 6-5 should be used for piles penetrating alluvium. Figure 6-6 should be used for piles which penetrate bedrock.

- <u>Pile Spacing and Lagging</u>: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. Exposed alluvial soils will be subject to ravelling and sloughing. Thus, it is recommended that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles and that pile spacing be limited to 8 feet center to center. The contractor should limit the temporarily exposed soil height to less than 3 feet to control ravelling problems, especially in the dewatered zone.

- <u>Excavation Stability</u>: As part of the shoring design, stability calculations should be performed to verify that the shoring/tieback system has an adequate safety factor against deep-seated failure.

6.4.3 <u>Internal Bracing and Tiebacks</u>

6.4.3.1 General: Tiebacks and/or internal bracing may both be suitable to support the temporary shoring wall for the proposed excavation. Tiebacks have the advantage of producing an open excavation which can significantly simplify the excavation procedure and construction of the permanent structure. However, at this site, installation of tiebacks may be difficult in the granular alluvium due to the potential for caving. Obtaining permission to install tiebacks under adjacent properties and encountering obstructions from adjacent below grade structures (such as basements) can also affect the economics and feasibility of tiebacks.



NOTE: 1) For seismic design, capacities may be increased 33%.

VERTICAL CAPACITY OF PILES FOR SHORING & DECKING (ALLUVIUM)

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Figure No.
6-3

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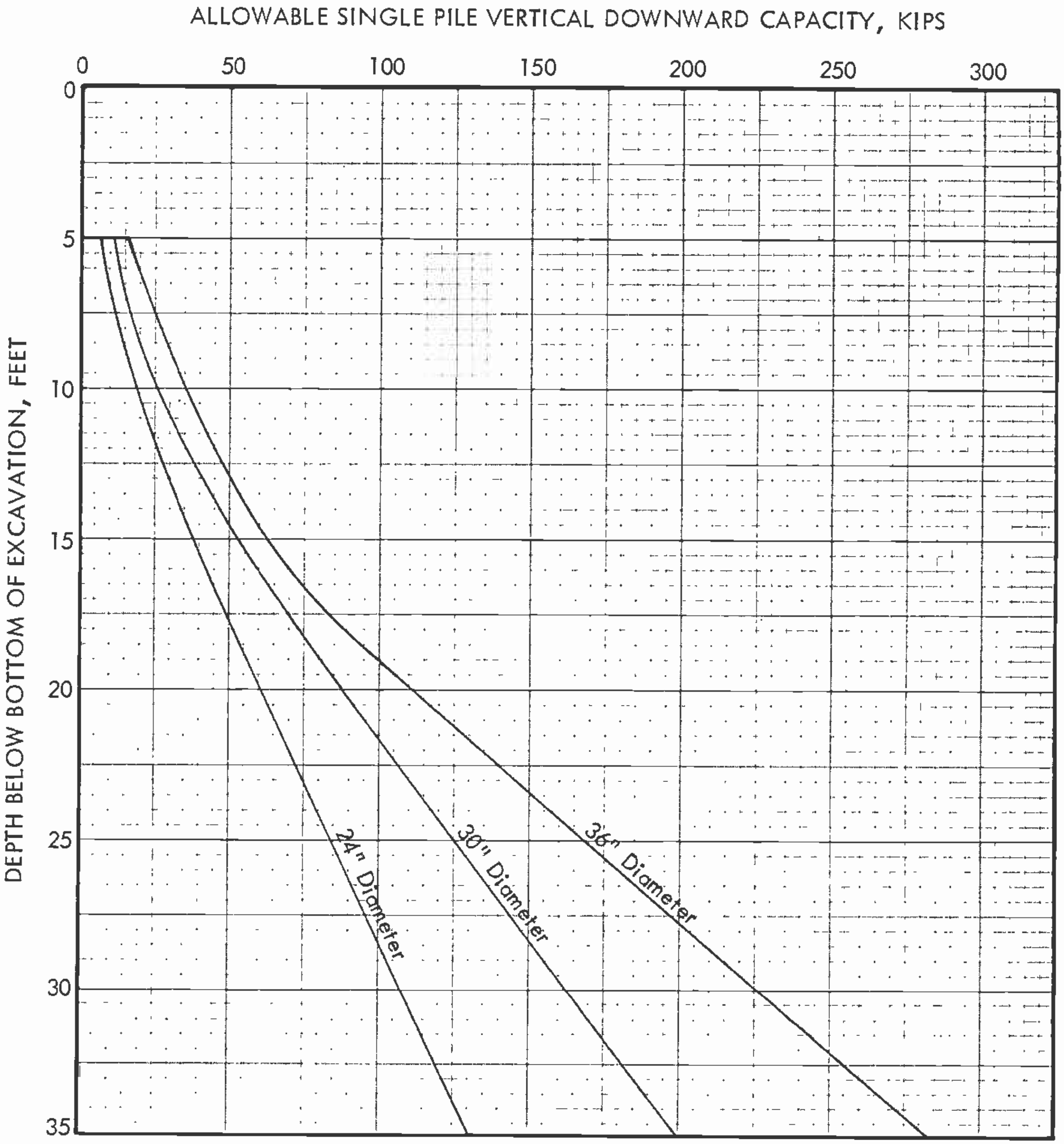

NOTE: 1) For Seismic Design, capacities may be increased 33%.

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VERTICAL CAPACITY OF PILES FOR SHORING & DECKING (BEDROCK)

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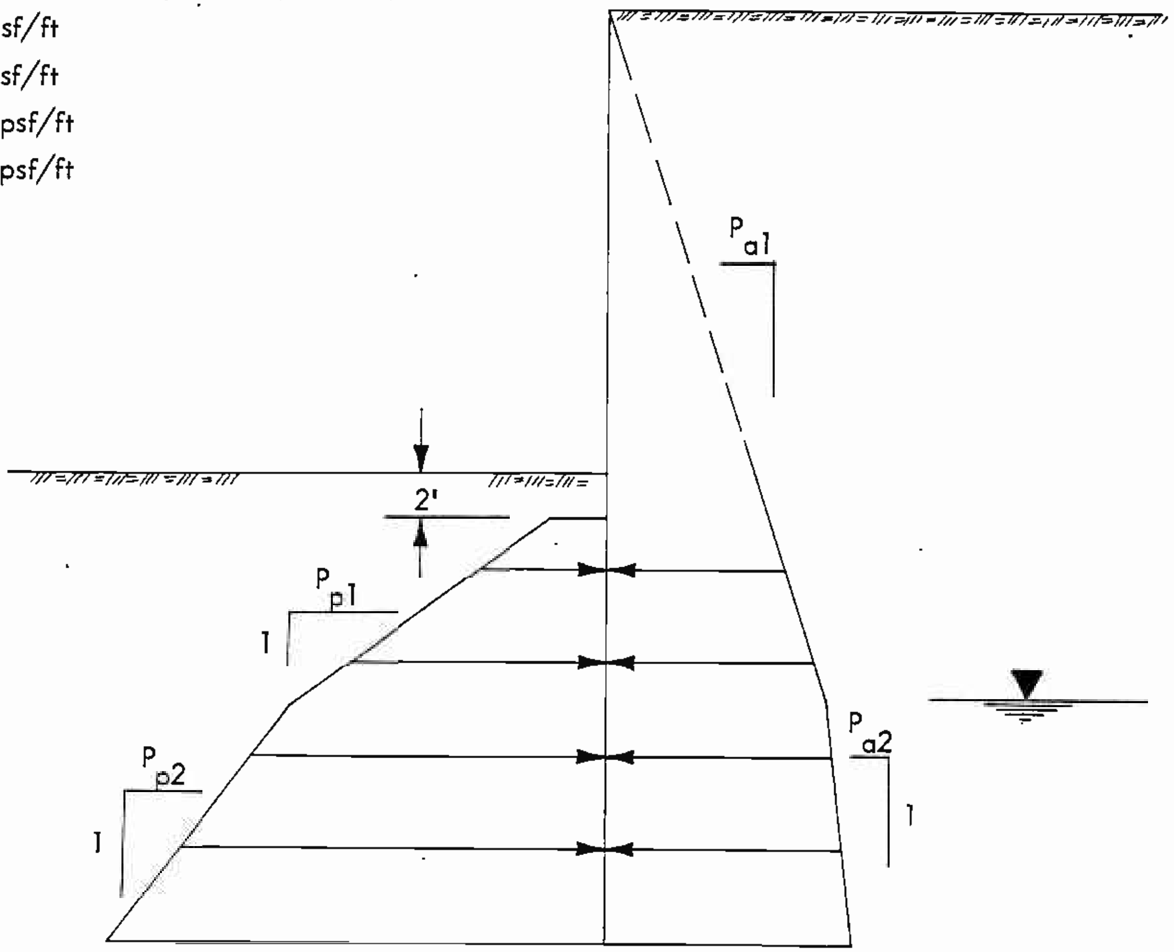
Figure No.
6-4

Recommended Unit Pressures

P_{a1} = 35 psf/ft
P_{a2} = 20 psf/ft
P_{p1} = 350 psf/ft
P_{p2} = 200 psf/ft



Where: P_p = Total allowable unit passive pressure
P_a = Unit active pressure

NOTES:
1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive - Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or $\frac{1}{2}$ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-2

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SOLDIER PILE PASSIVE RESISTANCE (ALLUVIUM)

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Figure No.
6-5

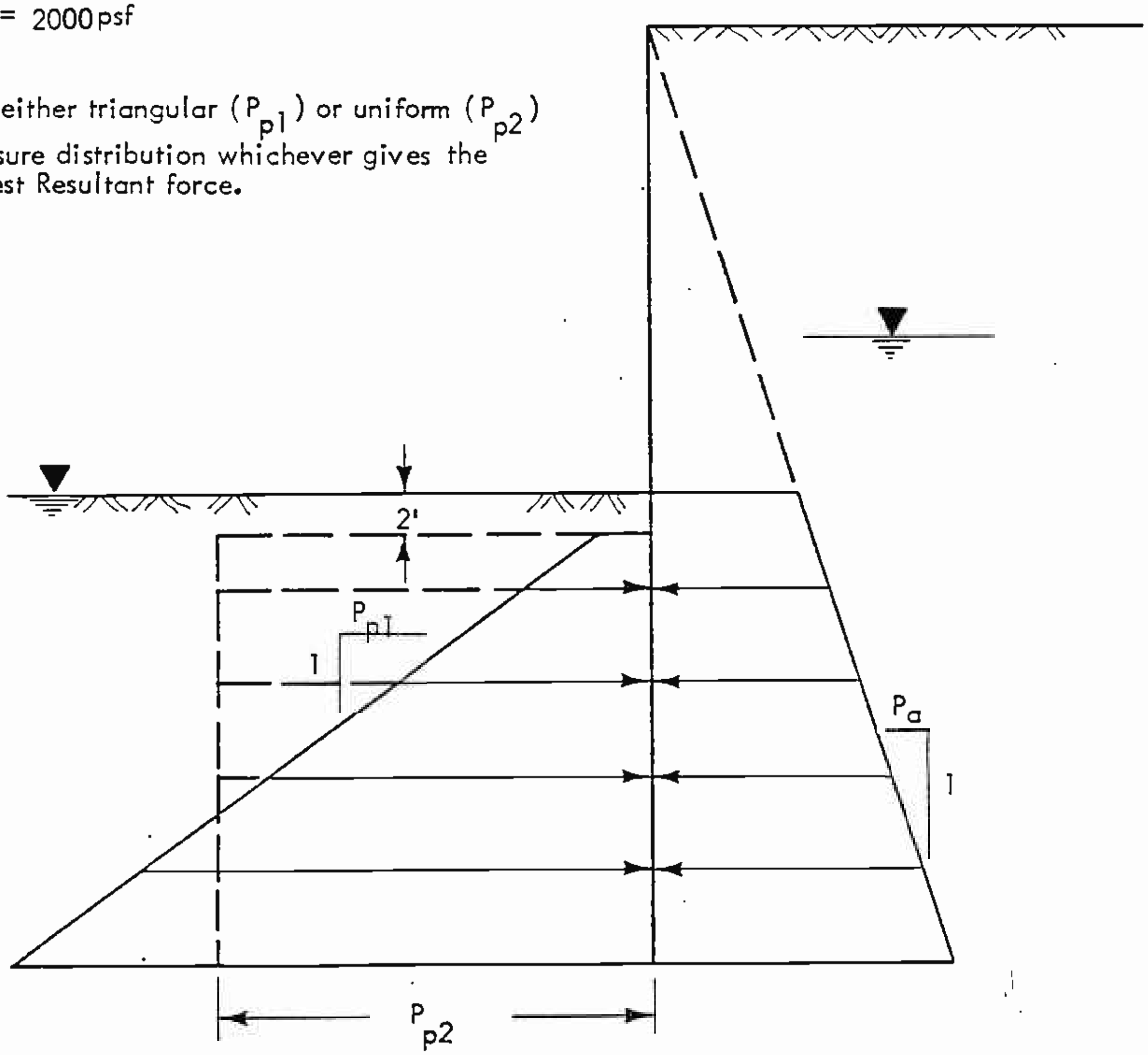
Recommended Unit Pressures

P_a = 20 psf/ft

P_{p1} = 160 psf/ft

P_{p2} = 2000 psf

Use either triangular (P_{p1}) or uniform (P_{p2}) pressure distribution whichever gives the lowest Resultant force.



Where: P_p = Total allowable unit passive pressure

P_a = Unit active pressure

NOTE:
1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive - Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-2

SOLDIER PILE PASSIVE RESISTANCE (BEDROCK)

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Figure No.
6-6

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6.4.3.2 Performance: Based on available field data there does not appear to be a significant difference between the maximum ground movements of properly designed and carefully constructed tieback walls or internally braced walls. However, there is a difference in the distribution of the ground movements. Prestressing of both tiebacks and struts is essential to confirm design capacities and minimize ground movements.

6.4.3.3 Internal Bracing: The contractor should not be allowed to extend the excavation an excessive distance below the lowest strut level prior to installing the next strut level. The maximum vertical distance depends on several specific details such as the design of the wall and the allowable ground movement. These details cannot be generalized. However, as a guideline, we recommend consideration of the following maximum allowable vertical distances between struts:

- Conventional Shoring System: 12 feet
- Conservative Shoring System: 8 feet

In addition, the contractor should not be allowed to extend the excavation more than 3 feet below the designated support level before placing the next level of struts. The contractor may be allowed to excavate a trench within the excavation to facilitate construction operations provided the trench is not less than 15 feet horizontally from the shoring and does not extend more than 6 feet below the designated support level.

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable. The shoring design, preload procedures, and monitoring/maintenance procedures must provide for the effects of temperature changes to maintain the shoring support.

6.4.3.4 Tieback Anchors: There are numerous types of tieback anchors available including large diameter straight shaft friction anchors, belled anchors, high pressure grouted anchors, high pressure regroutable anchors, and others. Generally, in the Los Angeles area, high capacity straight shaft or belled anchors have been used in soils which are stable and dewatered and where construction conditions are favorable.

Tieback anchor capacity can be determined only in the field based on anchor load tests. For estimating purposes, we recommend that the capacity of drilled straight shaft friction anchors be computed based on the following equation:

$$P = \pi DLq$$

Where:

P = allowable anchor design load in pounds
D = anchor diameter in feet
L = anchor length beyond no load zone in feet
q = soil adhesion in psf.

The design adhesion value (q) can be determined by:

$$q = 750 \text{ psf (in all bedrock)}$$

$$q = 20d_1 + 10D_2 <750 \text{ psf (in alluvium)}$$

Where:

d_1 = average depth (in feet) of the non-submerged anchor beyond the no-load zone; measured vertically from the ground surface.

d_2 = average depth (in feet) of the submerged anchor below the ground water level.

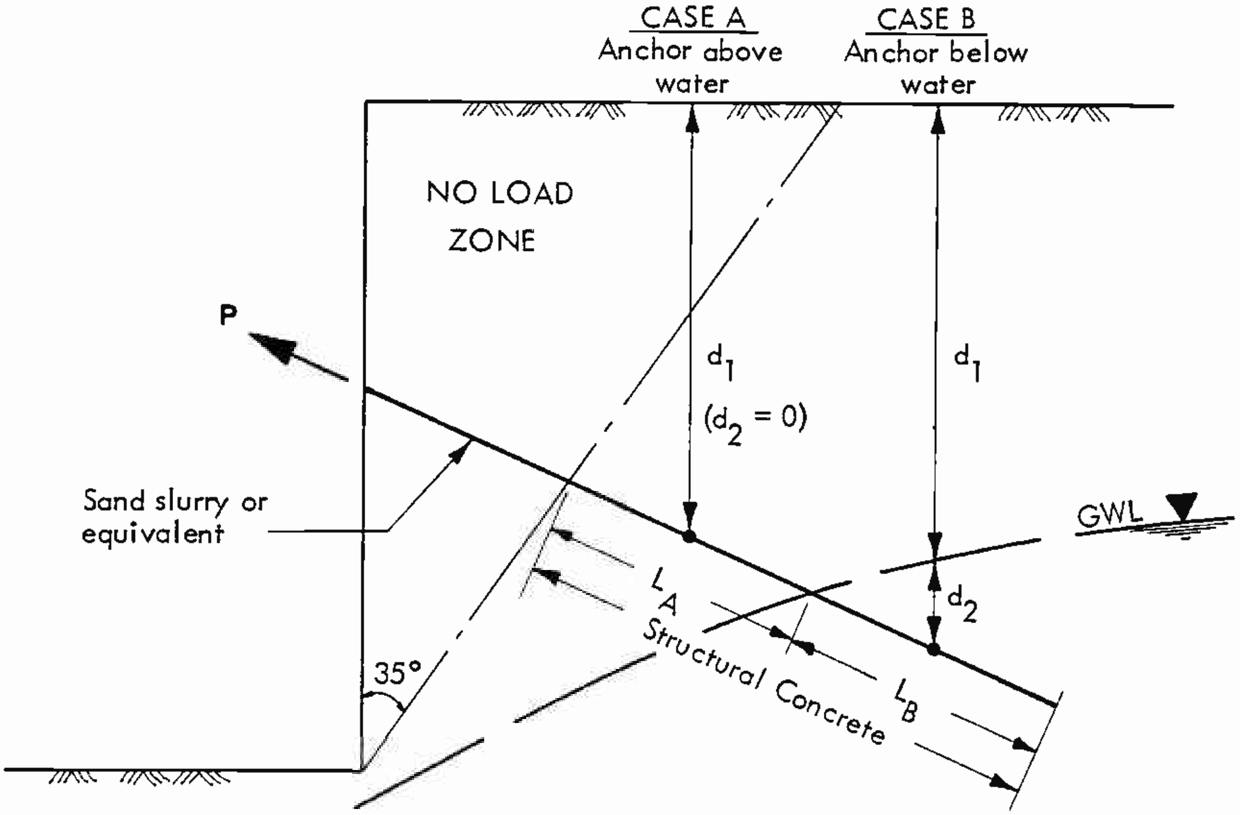
Figure 6-7 illustrates the tieback anchor guidelines.

The above allowable anchor capacity/length relationships are for straight shaft friction anchors only. Design parameters for other types of anchors such as high pressure grouted anchors and high pressure regroutable anchors must be based on experience in the field and on the results of test anchors.

For design purposes, it should be assumed that the potential wedge of failure behind the shored excavation is determined by a plane drawn at 35° with the vertical through the bottom of the excavation. Only the frictional resistance developed beyond the no-load zone should be assumed effective in resisting lateral loads. Based on specific site conditions, the extent of the no-load zone may be locally decreased to avoid underground obstructions.

Structural concrete should be placed in the lower portion of the anchor up to the limit of the no-load zone. Placement of the anchor grout should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the no-load zone and the face of the shoring must be backfilled with a sand slurry or equivalent after concrete placement. Alternatively, special bond breakers can be applied to the strands or bars in the no-load zone and the entire shaft filled with concrete.

For tieback anchor installations, the contractor should be required to use a method which will minimize loss of ground due to caving. Anchors installed in the clay/sand soils should not experience significant caving problems. However, caving of the granular alluvium is expected to occur due to vibration from the drilling equipment and/or ground water effects. Uncontrolled caving not only causes installation problems but could result in surface subsidence and settlement of overlying buildings. To minimize caving, casing could be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, a hollow stem auger could be used.



NOTE:

The design adhesion value, q, can be evaluated by

$q = 750$ psf (in all bedrock)

$q = 20d_1 + 10d_2 \leq 750$ psf (in alluvium)

d = average depth of anchor in feet beyond the no load zone. (d_1 for alluvium above water; d_2 for alluvium below water)

The total anchor capacity can be estimated by:

$$P = \pi DL_A q_A + \pi DL_B q_B$$

See also Section 6.4.4.4

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STRAIGHT SHAFT TIEBACK ANCHOR CAPACITY

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Figure No.
6-7

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It is recommended that each tieback anchor be test loaded to 150% of the design load and then locked off at the design load. At 150% of the design load, the anchor deflection should not exceed 0.1 inches over a 15-minute period. In addition, 5% to 10% of the anchors should be test-loaded to 200% of the design load and then locked off at the design load. At 200% of design load the anchor deflections should not exceed 0.15 inches over a 15-minute period. The rate of deflection should consistently decrease during the test period. If the rate of deflection does not decrease the test should not be considered satisfactory.

6.4.4 Anticipated Ground Movements

The ground movements associated with a shored excavation depend on many factors including the contractors procedures and schedule, and therefore, the distribution and magnitude of ground movements are difficult to predict. Based on shoring performance data for excavations combined with our engineering judgement, we estimate that the ground movements associated with properly designed and carefully constructed soldier pile shoring systems will be as follows:

- Conventional Wall With Tieback Anchors: The maximum horizontal wall deflection will equal about 0.1% to 0.2% of the excavation depth. The maximum horizontal movement should occur near the top of the wall and decrease with depth. The maximum settlement behind the wall should be equal to about 50% to 100% of the maximum horizontal movement and will probably occur at a distance behind the wall equal to about 25% to 50% of the excavation depth.

- Conventional Wall With Internal Bracing: The maximum horizontal and vertical ground movements will be similar to those anticipated with tiebacks. However, the maximum horizontal movement will probably occur near the bottom of the excavation decreasing to about 25% of the maximum at the surface.

- Conservative Wall with Tiebacks: We believe that the higher design pressure presented for conservative walls will reduce ground movements and limit the maximum horizontal and vertical movements to about 0.1% of the excavation depth.

- Conservative Wall With Internal Bracing: Similar to that described above for the conservative tieback supported wall.

6.5 SUPPORT OF TEMPORARY DECKING

We understand that temporary street decking for the Hollywood and Cahuenga Boulevard crossings will require center support piles. These piles would have to extend below the maximum proposed excavation level for support. At these depths, the piles would be founded within the granular alluvium and Topanga Formation bedrock. These materials are suitable for supporting pile loads.

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that he may use similar piles to support the center decking. Accordingly, we evaluated the allowable loads on these types of piles for several typical diameters. The recommended allowable design loads are shown on Figures 6-3 and 6-4. These values include both end bearing and shaft friction.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed A350 excavation should be instrumented to reduce liability (by having documentation of performance), to validate design and construction requirements, to identify problems before they become critical, and to obtain data valuable for future designs.

We recommend the following instrumentation program:

- Preconstruction Survey: A qualified civil engineer should complete a visual and photographic log of all streets and structures adjacent to the sites prior to construction or dewatering. This will minimize the risks associated with claims against the owner/contractor. If substantial cracks are noted in the existing structures, they should be measured and periodically remeasured during the construction period.

- Surface Survey Control: It is recommended that several locations around the excavations and on any nearby structures be surveyed prior to any construction activity and then periodically to monitor potential vertical and horizontal movement to the nearest 0.01 feet. In addition, survey markers should be placed at the top of piles spaced no more than every fourth pile or 25 feet, whichever is less.

- Tiltmeters: Tiltmeters are used to monitor the verticality of buildings adjacent to the excavation and can provide a forewarning of distress. Normally ceramic plates are glued to the building walls and read using a portable tiltmeter containing the same type of tilt sensor used in inclinometers. It is recommended that a few tiltmeters be placed on the exterior walls of buildings which are located within the underpinning zone defined on Figure 6-1. Baseline readings should be made prior to all construction activity, and subsequent readings should be made at several excavation/ construction stages through the end of construction.

- Observation Well Monitoring: Adequate ground water observation wells should be installed prior to dewatering operations. Ground water levels should be monitored frequently during construction.

- Inclinometers: It is recommended that several inclinometers be installed and monitored around the station excavation. Inclinometers should be located on each side of the excavation. The casing could be installed within the soldier pile holes or in separate holes immediately adjacent to the shoring wall. The casing should extend to a depth sufficient to assume fixity of the bottom of the casing. Baseline readings of the inclinometers should be made immediately upon installation. Subsequent readings should be made at regular time intervals at intervals of excavation progress.

- Heave Monitoring: The magnitude of the total ground heave should be measured. This information will be valuable in determining the ground response to load change and as an indirect check on the magnitude of the predicted settlement of the station structure.

  We recommend that heave gages be installed along the longitudinal centerline of the excavation on about 200-foot centers. The devices could consist of conical steel points, installed in a borehole, and monitored with a probing rod that mates with the top of the conical point. The borehole should be filled with a thick colored slurry to maintain an open hole and allow for easy hole location. The top of the points should be at least 2 feet below the bottom of the final excavation to protect them from equipment yet allow for easy access should the hole collapse.

  The points should be installed and surveyed prior to starting excavation. Once the excavation begins, readings should be taken at about two-week intervals until the excavation is completed and all heave has stopped.

- Convergence Measurements: We recommend the use of tape extensometers to measure the convergence between points at opposite faces of the excavation during various stages of excavation. These measurements provide inexpensive data to supplement the inclinometer and survey information.

- Measurement of Strut Loads: If internal bracing is used, we recommend that the loads on at least four struts at each support level be monitored periodically during the construction period. These measurements provide data on support loads and a forewarning of load reductions which would result in excessive ground movements. There should be a means of measuring the strut temperature at the time of the load readings.

- Frequency of Readings: An appropriate frequency of instrumentation readings depends on many factors including the construction progress, the results of the instrumentation readings (i.e., if any unusual readings are obtained), costs, and other factors which cannot be generalized. The devices should be installed and initial readings should be taken as early as possible. Readings should then be taken as frequently as necessary to determine the behavior being monitored. For ground movements this should be no greater than one to two-week intervals during the major excavation phases of the work. Strut load measurements should be more frequent, possibly even daily, when significant construction activity is occurring near the strut (such as excavation, placement of another level of struts, etc.).

  The frequency of the readings should be increased if unusual behavior is observed.

In our opinion, it is important that the installation and measurement of the instrumentation devices be under the direction and control of the Engineer. Experience has shown when the instrumentation program has been included in the bid package as a furnish and install item, the quality of the work has often been inadequate such that the data are questionable. By defining Support Work (Contractor) and Specialist Work (Engineer) in the bid documents, RTD could allow the contractor to provide support to the Engineer in installing the instrumentation.

6.7 EXCAVATION HEAVE AND SETTLEMENT OF THE STATION STRUCTURE & POCKET TRACK

6.7.1 General

The proposed excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed 55-foot excavation at the station will decrease the vertical effective ground stresses by about 7000 psf. The proposed 80-foot deep excavation at the north end of the Pocket Track will result in an effective stress reduction of about 8000 psf. Stress reduction caused by the excavation will result in rebound or heave of the alluvium and bedrock below the excavation. Since the excavation will be open for an extended period, the heave is expected to be completed prior to construction of the Station. The structure and subsequent backfilling will reload the soil. We estimate that the net loads will be about 4000 psf at the station and 4500 to 7000 psf along the Pocket Track. These loads will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil exceeds the weight of the final structure, the structure will experience some settlement due to recompression.

6.7.2 Excavation Heave

We estimate that the maximum heave at the center of the excavation will range from 3 to 5 inches. We believe that the majority of this will occur while the excavation is being made. These estimates are based on computations of elastic shear deformation (elastic rebound) within the alluvium and bedrock underlying the proposed excavation.

6.7.3 Total Settlement

Settlement calculations for the station and pocket track structures were performed based on the elastic properties of the subgrade materials and estimated imposed loads due to the structures and backfill given above. Total settlement of the station structure was estimated to range from 1 to 3 inches. Settlement of the pocket track was estimated to range from 2 to 4 inches. This range is considered applicable to both the alluvium and bedrock supported portions of the pocket track.

6.7.4 Differential Settlement

Due to the long narrow shape of the imposed load, the calculated differential settlement between the edge and center of the structure ranged between 1/2 to 3/4 inch considering both alluvial and bedrock subgrade conditions. This correlates to an angular rotation of 1:500 to 1:700. However, differential settlements due to variations of subgrade conditions along the structure could be 2 inches within the Hollywood fault zone. Differential settlement may occur over short distances such as at the contact between bedrock and alluvial. The exact location of such subgrade discontinuities cannot be determined at this time, and the accuracy of the estimated differential settlement cannot be further refined until more detailed information can be obtained regarding the characteristics of the bedrock and alluvial materials in the vicinity of the fault.

Geotechnical solutions to the problem of the subgrade discontinuity at the Hollywood Fault are generally limited to "smoothing" the discontinuity to reduce the angular distortion. This may include a wedge-shaped overexcavation of the bedrock material and replacement with compacted fill. However, the benefit of such solutions would be difficult to quantify for the purpose of design and, therefore, should be considered only supplemental. It is our conclusion that this discontinuity may best be handled by an increase in structure stiffness to "bridge" over the discontinuity. The structural designer should give this problem special attention.

6.8 FOUNDATION SYSTEMS

6.8.1 Main Structures

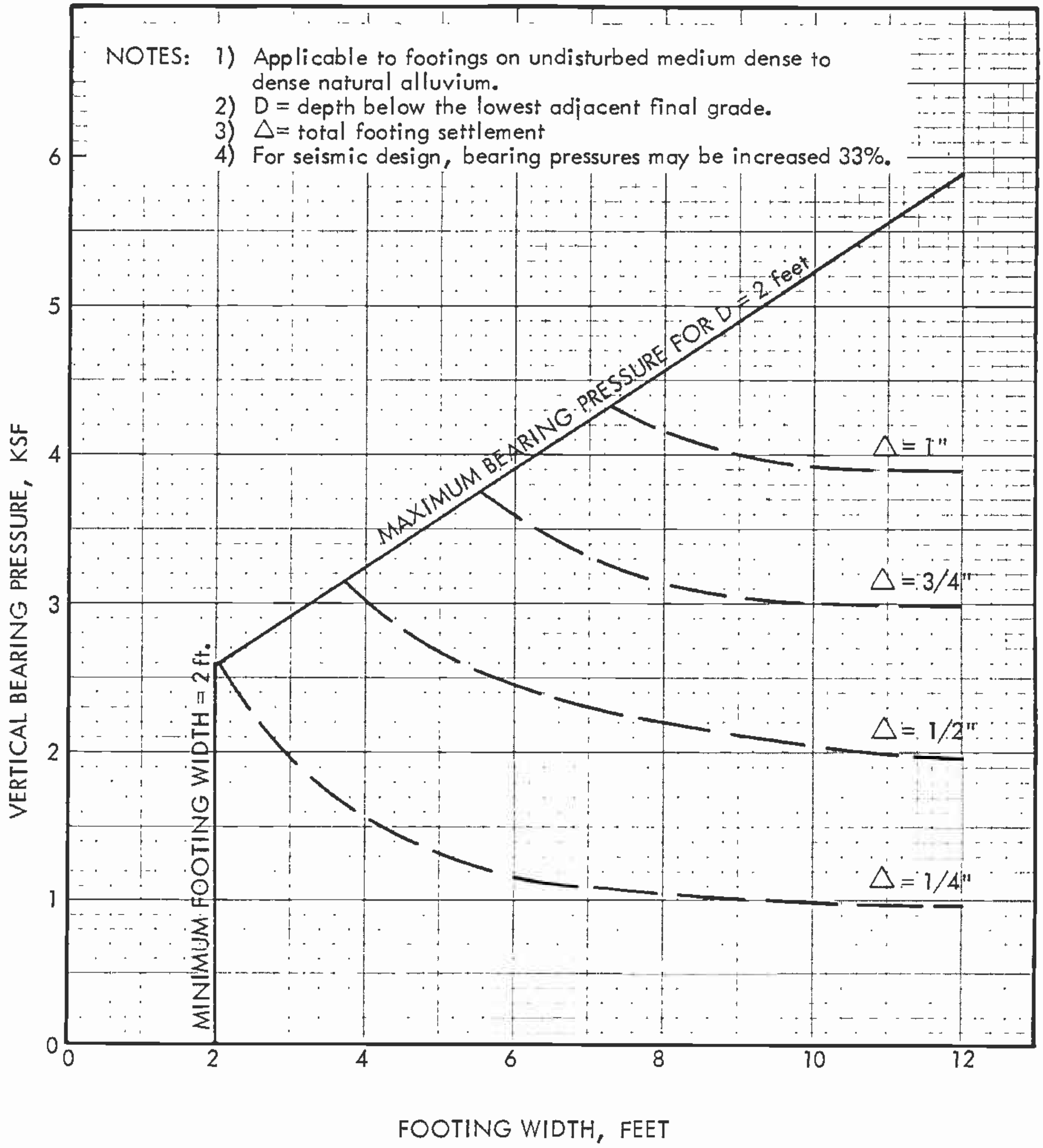
It is understood that the proposed Hollywood/Cahuenga Station and Pocket Track will be supported on a thick base slab which will function as a massive mat foundation. We estimate that the net mat foundation bearing pressures will be about 4000 to 7000 psf. In our opinion the station and Pocket Track can be adequately supported on mat foundations. However, special consideration must be given to potential differential settlements at the Hollywood Fault Zone crossing as discussed in Section 6.7.

6.8.2 Support of Surface Structures

Surface structures can be generally supported on conventional spread footings founded on undisturbed stiff or dense natural soils. If suitable natural soils do not exist at the surface structure site, footings may be founded on a zone of properly compacted structural fill (see Appendix E). Allowable bearing pressures and estimated total settlements of spread footings bearing on the natural alluvium or compacted fill can be determined based on Figures 6-8 and 6-9. These figures are based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed information on structural loadings and site conditions at specific surface structure locations. Detailed site specific studies should be performed to provide final design recommendations for individual structures.

All spread footing foundations should be founded at least 2 feet below the lowest adjacent final grade and should be at least 2 feet wide. The bearing values shown on Figures 6-8 and 6-9 are for full dead load and frequently applied live load. For transient loads, including seismic and wind loads, the bearing values can be increased by 33%. Differential settlements between adjacent footings should be estimated as 1/2 of the average total settlements or the difference in the estimated total settlements shown on Figures 6-8 and 6-9, whichever is larger.

For design, resistance to lateral loads on surface structures can be assumed to be provided by passive earth pressure and friction acting on the foundations. An allowable passive pressure of 200 psf/ft may be used for the sides of footings poured neat against undisturbed alluvium or properly compacted fill. Frictional resistance at the base of foundations should be determined using a frictional coefficient of 0.35 with dead load forces.



SPREAD FOOTING BEARING/SETTLEMENT ON ALLUVIUM

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Figure No.
6-8

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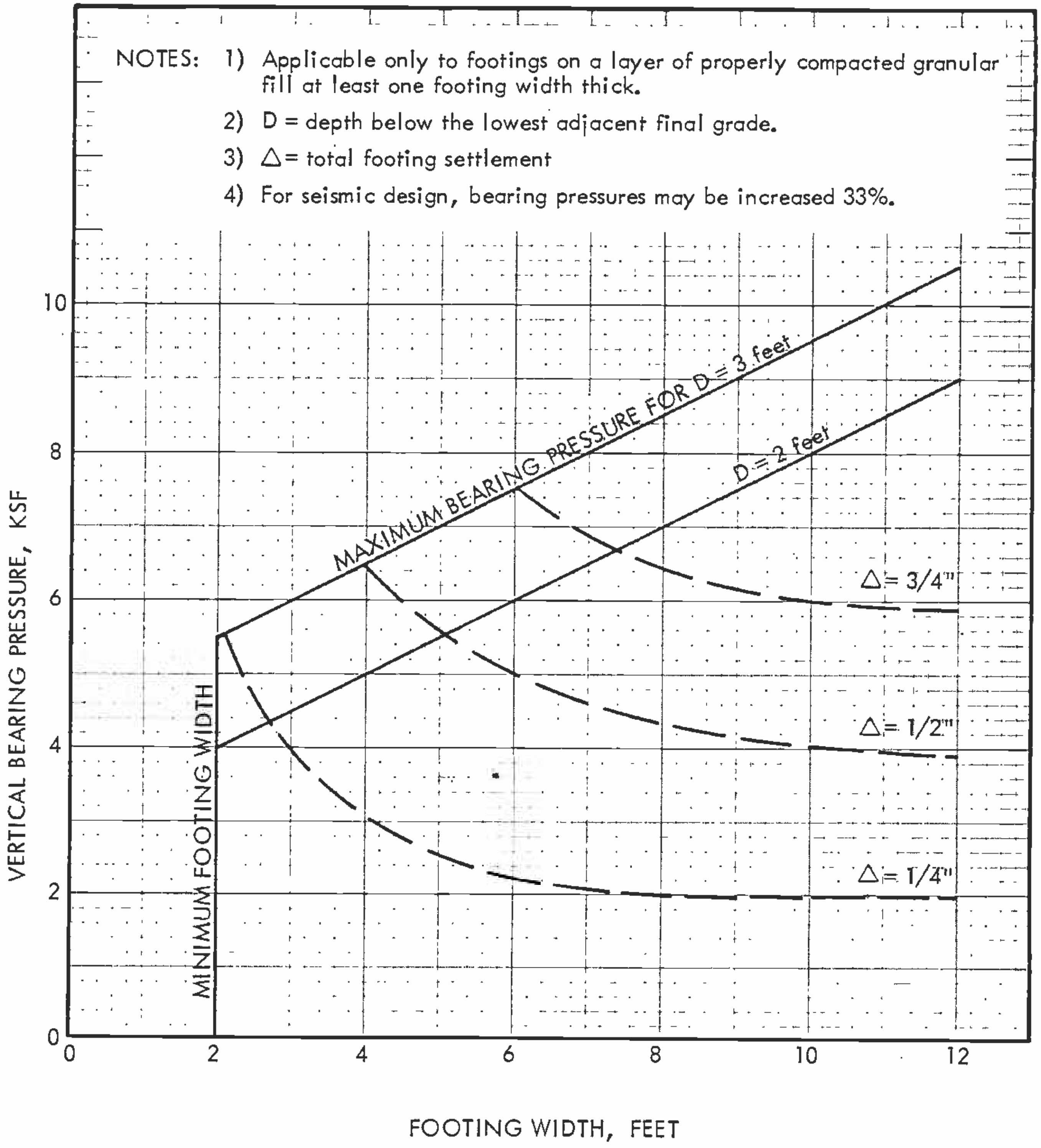

NOTES:
1) Applicable only to footings on a layer of properly compacted granular fill at least one footing width thick.
2) D = depth below the lowest adjacent final grade.
3) Δ = total footing settlement
4) For seismic design, bearing pressures may be increased 33%.

SPREAD FOOTING BEARING/SETTLEMENT ON COMPACTED FILL

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Figure No.
6-9

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6.9 PERMANENT GROUND WATER PROVISIONS

We understand that the station and Pocket Track will be designed to be watertight and to resist the full permanent hydrostatic pressures. We recommend that full waterproofing be carried at least 5 feet above the anticipated maximum ground water levels given in Section 6.10.

6.10 LOADS ON SLAB AND WALLS

6.10.1 Hydrostatic Pressures

As discussed in Section 5.3, the existing ground water levels are estimated to range from about Elevation 305 at the south end of the station to about Elevation 380 at the north end of the Pocket Track. It is recommended that the following ground water levels be assumed for determining hydrostatic pressures:

LOCATION	ELEVATION
North end of Pocket	390
South end of Station	315

6.10.2 Permanent Static Earth Pressures

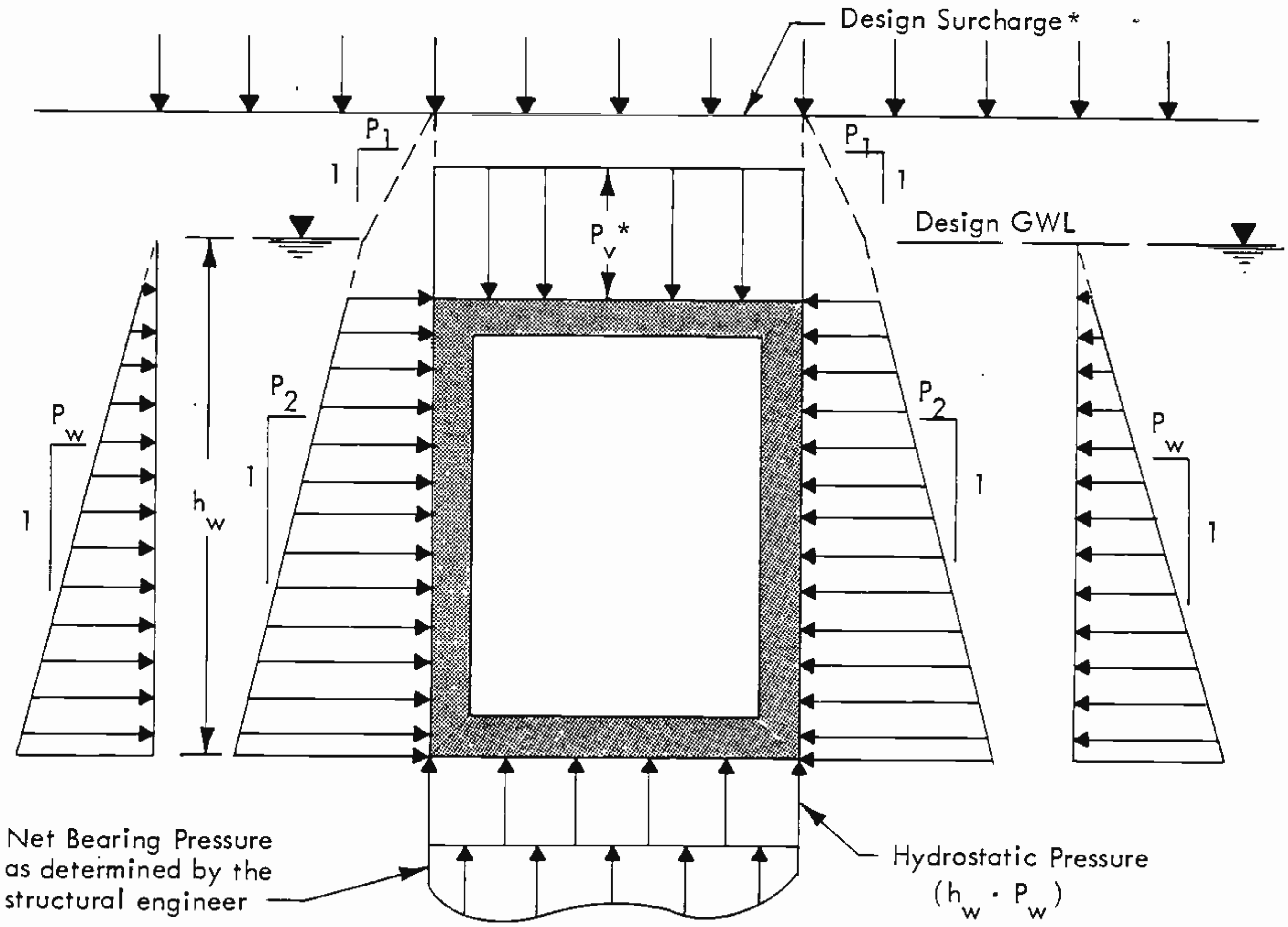
Figure 6-10 presents lateral earth pressure diagrams recommended for design of permanent subsurface walls.

Vertical earth pressures on the roof should be assumed equal to the full moist and/or saturated weight of overburden soil plus surcharge.

We understand that MRTC has modified the Design Criteria and Standards for underground structures to permit use of a simplifying and conservative assumption resulting in a uniform net foundation bearing pressure for the design of the invert slabs of box structures. The use of the elastic soil-structure analysis or the simplified uniform pressure approach is left to the discretion of MRTC and Section Designer.

6.10.3 Surcharge Loads

Lateral surcharge loads from existing or proposed buildings above the structure must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-2. Vertical surcharge loads due to possible future structures, surface traffic, etc. should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	40	20	62.4	*	**
Long Term	60	30	62.4	*	***
Side sway †	40/60	20/30	62.4	*	**

* P_v = full overburden pressure (depth x total density) plus design surcharge; distribution and magnitude of design surcharge to be determined by section designer.

** Designer should use a GWL (between the base of slab and long term water elevation) which will be critical for the loading condition.

*** Varies linearly from elev. 315 at the south end to elev. 390 at the north end.

† Sidesway condition assumes "End Construction" pressure on one side of the structure and "Long Term" on the other.

LOADS ON PERMANENT WALLS

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Figure No.
6-10

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6.11 SEISMIC CONSIDERATIONS

6.11.1 General

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the Southern California Rapid Transit District (SCRTD) 1984 report entitled "Guidelines for Design of Underground Structures". Evaluations of the seismological conditions which may impact the project and the probable maximum credible earthquakes, which may be anticipated in the Los Angeles area, are described in the SCRTD report entitled "Seismological Investigation and Design Criteria", dated May, 1983. The 1984 report complements and supplements the 1983 report.

6.11.2 Dynamic Material Properties

Dynamic soil parameters required for input into the various types of analyses recommended in the seismic criteria report are presented in Table 6-1. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels.

The compression and shear wave velocities are based on interpretation of limited geophysical surveys performed in Borings CEG-28 and CEG-34 during the 1981 investigation. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish modulus values at low strain levels.

TABLE 6-1
RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		MIXED ALLUVIUM (A_3/A_4)	GRANULAR ALLUVIUM (A_3)	BEDROCK (Tt)
Average Compression Wave Velocity, V_c (ft/sec)	- moist	2000	2000	6000
	- saturated	5000	5000	6000
Average Shear Wave Velocity, Vs (ft/sec)		1000	1300	1200
*Poisson's Ratio		0.35	0.35	0.35
Young's Modulus, E, (psi)	- moist	67,000	67,000	630,000
	- saturated	185,000	185,000	630,000
Constrained Modulus, E_c, (psi)	- moist	108,000	108,000	1,000,000
	- saturated	700,000	700,000	1,000,000
Shear Modulus, G_{max}, (psi)		27,000	45,500	40,000

* For saturated condition, use value of 0.45.

The variation of dynamic shear modulus, with shear strain is presented in Figure 6-11 for the various geologic units. Variation of the dynamic shear modulus is expressed as the ratio of the strain compatible modulus (G) to the very low strain modulus (G_{max}). Similar relationships for soil hysteretic damping are presented in Figure 6-12. The modulus and damping curves are based on dynamic laboratory tests performed during our 1981 investigation. Dynamic test results are presented in Vol. II, Appendix H of our 1981 report.

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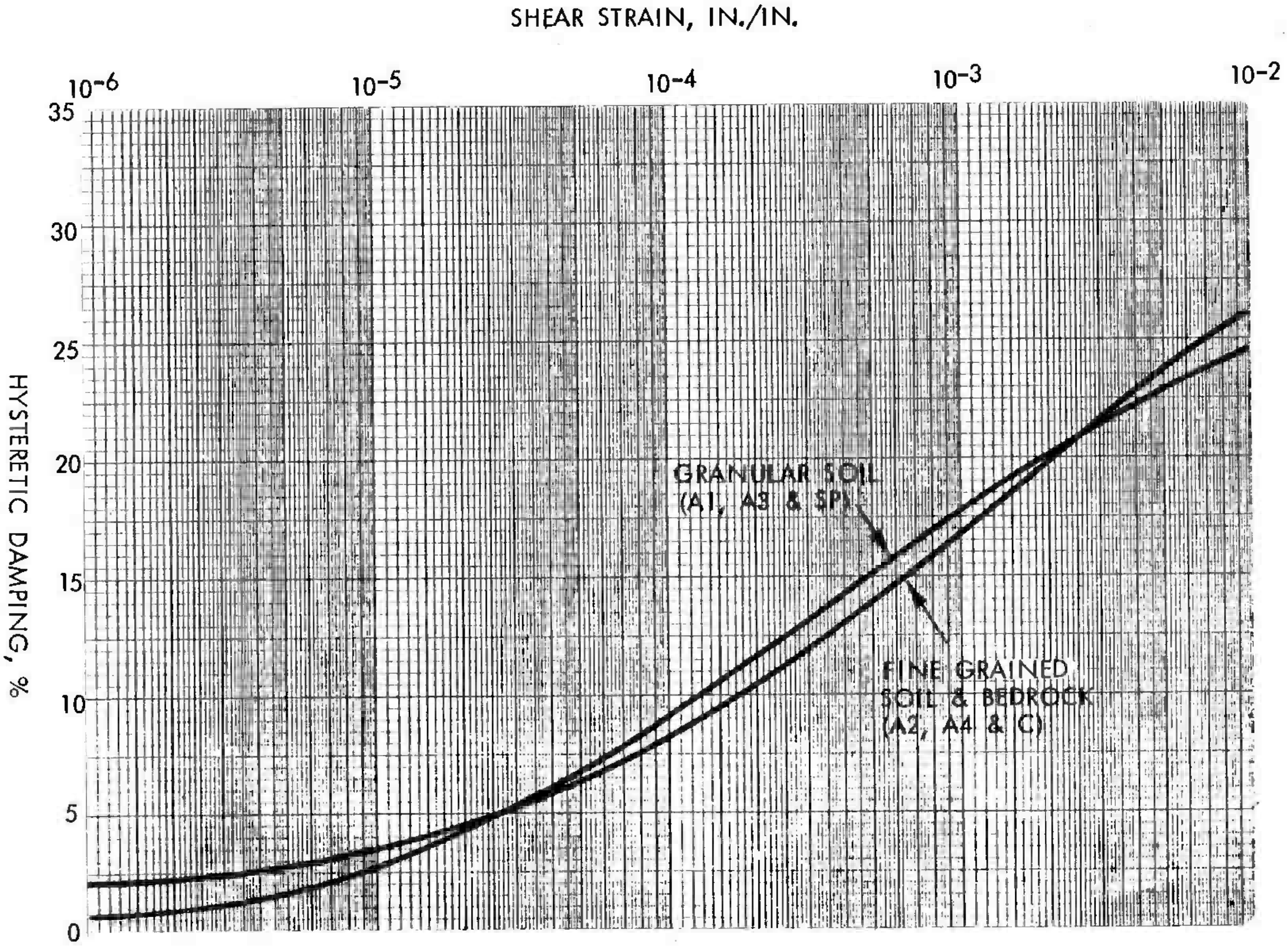

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RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No.
6-11

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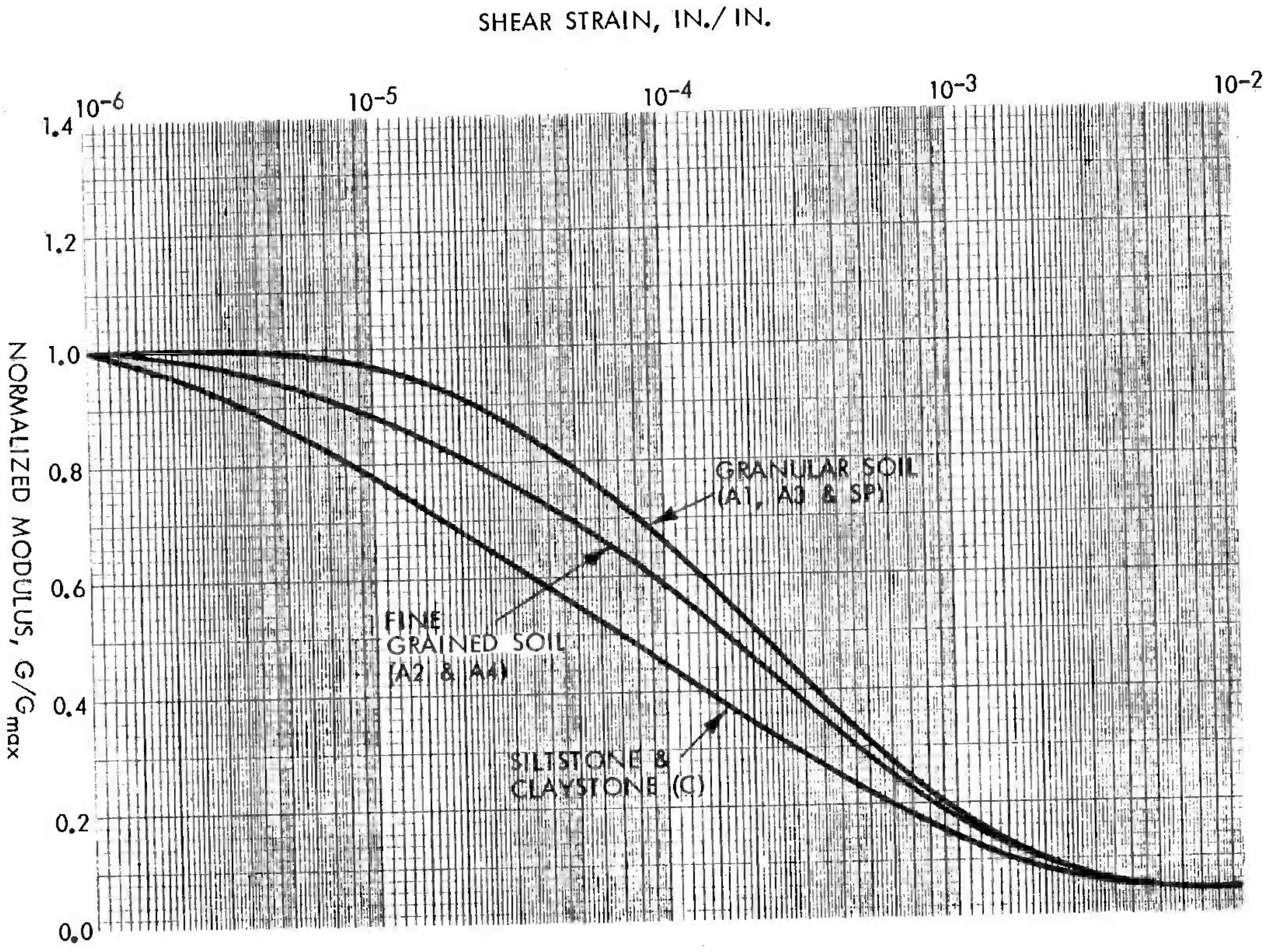

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RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
6-12

6.11.3 Liquefaction Potential

A generalized subsurface cross section has been described in Section 5.0 and is shown in Drawings 2 and 5. The ground water level appears to follow the predominately granular layer. Soils which are saturated and, therefore, must be evaluated for liquefaction potential include the silty sands and gravelly sands of the natural granular alluvial soils.

Liquefaction evaluation procedures were based mainly on correlations of field Standard Penetration Tests (SPT) and performance of soils during previous earthquakes. The field Standard Penetration Tests made at this site during this and the previous geotechnical investigation (1981 Geotechnical Investigation Report) were used for our evaluation of the liquefaction potential of the alluvial soils. Available field geophysical data from CEG-28 were also used in our evaluation as a general indicator of liquefaction potential.

Our analysis of the SPT data was performed in accordance with the simplified procedures of Seed at al (1983). Corrected "N" values (normalized to 2 ksf overburden pressure) for 24 SPT tests in saturated sand soils ranged from a minimum of 11 to a maximum exceeding 50, with an average of about 32. Determination of dynamic strength was based on an M6.0 earthquake for the ODE event and an M7.0 for the MDE event. Results of the analyses indicated that there would be essentially no liquefaction for the ODE event, but about 25% of the SPT values indicated liquefaction for the MDE event. Considering these results, it is our conclusion that the potential for liquefaction during the MDE event is low to moderate and would occur within isolated granular layers.

6.12 EARTHWORK CRITERIA

Site development is expected to consist primarily of excavation for the subterranean structure but will also include general site preparation, foundation preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations and dewatering are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill will be presented in Appendix E (which is not included in the present draft). Recommended specifications for compaction of fill will also be presented in Appendix E. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. Excavated fine-grained soils and bedrock material are not considered suitable because these fine-grained materials will make compaction difficult and could lead to fill settlement problems after construction. If the granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the geotechnical engineer.

6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

Based on the available data and the current design concepts, the following supplementary geotechnical services may be warranted:

- Observation Well Monitoring: The ground water observation wells should be read several times a year until project construction and more frequently during construction if possible. These data will aid in confirming the recommended maximum design ground water levels. They will also provide valuable data to the contractor in determining his construction schedule and procedures.

- Review Final Design Plans and Specifications: A qualified geotechnical engineer should be consulted during the development of the final design concepts and should complete a review of the geotechnical aspects of the plans and specifications.

- Shoring/Dewatering Design Review: Assuming that the shoring and dewatering systems are designed by the contractor, a qualified geotechnical engineer should review the proposed systems in detail including review of engineering computations. This review would not be a certification of the contractor's plan but rather an independent review made with respect to the owner's interests.

- Supplemental Investigation: Consideration should be given to performing supplemental geotechnical investigations at the sites of proposed peripheral at-grade structures near the station. The purpose of these studies would be to determine site specific subsurface conditions and provide site specific final design recommendations for these peripheral structures.

- Construction Observations: A qualified geotechnical engineer should be on site full time during installation of the dewatering system, installation of the shoring system, preparation of foundation bearing surfaces, and placement of structural backfills. The geotechnical engineer should also be available for consultation to review the shoring monitoring data and respond to any specific geotechnical problems that occur.

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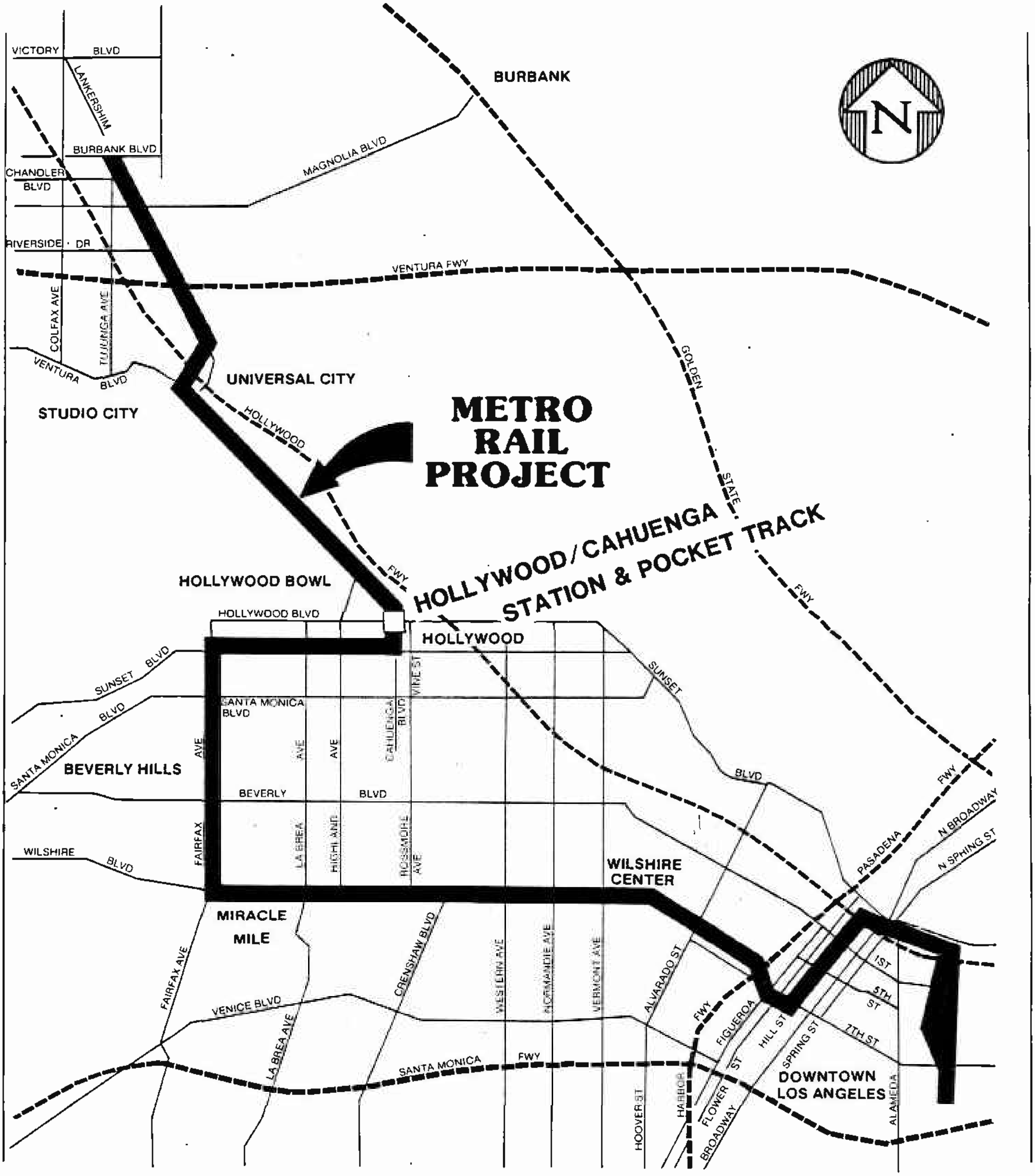
Drawings



VICINITY MAP

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Drawing No
1

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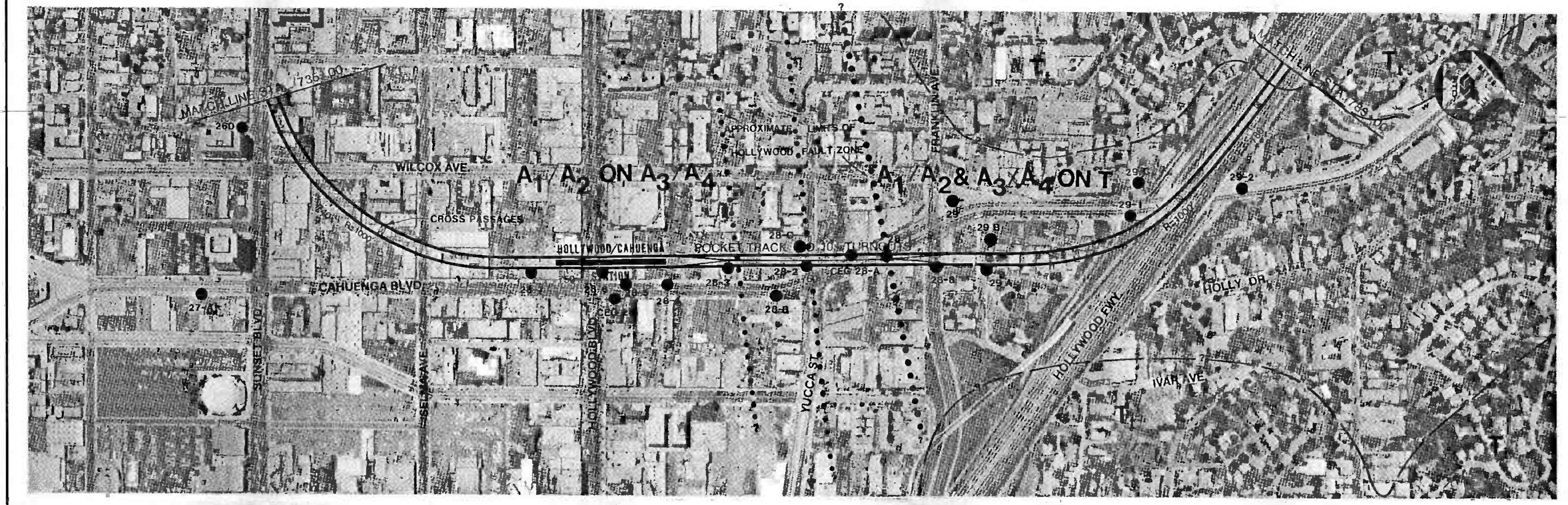

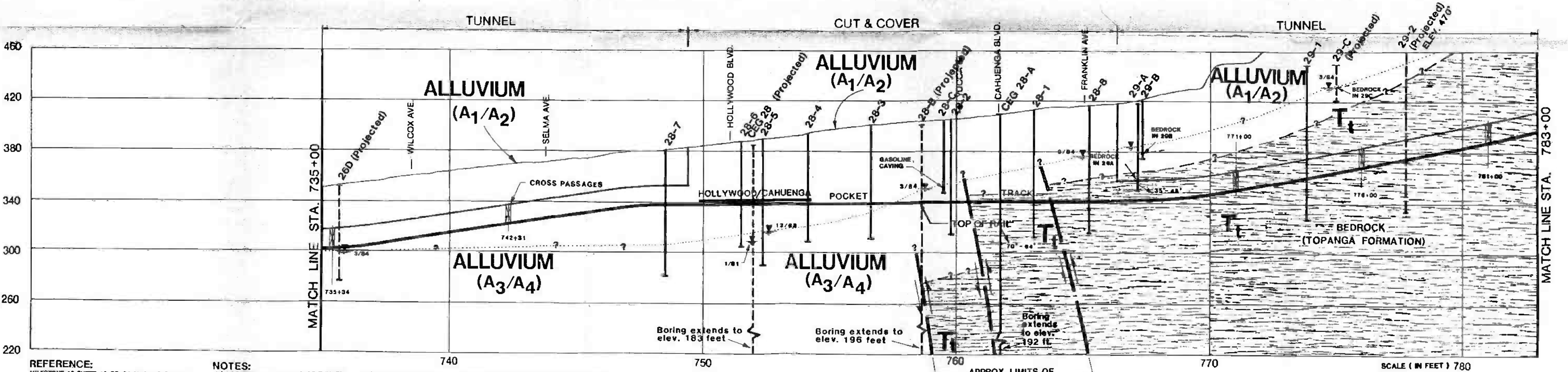

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MILESTONE 10 SHEET 15 OF 21 ALIGNMENT PLAN AND PROFILE STATION 735+00 TO STATION 783+00 DATED MARCH 1983

NOTES:
1.) LOCATION AND GRADE OF TUNNEL AND STATION SUBJECT TO CHANGE.
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5.
3.) THIS DRAWING WAS PREPARED AS AN AID IN DEVELOPING DESIGN RECOMMENDATIONS. SUBSURFACE INFORMATION PRESENTED ON THIS DRAWING IS BASED ON INTERPOLATION AND EXTRAPOLATION OF SUBSURFACE DATA BETWEEN AND BEYOND BORING LOCATIONS. ACTUAL CONDITIONS ENCOUNTERED DURING CONSTRUCTION MAY BE DIFFERENT.

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

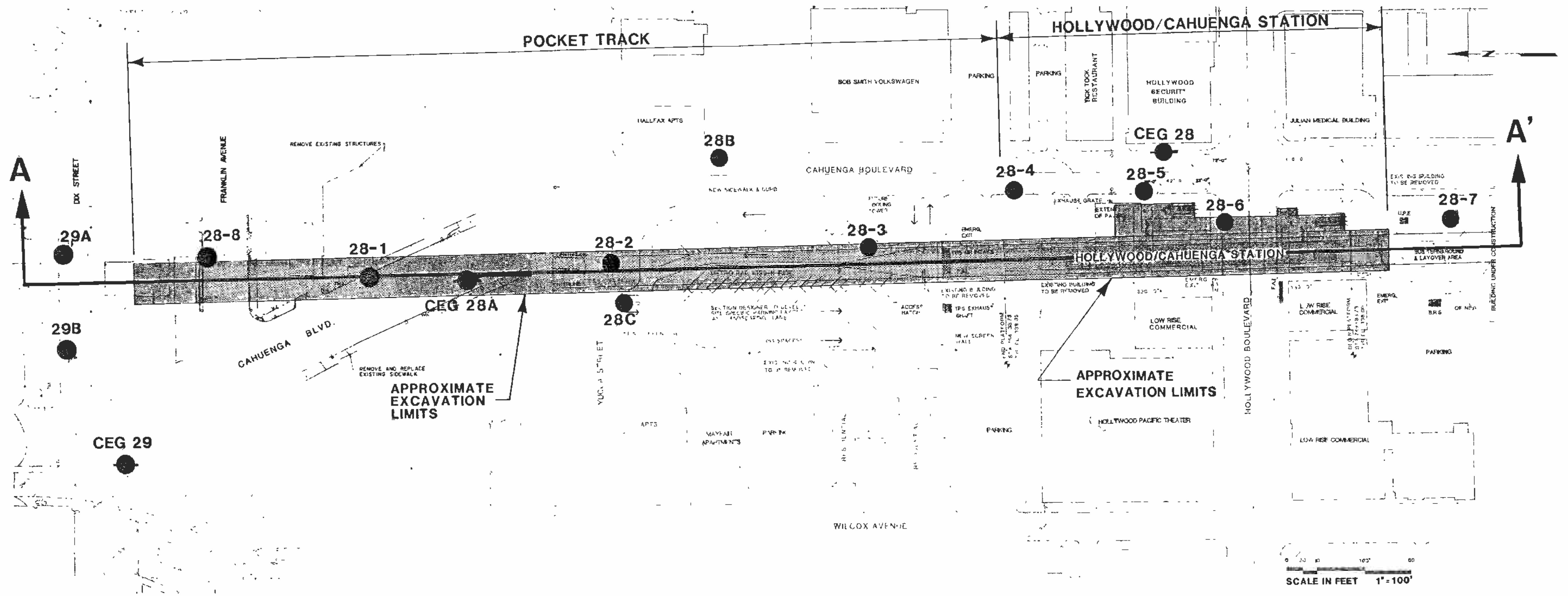
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted RM Pride Date May, 1984

DMJM/PBQD/KE/HWA
A JOINT VENTURE
GENERAL CONSULTANTS
APPROVED

DESIGN UNIT A350
LOCATION OF BORINGS AND GEOLOGIC SECTION

PROJECT NO. 83-1140
DRAWING NO. 2
REV.
SCALE AS SHOWN
SHEET NO.



REFERENCES:

"PRELIMINARY HOLLYWOOD/CAHUENGA STATION SITE", DRAWING #O-17, SHEET 12 OF 24, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"= 100', DATED 3-11-83.

"PRELIMINARY HOLLYWOOD/CAHUENGA STATION SITE", DRAWING #A-56, SHEET 10 OF 24, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"= 100', DATED 5-11-83.

NOTES:
1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 4
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

LOCATION OF BORINGS

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

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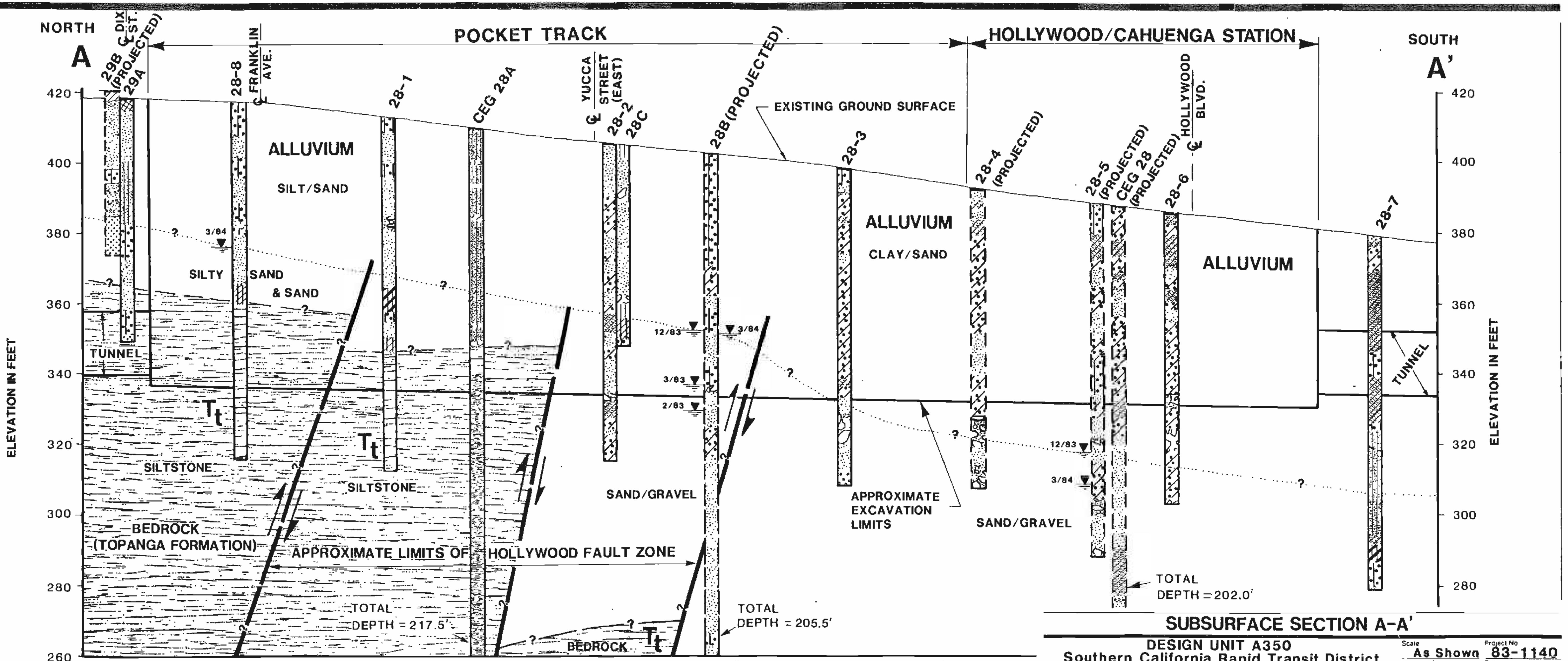
Geotechnical Engineering
and Applied Sciences

Scale As Shown
Date MAY, 1984
Prepared by CSJ/RG
Checked by JAD
Approved By HAS
Project No 83-1140
Drawing No 3



NOTES: 1.) FOR LOCATION OF SUBSURFACE SECTION A-A' SEE DRAWING NO. 3
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

SCALE: HOR. 100 0 100 200
VER. 20 0 20 40

SUBSURFACE SECTION A-A'

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METRO RAIL PROJECT

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Scale As Shown	Project No 83-1140
Prepared by APT/CSJ	Date MAY, 1984
Checked by RG	Drawing No 4
Approved by JAD	

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

QUATERNARY	HOLOCENE	A_1	YOUNG ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, sandy gravels, and locally contains cobbles and boulders. Primarily dense, but ranges from loose to very dense.
		A_2	YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.
	PLEISTOCENE	A_3	OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.
		A_4	OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from firm to hard.
		SP	SAN PEDRO FORMATION: Predominantly clean, cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium dense to very dense. Locally impregnated with oil or tar.
TERTIARY	PLIOCENE	C	FERNANDO AND PUENTE FORMATIONS: Claystone, siltstone, and sandstone; thinly to thickly bedded. Primarily low hardness, weak to moderately strong. Locally contains very hard, thin cemented beds and cemented nodules.
			ROCK TUNNELLING (Terzaghi Rock Condition Numbers apply)*
		3	Terzaghi Rock Condition Number Approximate boundary between Terzaghi numbers
	MIOCENE	2-5	TOPANGA FORMATION: Conglomerate, sandstone, and siltstone, thickly bedded; primarily hard and strong (Geologic symbol Tt).
		1-5	TOPANGA FORMATION: Basalt, intrusive, primarily hard and strong (Geologic symbol Tb).

TERZACHI ROCK CONDITION NUMBERS:*

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

6 Crushed but chemically intact rock or unconsolidated sand; may be running or flowing ground

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

*In practice, there are not sharp boundaries between these categories, and a range of several Terzaghi Numbers may best describe some rock.

SYMBOLS

- - - - - ?	Geologic contact: approximately located; queried where inferred
U ●●●●● ? D	Fault (view in plan): dotted where concealed; queried where inferred; (U) upthrown side, (D) downthrown side
⇌ ?	Fault (view in geologic section): approximately located; queried where inferred; arrows indicate probable movement; attitude in profile is an apparent dip and is not corrected for scale distortion
↙ 40	Dip of bedding: from unoriented core samples; bedding attitudes may not be correctly oriented to the plane of the profile, but represent dips to illustrate regional geologic trends; number gives true dip in degrees, as encountered in boring
·····▼·····?	Ground water level: approximately located; queried where inferred
●	Boring — CEG (1981)
●	Boring — CCI/ESA/GRC (1983)
○	Boring — Nuclear Regulatory Commission (1980)
⊕	Boring — Woodward-Clyde (1977)
⦶	Boring — Kaiser Engineers (1962)
⊖	Boring — Other (USGS 1977 and various foundation studies)

NOTES:

1) The geologic sections are based on interpolation between borings and were prepared as an aid in developing design recommendations. Actual conditions encountered during construction may be different.

2) Borings projected more than 100' to the profile line were considered in some of the interpretation of subsurface conditions. However, final interpretation is based on numerous factors and may not reflect the boring logs as presented in Appendix A.

3) Displacements shown along faults are graphic representations. Actual vertical offsets are unknown.

- ▥ SILT
- ▨ CLAY
- ▥ SANDY SILT
- ▨ SANDY CLAY
- ▥ CLAYEY SILT
- ▨ SILTY CLAY
- ▦ SILTY SAND
- ▨ CLAYEY SAND
- ░ SAND
- ░ GRAVELLY SAND
- ▨ SANDY GRAVEL
- ░ GRAVEL
- ▨ GRAVELLY CLAY
- ▤ TAR SILT & CLAY
- ▤ TAR SAND
- ▩ FILL
- ▤ SILTSTONE
- ▤ CLAYSTONE
- ▤ INTERBEDDED SANDSTONE WITH SILTSTONE OR CLAYSTONE
- ░ SANDSTONE
- ░ SANDSTONE, CONGLOMERATE
- xxxxxxx CEMENTED ZONE
- ▩ META—SANDSTONE
- ░ BASALT
- ░ BRECCIA
- ▨ SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale	N/A
Date	MAY, 1984
Prepared by	RG
Checked by	JAD
Approved By	HAS
Project No	83-1140
Drawing No	5

BRU 40 107

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A350 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, and 1983 borings drilled for this investigation. The specific boring logs included are numbered CEG-28, 28-A, 28-B, 28-C, 28-1 through 28-8, CEG-29, 29-A, 29-B and 29-C.

Locations of the borings are shown on Drawings 2 and 3. Ground water observation wells (piezometers) were installed in borings listed in Section 5.4 (Table 5-2). Geophysical downhole and crosshole surveys were made for the 1981 investigation at Boring CEG-28 (see Appendix B).

The borings were drilled to depths ranging from 47 to 217 feet, and penetrated through the alluvium into the underlying bedrock. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was generally good in both the siltstone and claystone bedrock and the alluvium.

The following subsections describe the field exploration procedures and provide explanations of symbols and notation used in preparing the field boring logs. Copies of the field boring logs are presented following the text of this appendix.

A.2 FIELD STAFF AND EQUIPMENT

A.2.1 Technical Staff

Members of the three firms (CCI/ESA/GRC) participated in the drilling exploration program. The field geologist continuously supervised each boring during the drilling and sampling operation. The geologist was also responsible for preparing detailed lithologic log and for sample/core identification, labelling and storage of all samples, and installation of piezometer pipe, gravel pack and bentonite seals.

A.2.2 Drilling Contractor and Equipment

The rotary wash drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failings 750 and 1500 rotary wash rigs, each operated by a two-man crew. The Mobile B-40 of P.C. Explorations was engaged for both rotary wash and rock coring. A Mayhew 1000 rotary wash and man-sized bucket auger equipments of A&W Drilling Company of Brea, California, were also used.

A.3 SAMPLING AND LOGGING PROCEDURES

Logging and sampling were performed in the field by the project geologists. The following describes sampling equipment and procedures and notations used on the lithologic logs to indicate drilling and sampling modes.

A.3.1 Sampling

In the overburden at about 10-foot intervals, the Converse ring sampler was driven using either a down-hole 450-pound or a 340-pound slip-jar hammer. The Converse sampler was followed with the standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the Topanga Formation was encountered, the borings were sampled using a Pitcher Barrel and Converse ring sampler at 20-foot intervals.

The most common cause for loss of samples or altering the sample interval was when gravel was encountered at the desired sampling depth. Standard penetration blow count information can often be misleading in this type of formation, and it is difficult to recover an undisturbed sample. Therefore, at some locations, borings were advanced until drill response and cutting suggested a change in formation.

The following symbols were used on the logs to indicate the type of sample and the drilling mode:

Log Symbol	Sample Type	Type of Sampler
B	Bag	-
J	Jar	Split Spoon
C	Can	Converse Ring
S	Shelby Tube	Pitcher Barrel
Box	Box	Pitcher Barrel, Core Barrel

Log Symbol	Drilling Mode
AD	Auger Drill
RD	Rotary Drill
PB	Pitcher Barrel Sampling
SS	Split Spoon
DR	Converse Drive Sample
C	Coring

A.3.2 Field Classification of Soils

All soil types were classified in the field by the field geologist using the "Unified Soil Classification System". Based on the characteristics of the soil, this system indicates the behavior of the soil as an engineering construction material.* Although particle size distribution estimates were based on volume rather than weight, the field estimates should fall within an acceptable range of accuracy. A description of the Unified Soil Classification Symbols used on boring logs is presented in Table A-1.

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* For a more complete discussion of the Unified Soil Classification System, refer to Corps of Engineers, Technical Memorandum No. 3-357, March 1953, or Department of the Interior, Bureau of Reclamation, Earth Manual, 1963.

TABLE A-1
UNIFIED SOIL CLASSIFICATION SYMBOLS

GRANULAR SOILS

SYMBOL	DESCRIPTION
GW	Well-graded gravels, gravel-sand mixtures, little or no fines
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
GM	Silty gravels, gravel-sand-silt mixtures
GC	Clayey gravels, gravel-sand-clay mixtures
SW	Well-graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, sand-silt mixtures
SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS

SYMBOL	DESCRIPTION
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silty clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays or medium to high plasticity, organic silts
Pt	Peat and other highly organic soils

Table A-2 shows the correlation of standard penetration information and the physical description of the consistency of clays (hand-specimen) and the compactness of sands used by the field geologists for describing the materials encountered.

TABLE A-2 Correlation of N-Values and Consistency/Compactness of Soil Obtained in the Field

N-Values (blows/foot)	Hand-Specimen (clay only)	Consistency (clay or silt)	Compactness (sand only)	N-Values (blows/foot)
0 - 2	Will squeeze between fingers when hand is closed	Very soft	Very loose	0 - 4
2 - 4	Easily molded by fingers	Soft	Loose	4 - 10
4 - 8	Molded by strong pressure of fingers	Firm	---	---
8 - 16	Dented by strong pressure of fingers.	Stiff	Medium dense	10 - 30
16 - 32	Dented only slightly by finger pressure	Very stiff	Dense	30 - 50
32+	Dented only slightly by pencil point	Hard	Very dense	50+

A.3.3 Field Description of the Formations

The description of the formations is subdivided in two parts: lithology and physical condition. The lithologic description consists of:

- rock name;
- color of wet core (from GSA rock color chart);
- mineralogy, textural and structural features; and
- any other distinctive features which aid in correlating or interpreting the geology.

The physical condition describes the physical characteristics of the rock believed important for engineering design consideration. The form for the description is as follows:

Physical condition: ____________ fractured, minimum ______, maximum ________, mostly __________; __________ hardness; ________ strength; __________ weathered.

Bedrock description terms used on the boring logs are given on Table A-3.

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings 28-A, 28-B, 28-5, 28-8, 29, 29-A, 29-B and 29-C. Procedures for piezometer installation were as follows:

A 2-inch diameter plastic ABS pipe was installed in the boring. At least the lower 20 feet of the ABS pipe was perforated, and the annulus of the boring around the perforated portion of the pipe was backfilled with a coarse sand/pea gravel aggregate. Concrete/bentonite slurry was used to backfill around the non-perforated portion of the pipe to prevent surface water from artificially recharging the gravel-packed hole or contaminating local ground water. After the piezometer was installed, the boring was flushed using air lift provided by a trailer-mounted air compressor. The piezometer was covered with a standard 7-inch diameter steel water meter cap held at surface grade by a grouted in-place 3- to 4-foot long, 5-inch diameter plastic sleeve. Ground water data obtained from the piezometers are presented in Section 5.4 of the text.

TABLE A-3 Bedrock Description Terms

PHYSICAL CONDITION*	SIZE RANGE	REMARKS
Crushed	-5 microns to 0.1 ft	Contains clay
Intensely Fractured	0.05 ft to 0.1 ft	Contains no clay
Closely Fractured	0.1 ft to 0.5 ft	
Moderately Fractured	0.5 ft to 1.0 ft	
Little Fractured	1.0 ft to 3.0 ft	
Massive	4.0 ft and larger	

HARDNESS**	
Soft	- Reserved for plastic material
Friable	- Easily crumbled or reduced to powder by fingers
Low Hardness	- Can be gouged deeply or carved with pocket knife
Moderately Hard	- Can be readily scratched by a knife blade; scratch leaves heavy trace of dust
Hard	- Can be scratched with difficulty; scratch produces little powder & is often faintly visible
Very Hard	- Cannot be scratched with knife blade

STRENGTH	
Plastic	- Easily deformed by finger pressure
Friable	- Crumbles when rubbed with fingers
Weak	- Unfractured outcrop would crumble under light hammer blows
Moderately Strong	- Outcrop would withstand a few firm hammer blows before breaking
Strong	- Outcrop would withstand a few heavy ringing hammer blows but would yield, with difficulty, only dust & small fragments
Very Strong	- Outcrops would resist heavy ringing hammer blows & will yield with difficulty, only dust & small fragments

WEATHERING	DECOMPOSITION	DISCOLORATION	FRACTURE CONDITION
Deep	- Moderate to complete alteration of minerals, feldspars altered to clay, etc.	Deep & thorough	All fractures extensively coated with oxides, carbonates, or clay
Moderate	- Slight alteration of minerals, cleavage surfaces lusterless & stained	Moderate or localized & intense	Thin coatings or stains
Little	- No megascopic alteration in minerals	Slight & intermittent & localized	Few stains on fracture surfaces
Fresh	- Unaltered, cleavage surface glistening	None	

*Joints and fractures are considered the same for physical description, and both are referred to as "fractures"; however, mechanical breaks caused by drilling operation were not included.

**Scale for rock hardness differs from scale for soil hardness.

BORING LOG 28

Proj: DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Ground Elev.** 385'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 202'

Hole Diameter 4 7/8" **Hammer Weight & Fall** SS 140 lb @ 30" DR: 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-1.2 ASPHALT			AD	Auger to 10'
2	SC	ALLUVIUM 1.2-9.0 CLAYEY SAND: dark yellowish brown; dry to moist; very loose; occasional fine gravel				
4						
6			J-1	2 1 2	SS	1.5/1.5 recovery
					AD	
8						
	CL	9.0-14.0 SANDY CLAY: dark yellowish brown; moist; stiff				
10			J-2	5 5 5	SS	1.3/1.5 recovery
12					RD	Rotary wash, 4 7/8" drag bit
		becoming more sandy				
14	SC	14.0-19.0 CLAYEY SAND: moderate yellowish brown; moist; loose				
			J-3	3 3 3	SS	1.2/1.5 recovery
16					RD	
18						
	CL	19.0-21.0 SANDY CLAY: moderate yellowish brown;				
20						

Sheet 1 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.0-21.0 SANDY CLAY: (continued) wet; soft	C-1	2 2	DR	1.0/1.0 recovery
22	SC	21.0-23.0 CLAYEY SAND: moderate yellowish brown; wet; loose	J-4	5 3 6	SS	1.3/1.5 recovery
					RD	rig chatter
24	GP	23.0-24.0 GRAVEL: subangular to subrounded; fine to coarse				
	SP	24.0-31.0 SAND: moderate yellowish brown; dense; occasional gravel; wet				
26			J-5	10 15 16	SS	0.7/1.5 recovery
28					RD	
30			J	6 20 24	SS	0.0/1.5 recovery rock stuck in bit
32	SC	31.0-54.8 CLAYEY SAND: moderate yellowish brown; medium dense to dense; wet; occasional fine to coarse gravel			RD	
34						
36			J-6	9 11 12	SS	0.7/1.5 recovery 1/5/81
38					RD	1/6/81 water at 15'
40		becoming silty and dense	C-2	17 13	DR	0.7/1.0 recovery
			J	17 19 20	SS	0.0/1.5 recovery
42					RD	
44						

Sheet 2 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	31.0-54.8 CLAYEY SAND: (continued)			RD	
			J-7	8 11 11	SS	1.1/1.5 recovery
46						
					RD	
48						
50			J-8	9 13 13	SS	1.0/1.5 recovery
	(SP)	interbedded sand				
52					RD	
54	(GP)	- 54.5 thin gravel lens				
		54.8-59.8 SANDY CLAY: moderate brown; moist; very stiff				
	CL		J-9	5 8 8	SS	1.1/1.5 recovery
56						
					RD	
58						
			C-3	6 12	DR	0.7/1.0 recovery
60	SC	59.8-64.7 CLAYEY SAND: moderate yellowish brown; occasional gravel; moist; dense; interbeds of sandy clay and sand	J-10	11 17 17	SS	1.1/1,5 recovery
62					RD	
64						
	SP	64.7-96.5 SAND: moderate yellowish brown; moist; dense; occasional gravel	J-11	20 18 26	SS	1.1/1.5 recovery
66						
					RD	
68						

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	64.7-96.5 SAND: (continued)			RD	
70				29	SS	1.1/1.5 recovery
		becoming very dense	J-12	36		
				34		
72	(GW)	71.5-73.5' gravel lens			RD	chatter
74						
				30	SS	0.5/1.5 recovery
76			J-13	44		
				42		
					RD	
78						
	(SC)	moderate brown; clay increase		15		
			C-4	23	DR	0.7/1.0 recovery
80				37	SS	1.0/1.5 recovery
			J-14	35		
				40		
82		cobbles			RD	rig chatter @ 81.5', cemented sandstone in shoe of SPT
84						rig chatter
		weakly cemented; very dense	J-15	50	SS	0.25/0.25 recovery
					RD	
86						
		increased cementation				
88						
		moderate yellowish brown				
90			J-16	50	SS	0.2/0.2 recovery
92						

Sheet 4 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SP	64.7-96.5 SAND: (continued)			RD	
	(CL)	93.0-94.0' sandy clay				
94						
	(GW)	gravel layer				
			J-17	50	SS	0.3/0.3 recovery
96						intense chatter
	CL	96.5-109.0 SANDY CLAY: moderate brown; moist; very stiff				
98						
			C-5	11 14	DR	1.0/1.0 recovery
100				9 11 16	SS	0.0/1.5 recovery
102					RD	intermittent chatter
104						
			J-18	18 18 23	SS	0.6/1.5 recovery
106					RD	
108						
						rig chatter
110	SP GW	109.0-113.5 SAND/GRAVEL: moderate brown; interbedded, sand with occasional well graded gravel	J-19	50 56	SS	0.7/1.0 recovery
					RD	
112						
114	CL	113.5-118.0 SANDY CLAY: moderate brown; interbedded with clayey sand; moist to wet; dense				
			J-20	18 28	SS	
116						

Sheet 5 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	CL	113.5-118.0 SANDY CLAY: (continued)	J-20	28	SS	1.1/1.5 recovery
					RD	
118	SC	118.0-125.0 CLAYEY SAND: moderate yellowish brown; moist; very dense; interbedded with sandy gravel and clayey gravel				
120			C-6	42 51	DR	1.0/1.0 recovery
			J-21	22 34 50	SS	1.2/1.3 recovery
122					RD	
124	(GW)	gravel lens				chatter
	SP	125.0-134.0 GRAVELLY SAND: moderate yellowish brown; moist to wet; very dense	J-22	62 51	SS	1.0/1.0 recovery
126					RD	
128						
						chatter
130			J-23	56	SS	0.2/0.5 recovery
					RD	
132						slight chatter
134	SC	134.0-156.0 CLAYEY SAND: moderate brown; occasional gravel; moist to wet; very dense				
			J-24	24 45 59	SS	1.3/1.5 recovery
136					RD	
		increasing clay with depth				
138						
			C-7	25 20	SS	1.0/1.0 recovery
140						

Sheet 6 **of** 9

Project DESIGN UNIT A-350 Date Drilled 1/5-7/81 Hole No. 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	SC	134.0-156.0 CLAYEY SAND: (continued)	J-25	41 50	SS	0.8/0.8 recovery
		becoming gravelly			RD	
142						
144						
			J-26	28 41 60	SS	1.3/1.5 recovery 1/6/81
146						
					RD	1/7/81
148						
150			J-27	43 50	SS	0.75/0.75 recovery
					RD	
152		becoming less clayey				
154						
			J-28	50	SS	
					RD	0.3/0.3 recovery
156						
	SP	156.0-178.6 SAND: moderate yellowish brown; moist; very dense; fine to coarse gravel				
158						
			C-8	65 68	DR	1.0/1.0 recovery
160		becoming silty	J	50	SS	0.0/0.25 recovery
					RD	intense chatter
162						
164						

Sheet 7 of 9

Project ____________________ Date Drilled ____________________ Hole No. __________

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	SP	156.0-178.6 SAND: (continued)			RD	
			J	50	SS	0.0/0.3 recovery
166						
	(SM)	167.0-168.5' silty sand				
168						
170		interbedded with fine sand	J-29	39 46 46	SS	1.4/1.5 recovery
172					RD	
174						
		clayey sand with occasional sand lenses	J	28 34 42	SS	0.0/1.5 recovery
176					RD	
178						
	SC	178.6-188.4 CLAYEY SAND: moderate yellowish brown; moist; very dense	C-9	42 50	DR	1.0/1.0 recovery
180			J-30	71 41 50	SS	1.0/1.3 recovery
		occasional gravel			RD	
182						chatter
184						
		thin gravel lenses	J	61	SS	0.0/0.5 recovery
186					RD	
188						

Sheet 8 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188	SC	178.6-188.4 <u>CLAYEY SAND</u>: (continued)			RD	intense chatter
	SC GC	188.4-196.0 <u>CLAYEY SAND/GRAVEL</u>: moderate yellowish brown; moist; dense				
190			J	50	SS	0.0/0.25 recovery
						intense chatter
192						
194						
			J-31	50	SS	0.1/0.1 recovery
					RD	
196	CL	196.0-202.0 <u>SANDY CLAY</u>: moderate yellowish brown; moist; very stiff				
198						
200			C	100	DR	0.0/0.5 recovery
			J-32	58	SS	1.5/1.5 recovery
				51		
				36		
202		B.H. 202.0' Terminated hole; 1/7/81 downhole geophysical survey (GRC) 1/7/81 E-logs (ESA) 1/7/81 water at 75' 1/12/81 cased (4" PVC) and grouted to 100'				
204						
206						
208						
210						
212						

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28A

Proj: DESIGN UNIT A350 **Date Drilled** 3-24-25-81 **Ground Elev.** 410'
Drill Rig FAILING 1500 **Logged By** S. Slaff **Total Depth** 217.5'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lbs 30" (hammer not used)

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.1 ASPHALT 0.1-1.3 CONCRETE			AD	2/24/81
	ML	ALLUVIUM 1.3-61.6 SANDY SILT: moderate yellowish brown; moist; soft				
2						
4						
6						
					RD	
8						
	SP	9.0-9.8 GRAVELLY SAND: moderate yellowish brown; medium dense				rig chatter drill rate 0.3'/minute
10	ML					
12						
	SP	13.0-13.6 GRAVELLY SAND: moderate yellowish brown; medium dense				rig chatter
14	ML					
	SP	15.4-16.2 GRAVELLY SAND: moderate yellowish brown; medium dense; increasing gravel.				rig chatter
16						
18						drill rate 2'/minute
20						

Sheet 1 of 10

Project DESIGN UNIT A350 **Date Drilled** 2-24-81 **Hole No.** 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	ML	1.3-61.6 SANDY SILT: continued			RD	
22		grading coarser				
24	SP	24.0-25.4 GRAVELLY SAND: moderate yellowish brown; medium dense				rig chatter
26	ML					
28						drill rate 0.75'/minute
30						
32		grading less sandy				
34						
36						drill rate 1.1'/minute
38						
40						
42						drill rate 1.5'/minute
44						

Sheet 2 **of** 10

Project DESIGN UNIT A350 **Date Drilled** 2-24-81 **Hole No.** 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	1.3-61.6 SANDY SILT: continued slightly darker; grading sandier			RD	
46						
						drill rate 0.75'/minute
48						
		light olive brown				
50						
52						
54						
	SP	55.0-55.8 GRAVELLY SAND: moderate yellowish brown; medium dense				
56	ML					rig chatter
58						
						drill rate 1'/minute
60						
	SP	60.6-61.6 GRAVELLY SAND: moderate yellowish brown; medium dense				rig chatter
62		TOPANGA FORMATION 61.6-69.0 SANDY SILTSTONE: dark yellowish orange				
64						
66						
68						

Sheet 3 **of** 10

Project DESIGN UNIT A350 **Date Drilled** 2-24-81 **Hole No.** 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	ML	61.6-69.0 SANDY SILTSTONE: continued			RD	
	CL (SM)	69.0-109.0 SILTY CLAYSTONE: mottled: very pale orange, light olive gray, medium gray and dusky brown; moist; very stiff; very thinly interbedded with SILTY SAND: dark yellowish orange; moist; very dense	Box		PB	2.7/2.7 recovery drill rate 0.75'/minute
70						
72						
74						
76						
78						
80						
		dusky yellow to light olive gray				smooth drilling
82						
84						
86		decreasing silty sand				drill rate 0.5'/minute
88						
90		grading sandier with depth				
92						

Sheet 4 of 10

Project DESIGN UNIT A350 Date Drilled 2-24-81 Hole No. 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	CL (SM)	69.0-109.0 SILTY CLAYSTONE: continued			RD	
94						
96						
98		occasional dark gray silty clay; moist; very stiff				
			Box 1		PB	1.6/2.5 recovery cobble dented tube and cut recovery
100						
102						
					RD	
104						
106						
108						rig chatter
		109.0-217.5 CLAYSTONE: olive black; moist; hard; thinly to very thinly interbedded with SILTSTONE: light gray; moist; hard; SANDSTONE: light gray; moist; hard; micaceous	BOX 1		PB	2.5/2.5 recovery
110						
112					RD	
114						drill rate 0.2'/minute
116						

Sheet 5 of 10

Project DESIGN UNIT A350 **Date Drilled** 2-24-25-81 **Hole No.** 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		109.0-217.5 CLAYSTONE: continued			RD	
		massive; friable to low hardness; friable to weak strength; fresh; closely fractured				
118						rig chatter
120						
122						2/24/81
						2/25/81 Gas Test - 0% combustible gas. Water table 18.3' below ground surface.
124						
126						drill rate 0.34'/min.
128						
130						
132		silty sand and silty clay				drill rate 0.45'/min.
134						
136						
						drill rate 0.34'/min.
138						
140						

Project DESIGN UNIT A350 Date Drilled 2-25-81 Hole No. 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		109.0-217.5 CLAYSTONE: continued Physical Condition: massive; friable to low hardness; friable to weak strength; fresh; closely fractured			RD	
142						rig chatter
144						no rig chatter
146						
148						
150						rig chatter
152		near-vertical bedding	Box 1		PB	2.5/2.5 recovery drill rate 0.16'/min.
154					RD	
156						
158						
160						
162						
164						

Sheet 7 of 10

Project DESIGN UNIT A350 Date Drilled 2-25-81 Hole No. 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164		109.0-217.5 CLAYSTONE: continued Physical Condition: massive; friable to low hardness; friable to weak strength; fresh; closely fractured			RD	
166						rig chatter
168						
170						
172						
174						drill rate 0.3'/min.
176						
178						
180						
						drill rate 0.25'/min.
182						
184						
186						
188			Box 2		PB	

Sheet 8 of 10

Project DESIGN UNIT A350 Date Drilled 2/25/81 Hole No. 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		188.0-189.5 SILTSTONE: massive; low to moderate hardness; weak strength; fresh; closely fractured	Box 2		PB RD	1.5/1.5 recovery then refusal
190		109.0-217.5 CLAYSTONE: continued Physical Condition: massive; friable to low hardness; friable to low strength; fresh; closely fractured				
192						
194						
196						drill rate 0.35'/min.
198						
200						
202						drill rate 0.4'/min.
204						
206						
208						drill rate 0.3'/min.
210						
212						

Sheet 9 of 10

Project DESIGN UNIT A350 Date Drilled 2-25-83 Hole No. 28A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
212 214		109.0-217.5 CLAYSTONE: continued Physical Condition: massive; friable to low hardness; friable to weak strength; fresh; closely fractured; moderately cemented; formation			RD	
216		consists of sandy siltstone; light gray; moderately hard; weak; silty claystone; grayish olive; low hardness; weak strength; very thinly to thinly bedded	Box 2		PB	2.0/2.5 recovery drill rate .4'/min. 2/25/81
218		B.H. 217.5' Terminated hole.				2/26/81 0% combustible gas. Water table 24.2' below ground surface. Electric logs run. Installed 2" diameter PVC casing from 0.0-217.5', perforated from 77.5-97.5' and 177.5-212.5'. Set bentonite plug from 51.7-54.0. Installed 1" diameter PVC casing from 0.0-40.0', perforated from 20.0-40.0.
220						
222						
224						
226						
228						
230						
232						
234						
236						

Sheet 10 of 10

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28B

Proj: DESIGN UNIT A 350
Date Drilled 2/14-15-16/83
Ground Elev. 401.0'
Drill Rig Mayhew 1000
Logged By G. Halbert
Total Depth ~~205.5'~~
Hole Diameter 4 7/8"
Hammer Weight & Fall 340 lb, 24" drop

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.4 A.C. PAVEMENT			AD	
	SM	ALLUVIUM 0.4-32.9 SILTY SAND: moderate yellowish brown				
2						
	SP	3.0-5.0 sand layer			RD	
4						
	SM					
6						
8	ML	8.0-10.0 sandy silt layer				
10	SM					
12						drill rate 1.5' to 2'/minute
14						
16						
18	SP	18.0-18.6 sand lens				
20						

Sheet 1 of 9

Project DESIGN UNIT A 350 **Date Drilled** 2/14-17/83 **Hole No.** 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	0.4-32.0 SILTY SAND: continued with interbedded sand and gravelly sand lenses; moderate yellowish brown; medium dense to dense			RD	drill rate 2'/minute
22						
	SP	23.0-24.0 gravelly sand lens				chatter
24	SM					
26	SP	26.0-27.0 gravelly sand lens				chatter
	SM					
28						slight chatter
30	SP	30.0-32.0 sand lens				
32	SC	32.0-41.0 CLAYEY SAND: moderate yellowish brown; very dense				drill rate 1.5'/minute
34						
36						
38						
40						
	SP	41.0-44.0 SAND				
42						
44						

Sheet 2 of 9

Project DESIGN UNIT A350 Date Drilled 2-14-17-83 Hole No. 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM	44.0-51.0 SILTY SAND: moderate yellowish brown			RD	
46						
48						
						drill rate 1.5'/minute
50						
	SW	51.0-60.0 SAND				
52						
54						
56						
						moderate rig chatter
		becoming gravelly				
58						
60	SM	60.0-66.0 SILTY SAND: moderate yellowish brown				
62						
64						
66	GW	66.0-68.0 SANDY GRAVEL				moderate to heavy chatter
68						

Sheet 3 of 9

Project DESIGN UNIT A350 **Date Drilled** 2-14-17-83 **Hole No.** 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW SM	68.0-77.8 SAND: occasional thin gravelly sand lenses			RD	
						intermittent chatter
70						
		occasionally silty				
72						
74		74.0-75.0 cobble lens				heavy chatter
	SC	75.0-76.0 clayey sand lens				
76	SW SM					drill rate 1.4'/minute
78		77.8-86.0 CLAYEY SAND: moderate brown; very dense				
80						
82						
84						
			C-1	8 17	DR	0.8/1.0 recovery
86	SW	86.0-122.0 SAND: very dense				
88						
						2/14/83
90						2/15/83
92						

Sheet 4 of 9

Project DESIGN UNIT A350 **Date Drilled** 2-14-17-83 **Hole No.** 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SW	86.0-122.0 SAND: continued moderate to yellowish brown, light brown; very dense; occasional fine gravel			RD	
94						
						drill rate 0.6'/minute
96						
98		becoming silt moderate yellow brown, very dense				
100						
102	SW/SC	becoming clayey				
104						
						0.5'/minute drill rate
106						
108						
	SW					
110			C-2	35 30	DR	1.0/1.0 recovery
					RD	
112		occasional cobble				heavy chatter
114						drill rate 0.4'/minute
116						

Sheet 5 **of** 9

Project DESIGN UNIT A350 Date Drilled 2/14-17/83 Hole No. 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	SW/ SC	86.0-122.0 SAND: (continued)			RD	
118		occasional clayey zones				heavy chatter
						drill rate 0.2'/minute
120		gravel				heavy chatter
122	SP/ SM	122.0-130.4 BRECCIA ZONE - ANGULAR SANDSTONE FRAGMENTS & SAND/SILTY SAND MIXTURES:				drill rate 0.3.0.4'/ minute
124		mottled colors (light brown, moderate brown, reddish brown, grayish orange), dense to very dense; contains cemented angular sandstone and siltstone fragments, gravelly sand(1" diameter - round) all densely packed at skewed angles				
126						
128						
			C-3	35	No Rec	no recovery
130						
	SP	130.4-136.0 SAND: fine to medium			RD	
132						
134						
136	SM	136.0-139.0 SILTY SAND: moderate yellow brown				
			C-4	30 38	DR	chatter
138					RD	
						0,2/1.0 recovery
	SP	139.0-140.0 SAND lens				
140						

Sheet 6 of 9

Project DESIGN UNIT A350 Date Drilled 2-14-17-83 Hole No. 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	SM/SW	140.0-150.0 SILTY SAND: moderate brown; dense; occasional gravel and occasional yellow brown fragments of SANDSTONE	C-5	25 35	PB-1	0.5/2.2 recovery shelby tube damaged sudden harder drilling, piece of bedrock in cuttings
			PB			
142					RD	
144						
		becoming clayey sand				
146		occasional gravel				drill rate 0.2 to 0.3'/minute intermittent chatter
148						
150	ML/SM	150.0-170.0 SANDY SILT/SILTY SAND: moderate yellow brown; very dense; occasional gravel				
152						
154						
156						occasional chatter
158		becoming clayey				
160						
162						
164						

Sheet 7 of 9

Project DESIGN UNIT A350 **Date Drilled** 2-14-17-83 **Hole No.** 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	ML SM	150.0-170.0 SANDY SILT/SILTY SAND: cont. occasional gravel			RD	
166						
168		becoming more silty light yellow brown				
170	SP	170.0-193.0 SAND: fine grained, silty in places				drill rate 0.5'/minute
172						
174						
176		interbedded silty fine sand				
178		decreasing silt				
180						
182						
184						
186	GP	SILTY SAND: mottled: light yellow brown and moderate yellow brown; very dense; occasional gravel and cobble				
188						

Sheet 8 of 9

Project DESIGN UNIT A350 **Date Drilled** 2-14-17-83 **Hole No.** 28B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188	SP/ GP	170.0-193.0 SAND: continued with occasional gravel and cobble			RD	
190			C-5	80		0.4/0.4 recovery disturbed
			PB-2		PB	
192						
	SP	193.0-205.5 SAND: moderate yellow brown; occasional silty inclusions				2-17-83
194						
196						
198		occasional gravel				chatter
200						
202						
204		occasional gravel				drill rate 0.4'/minute
		B.H. 205.5'				intalled 2" PVC perforated from 155'-205'
206						
208						
210						
212						

Sheet 9 **of** 9

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Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28C

Proj: DESIGN UNIT A-350 **Date Drilled** 10-10-83 **Ground Elev.** 406'

Drill Rig Bucket **Logged By** J. Stellar **Total Depth** 57'

Hole Diameter 32" **Hammer Weight & Fall**

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.2 A.C. PAVEMENT			AD	observation hole no sampling required
		FILL				
	ML	0.2-4.0 SILT: dark brown; firm; moist; with sand				
2						
4		ALLUVIUM				
	SP	4.0-12.0 SAND: light reddish brown; slightly moist; loose to medium dense; occasional silt inclusions; trace of fine gravel				
6						
8						
10						
		coarse gravel				
12						
	SP	12.0-16.0 GRAVELLY SAND: light reddish brown; moist; medium dense; occasional silt inclusions				
14						
		cobbles				
16						
	SP/SM	16.0-42.0 SAND: medium brown; moist; medium dense; silty in places				
18						
20						Sheet 1 of 3

Project DESIGN UNIT A-350 Date Drilled 10-10-83 Hole No. 28C

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	16.0-42.0 SAND: (continued)			AD	
22						
24						
26						
		becoming silty				
28						
30						
32						
34						
		becoming silty				
36		light greenish brown; very moist				
38						
40						
42						
	SP	42.0-45.0 GRAVELLY SAND: brown to light greenish brown; very moist; medium dense				Sheet 2 of 3
44						

Project DESIGN UNIT A-350 **Date Drilled** 10-10-83 **Hole No.** 28C

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	42.0-45.0 GRAVELLY SAND: (continued)			AD	
46	ML	45.0-49.0 SILT: light greenish brown to medium brown; firm; very moist; with lenses of silty sand				
48		becoming dark brown slight petroleum odor				
50	ML	49.0-55.0 CLAYEY SILT: dark brown; very moist; firm to stiff				
52		wet; very strong petroleum odor				standing water @ 52.0'
54		becoming sandy and gravelly				
56	SM	55.0-57.0 SILTY SAND: interlaced with sandy silt; wet				bag sample at 55.0'
58		B.H. 57.0' Terminated hole Terminated due to sloughing. Gas in hole is 3% level. Gasoline (±1") on top of GWT.				case hole to 50.0'; hole belled about 6'-8' at 52' (GWT) but did not cave above 52' when casing was pulled
60						
62						
64						
66						
68						

Sheet 3 of 3

Converse Consultants, Inc.
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BORING LOG 28-1

Proj: DESIGN UNIT A350 **Date Drilled** 2/22-23/84 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** M. Schluter **Total Depth** 100.0'

Hole Diameter 4 7/8" **Hammer Weight & Fall** 325 lb @ 18", SPT 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 A.C. PAVEMENT			C	started drilling
	SM	ALLUVIUM 0.4-5.0 SILTY SAND: moderate brown; trace of gravel; loose; moist				
2			C-1	5 6	DR	2/22/84
					RD	2/23/84 rotary wash
4						
	SW	5.0-13.0 SAND: moderate yellowish brown; trace of fines and gravel; medium dense; moist				
6			PB-1		PB	2.5/2.5 recovery
8			C-2	7 15	DR	
10					RD	
		fine to coarse gravel				
12			J-1	3 7 13	SS	0.1/1.5 recovery end of sampler blocked by coarse gravel
					RD	
	SM	13.0-17.0 SILTY SAND: moderate yellowish brown; trace of gravel; medium dense; moist				
14			C-3	7 13	DR	
16			PB-2		PB	1.9/2.5 recovery
	SW	17.0-28.0 SAND: moderate yellowish brown; trace of gravel; medium dense				
18				7 9	DR	sample not recovered
20					RD	

Sheet 1 of 5

Project DESIGN UNIT A350 Date Drilled 2-23-84 Hole No. 28-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	17.0-28.0 SAND: continued			RD	
		with fine to coarse gravel	J-2	9 16 12	SS	0.7/1.5 recovery gravel reducing recovery by blocking barrel
22					RD	
24			C-4	18 23	DR	
					RD	
26			PB-3		PB	2.0/2.5 recovery tip of sampler damaged by gravels/cobbles
28	GM	28.0-31.5 SANDY GRAVEL: moderate yellowish brown; trace of fines; medium dense	C-5	9 23	DR	disturbed sample
30					RD	rig chatter
			J-3	9 10 15	SS	1.0/1.5 recovery
32	SP/SM	31.5-34.0 SAND/SILTY SAND: moderate yellowish brown; trace of gravel; medium dense				
					RD	
34	SW/SM	34.0-48.0 SAND/SILTY SAND: moderate yellowish brown; trace of gravel; medium dense	C-6	18 23	DR	
					RD	
36			PB-4		PB	damaged barrel with gravels 2.3/2.3 recovery
38			C-7	16 28	DR	
40					RD	
			J-4	6 7 8	SS	1.2/1.5 recovery
42					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A350 **Date Drilled** 2-23-84 **Hole No.** 28-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW SM	34.0-48.0 SAND/SILTY SAND: continued dark yellowish brown; gravel lenses	C-8	49 44	DR	0.4/1.0 recovery
					RD	
46					PB	0.0/2.5 recovery
48	CL ML	48.0-55.0 SILTY CLAY/CLAYEY SILT: light olive gray; some sand; firm; moist to very moist	C-9	8 10	DR	
50					RD	
				1 1 6	SS	0.0/1.5 recovery
52					RD	
54				13 19	DR	no recovery
	SM	55.0-58.0 SILTY SAND: moderate yellowish brown; trace of gravel; medium dense; moist			RD	
56			PB-5		PB	2.0/2.5 recovery
58	SW GM	58.0-61.0 SAND/GRAVELLY SAND: moderate yellowish brown; dense; with gravel lenses	C-10	58 73	DR	
60					RD	
	SP SM	61.0-62.0 SAND/SILTY SAND: moderate yellowish brown; dense; cemented	J-5	99	SS	SPT refusal @ 6"
62	ML SM	62.0-66.0 CLAYEY SILT AND SILTY SAND: moderate yellowish brown; dense			RD	moderate-heavy rig chatter
64			C-11	43 67	DR	top of sample disturbed by cobble
					RD	
66		TOPANGA FORMATION 66.0-100.0 SILTSTONE: moderate yellowish				
68						

Sheet 3 of 5

Project DESIGN UNIT A350 Date Drilled 2-23/23-84 Hole No. 28-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		66.0-100.0 SILTSTONE: continued brown; soft-moderately hard; thinly laminated; interbedded sandstone and claystone; medium to light gray to moderate yellowish brown	PB-6		PB	0.5/0.5 recovery
					RD	
70						
				70	DR	
			C-12	90		
72					RD	
			PB-7		PB	1.9/2.5 recovery
74		very thin olive black and dark gray laminations				
					RD	
76						
78		very thinly laminated sandstone/siltstone; weakly cemented; soft; 60-70'	C-13	93	DR	
				75	RD	
80						
82		moderately hard, moderate to well cemented	PB-8		PB	1.9/2.0 recovery
84					RD	
86		medium gray to dark gray; soft to moderately hard; trace petroleum	C-14	37 62	DR	
					RD	
88						
					PB	
90			PB-9			
					RD	2/22/84
92						

Sheet 4 of 5

Project DESIGN UNIT A350 **Date Drilled** 2-24-84 **Hole No.** 28-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		66.0-100.0 <u>SILTSTONE</u>: continued interbedded sandstone and claystone			RD	
			C-15	60 60	DR	refusal @ 11"
94					RD	
					PB	2.2/2.5 recovery
96			PB-10			
98					RD	
		99.0-100.0 medium light gray to medium dark gray	C-16	30 69	DR	
100		B.O.H. 100' Terminated hole. Filled hole with 3 sac/90 gallon cement slurry into hole.				3/24/84
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 **of** 5

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Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-2

Proj: DESIGN UNIT A350
Date Drilled 11-21-83
Ground Elev. 406'
Drill Rig FAILING 1500
Logged By Moon
Total Depth 90.5'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS: 140 lbs @ 30" DR; 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		6" ASPHALT CONCRETE 6" BASE ROCK			GB	
2	SW	ALLUVIUM 1.0-3.5 SAND: yellowish brown; trace of gravel; moist; loose	C-1	10 9	DR	1.0/1.0 recovery
					AD	
4	SP	3.5-6.5 SAND:moderate brown; moist; medium dense; trace of fines	J-1	6 7 9	SS	
6					RD	
8	SW	6.5-9.0 SAND: moderate yellowish brown; trace of gravel; moist to wet; medium dense	C-2	12 14	DR	1.0/1.0 recovery
					RD	
10	SP	9.0-11.5 SAND: moderately brown; wet; medium dense; trace of silt	J-2	7 7 8	SS	
					RD	
12	SW	11.5-15.5 SAND:light yellowish brown; wet to very dense; trace of gravel	C-3	14 19	DR	1.0/1.0 recovery
					RD	
14			J-3	22 30 32	SS	
16		15.5-26.0 CLAYEY SAND/SILTY SAND: moderate yellowish brown; wet; medium dense			RD	
18	SC/ SM		C-4	16 24	DR	1.0/1.0 recovery
					RD	
20			J-4	4 5	SS	

Sheet 1 of 4

Project DESIGN UNIT A350 **Date Drilled** 11-21-83 **Hole No.** 28-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20		15.5-26.0 CLAYEY SAND/SILTY SAND: moderate yellowish brown; wet; medium dense	J-4	6	SS	
22			C-5	7 12	SR	1.0/1.0 recovery
	SC/ SM				RD	
24			J-5	8 7 6	SS	
26		26.0-32.0 SAND/CLAYEY SAND: moderate yellowish brown; moist; medium dense to dense; trace of gravel			RD	1.0/1.0 recovery
28			C-6	11 26	DR	
					RD	
30	SW/ SC		J	8 15 20	SS	no recovery
					RD	
32	SC	32.0-34.0 CLAYEY SAND: medium brownish grey; wet; medium dense	C-7	8 18	DR	1.0/1.0 recovery "N" valve suspect
					RD	
34	SW	34.0-36.5 SAND: medium brownish grey; wet/ very dense; trace of fines	J-6	22 24 30	SS	
36		36.5-42.0 CLAYEY SAND/SILTY SAND:mottled moderate brown and light brownish grey; dense			RD	
38			C-8	14 21	DR	1.0/1.0 recovery
40	SC/ SM		J-7	13 21 24	SS	
					RD	
42		medium olive green grey 42.0-46.0 SAND: medium grey; trace of fines; wet; dense	C-9	43 55	DR	
44	SP				RD	

Sheet 2 of 4

Project DESIGN UNIT A350 **Date Drilled** 11-21-83 **Hole No.** 28-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	42.0-46.0 SAND: (continued)	J-8	17 18 23	SS	
46		46.0-48.5 CLAYEY SAND/SILTY SAND: moderate yellowish brown; medium dense to dense; wet			RD	
	SC/SM		C-10	15 15	DR	1.0/1.0 recovery
48		48.5-54.0 SANDY CLAY: moderate brownish grey; wet; stiff			RD	
			J-9	2 6 8	SS	
50	CL				RD	
52		becoming silty	C-11	9 13	DR	1.0/1.0 recovery
					RD	
54		54.0-58.5 SAND: medium grey; wet; medium dense; trace of gravel	J-10	13 7 16	SS	
56	SW/SC				RD	
			C-12	28 41	DR	0.8/1.0 recovery
58	SC	58.5-61.5 CLAYEY SAND: medium grey; medium dense; wet			RD	
			J-11	6 7 18	SS	
60		medium dense; wet			RD	
62	SW	61.5-71.0 SAND: medium grey; wet; very dense; becoming gravelly	C-13	14 32	DR	0.6/1.0 recovery
					RD	
64			J-12	22 31 28	SS	
66					RD	0.5/1.0 recovery
68			C-14	77 100	DR	

Sheet 3 **of** 4

Project DESIGN UNIT A350 **Date Drilled** 11-21-83 **Hole No.** 28-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	61.5-71.0 SAND: (continued)			RD	
			J-13	33 47 50	SS	refusal at 15"
70					RD	
	GW	71.0-74.0 SANDY GRAVEL: dense				
72						
74	CL	74.0-78.0 SANDY CLAY: reddish brown; moist hard	J-14	200	DR	refusal at 4"
					RD	
76			C-15	90 95	DR	
					RD	
78	SC	78.0-81.5 CLAYEY SAND: reddish brown; wet dense				
			J-15	20 33 50	SS	
80					RD	
	SW	81.5-83.0 SAND: moderate yellowish brown; very dense; wet				
82			J-16	72	DR	0.3/0.4 recovery refusal at 5"
	SC	83.0-90.5 CLAYEY SAND: reddish brown; wet very dense				
84			J-17	19 28 32	SS	
86					RD	
			C-16	79 100	DR	
88					RD	0.5/0.9 recovery
			J-18	27 32 43	SS	
90						
	B.H.	90.5 Terminate hole			RD	
92						

Sheet 4 **of** 4

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-3

Proj: DESIGN UNIT A350 **Date Drilled** 11-18-83 **Ground Elev.** 399'

Drill Rig Failing 1500 **Logged By** P Moon **Total Depth** 90'

Hole Diameter 4 7/8" **Hammer Weight & Fall** SS:140 lb, 30",DR: 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.1 ASPHALT CONCRETE				
		0.1-0.5 ROCK BASE				
	SM	FILL 0.5-5.0 SILTY SAND: dark brown; moist; loose			GB	
2				4	DR	
			C-1	4		
					AD	
4			J-1	2	SS	
				3		
	SW	ALLUVIUM 5.0-6.0 SAND: moderate yellowish brown; moist; loose		6		
6					RD	
	GW	6.0-7.5 SANDY GRAVEL: moderate brown; wet loose				
				3	DR	0.4/1.0 recovery disturbed sample
8	SC	7.5-13.5 CLAYEY SAND: moderate yellowish brown; wet; medium dense	C-2	5		
					RD	
			J-2	3	SS	
				5		
10				8		
					RD	
12				6	DR	1.0/1.0 recovery
			C-3	9		
					RD	
14	CL	13.5-16.0 SANDY CLAY: moderate yellowish brown; wet; very stiff	J-3	4	SS	
				6		
				11		
16					RD	
	SM	16.0-24.0 SILTY SAND: moderate yellowish brown; wet; medium dense				
				15	DR	0.4/1.0 recovery
18			C-4	29		
					RD	
			J-4	17	SS	
20				10		

Sheet 1 of 4

Project DESIGN UNIT A350 **Date Drilled** 11-18-83 **Hole No.** 28-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	16.0-24.0 SILTY SAND: continued	J-4	12	SS	
					RD	
22				15	DR	1.0/1.0 recovery
			C-5	19		
					RD	
24	SW	24.0-26.5 SAND: moderate yellowish brown; wet; very dense	J-5	24 33 31	SS	
					RD	
26	SC	26.5-60.0 CLAYEY SAND: moderate yellowish brown; medium dense, becoming dense; with gravel		45	DR	
			C-6	33		
28					RD	
				9 13 16	SS	no recovery - rock lodged in drive shoe
30					RD	
32				32	DR	1.0/1.0 recovery
	SM	becoming silty	C-7	36		
					RD	
34			J-6	7 10 18	SS	
36					RD	
				45	DR	0.9/1.0 recovery disturbed sample
			C-8	90		
38					RD	
			J-7	10 16 20	SS	
40					RD	
42				28	DR	1.0/1.0 recovery
			C-9	39		
44					RD	

Sheet 2 of 4

Project DESIGN UNIT A350 Date Drilled 11-18-83 Hole No. 28-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	26.5-60.0 CLAYEY SAND: cont. moderate yellowish brown; wet; very dense		10 14 23	SS	no recovery
46					RD	
				43	DR	1.0/1.0 recovery
48			C-10	54		
					RD	
				15 20 16	SS	no recovery
50					RD	
52				26	DR	1.0/1.0 recovery
	SC CL	silty clay	C-11	29		
					RD	
54			J-8	6 10 15	SS	
56		moderate brown; wet; medium dense			RD	
				12	DR	
58			C-12	12		
					RD	
			J-9	11 12 17	SS	
60	CL	60.0-62.0 SANDY CLAY: moderate brown; very stiff; wet			RD	
62	SC	62.0-65.0 CLAYEY SAND: brown; occasional fine gravel; wet; dense		14	DR	1.0/1.0 recovery
			C-13	18		
					RD	
64			J-10	8 19 28	SS	
	SW SM	65.0-69.0 SAND/SILTY SAND: medium gray; very dense; wet; trace of gravel				
66					RD	
				83	DR	refusal at 11"
68			C-14	117		

Sheet 3 of 4

Project DESIGN UNIT A350 Date Drilled 11-18-83 Hole No. 28-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	65.0-69.0 SAND/SILTY SAND: continued becoming more gravelly			RD	
	GW	69.0-71.- SANDY GRAVEL: medium gray; very dense; wet	J-11	56	SS	
70					RD	
	SW	71.0-78.0 GRAVELLY SAND: moderately greenish gray; wet; very dense; localized cementation				
72			C-15	150 100	DR	0.6/1.0 recovery
					RD	
74			J-12	39 36 32	SS	
76					RD	
		moderately yellowish brown	C-16	100 106	DR	1.0/1.0 recovery
78	SP	78.0-86.0 SAND: medium gray; very dense; wet			RD	
			J-13	23 34 28	SS	
80					RD	
82		becoming silty	C-17	150 80	DR	0.4/1.0 recovery strong gasoline odor
					RD	
84		dark gray	J-14	27 32 29	SS	
86	GW	86.0-90.0 SANDY GRAVEL: very dense			RD	
88						
			C	84 120	DR	no recovery
90		B.O.H. 90'				
92						

Sheet 4 of 4

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Converse Consultants, Inc.
Earth Sciences Associates
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BORING LOG 28-4

Proj: DESIGN UNIT A-350
Date Drilled 11/20/83
Ground Elev. 392'
Drill Rig Failing 1500
Logged By P. Moon
Total Depth 85.0'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS140 lb @ 30", DR: 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT CONCRETE				
		0.4-1.0 GRAVEL BASE			GB	
		ALLUVIUM				
	SP	1.0-3.5 SAND: moderate brown; wet; loose				
2				4	DR	1.0/1.0 recovery
			C-1	4		
					AD	
4	CH	3.5-6.0 SANDY CLAY: moderate greenish brown; wet; very soft	J-1	P P 2	SS	
6					RD	
	SW	6.0-9.0 SAND: moderate brown; wet; loose; trace of gravel				
				5	DR	
			C-2	13		1.0/1.0 recovery
8					RD	
	CL	9.0-13.5 SANDY CLAY: moderate yellowish brown; wet; stiff	J-2	1 3 6	SS	
10						
					RD	
12				6	DR	0.8/1.0 recovery
			C-3	12		
					RD	
14	SC	13.5-41.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense	J-3	1 4 7	SS	
	SC & SM	with silty sand lenses				
16					RD	
				7	DR	1.0/1.0 recovery
			C-4	13		
18					RD	
		occasional fine gravel	J-4	9 11	SS	
20						

Sheet 1 of 4

Project DESIGN UNIT A-350 **Date Drilled** 11/20/83 **Hole No.** 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SC	13.5-41.0 CLAYEY SAND: (continued) moderate yellowish brown; trace of gravel; wet; medium dense to dense	J-4	12	SS	
					RD	
22	SC & SM	with silty sand lenses		13	DR	0.7/1.0 recovery
			C-5	21		
					RD	
24			J-5	5	SS	
				6		
				10		
26					RD	
				20	DR	
28			C-6	46		caliche nodule lodged in drive shoe
					RD	
			J-6	6	SS	
30				12		
				13		
					RD	
32				39	DR	1.0/1.0 recovery
			C-7	51		
					RD	
34			J-7	12	SS	
				15		
				19		
36					RD	
				33	DR	0.8/1.0 recovery
38			C-8	46		
					RD	
			J-8	14	SS	
40				8		
				10		
	SW	41.0-53.0 SAND: moderate yellowish brown; wet; dense to very dense			RD	
42				33	DR	1.0/1.0 recovery
			C-9	47		
44					RD	

Sheet 2 **of** 4

Project DESIGN UNIT A-350 Date Drilled 11/20/83 Hole No. 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW	41.0-53.0 SAND: (continued) moderate yellowish brown; wet; very dense	J-9	21 25 29	SS	
46					RD	
			C-10	23 29	DR	1.0/1.0 recovery
48		becoming silty			RD	
		thin interbeds of clayey fine sand	J-10	29 23 23	SS	
50					RD	
52			C-11	36 44	DR	1.0/1.0 recovery
	SC	53.0-64.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense			RD	
54	SC & SM	with silty sand lenses	J-11	7 13 14	SS	
56					RD	
			C-12	18 37	DR	1.0/1.0 recovery
58					RD	
				10 11 19	SS	no recovery
60					RD	
62		scattered gravel	C-13	24 29	DR	1.0/1.0 recovery
					RD	
64	SW	64.0-73.5 GRAVELLY SAND: mottled yellowish brown and pinkish brown; wet; very dense; with silty sand	J-12	69	SS	
66					RD	
68			C-14	72 105	DR	

Sheet 3 of 4

Project DESIGN UNIT A-350 **Date Drilled** 11/20/83 **Hole No.** 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	64.0-73.5 GRAVELLY SAND: (continued) moderate yellowish brown; wet; very dense			RD	
				72	SS	
70					RD	
72				76	DR	
			C-15	120		
					RD	
74	SW SM	73.5-78.0 SAND/SILTY SAND: moderate brown; wet; very dense; trace of gravel	J-13	33 40 51	SS	
76					RD	
			C-16	98 98	DR	
78	GW	78.0-85.0 SANDY GRAVEL: moderate brown; wet; very dense			RD	
				15 9 17	SS	no recovery
80					RD	
82				200	DR	no recovery, cobble lodged in dirve shoe refusal at 5"
					RD	
84						
		B.H. 85.0' Terminated hole				
86						
88						
90						
92						

Sheet 4 of 4

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Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-5

Proj: DESIGN UNIT A350 **Date Drilled** 11-19-83 **Ground Elev.** 387.5'

Drill Rig Failing 1500 **Logged By** P. Moon **Total Depth** 100.0'

Hole Diameter 4 7/8" **Hammer Weight & Fall** SS:140 lb, 30", DR 320 lbs. @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS /6"	DRILL MODE	REMARKS
0		0.0-0.8 ASPHALT CONCRETE				
		0.8-1.0 BASE ROCK			GB	
		ALLUVIUM				
2	SM	1.0-8.5 SILTY SAND: moderate brown; moist; loose				
				9	DR	1.0/1.0 recovery
			C-1	9		
					AD	
4			J-1	2	SS	
				3		
				4		
6					RD	
				6	DR	1.0/1.0 recovery
8			C-2	7		
					RD	
	CL	8.5-16.0 SANDY CLAY: moderate yellowish brown; moist; stiff				
			J-2	4	SS	
10				4		
				5		
					RD	
12				4	DR	1.0/1.0 recovery
			C-3	7		
					RD	
14			J-3	4	SS	
				4		
				6		
16					RD	
	SC	16.0-18.5 CLAYEY SAND: moderate yellowish brown; moist; medium dense				
				9	DR	1.0/1.0 recovery
18			C-4	13		
					RD	
	SW	18.5-23.5 SAND: moderate yellowish brown;				
			J-4	10	SS	
20				12		

Sheet 1 of 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	18.5-23.5 SAND: continued medium dense to dense; wet	J-4	19	SS	
					RD	
22				113 30	DR	no recovery, refusal at 6-1/2"
	SC	23.5-31.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense to dense			RD	
24				29 31 24	SS	no recovery
26					RD	
				31	DR	1.0/1.0 recovery
			C-5	30		
28					RD	
				10 13 16	SS	no recovery
30						
					RD	
	SW	31.0-38.0 SAND: moderate yellowish brown; wet; dense to very dense				
32				41	DR	1.0/1.0 recovery
			C-6	62		
					RD	
34			J-5	17 21 29	SS	
36					RD	
				41		0.8/1.0 recovery
			C-7	47		
38	SC	38.0-44.5 CLAYEY SAND: moderate yellowish brown; wet; dense			RD	
			J-6	12 13 17	SS	
40						
					RD	
42				21	DR	1.0/1.0 recovery
			C-8	24		6 rings
					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC SM	38.0 -44.5 CLAYEY SAND: continued 44.5-53.5 SILTY SAND: moderate brown; wet; very dense	J-7	21 28 37	SS	
46					RD	
			C-9	32 47	DR	1.0/1.0 recovery
48					RD	
50			J-8	17 18 18	SS	
					RD	
52			C-10	28 32	DR	1.0/1.0 recovery
					RD	
54	SW	53.5-56.0 SAND: moderate yellowish brown; wet; very dense; trace of gravel	J-9	22 28 46	SS	
56	SC	56.0-61.0 CLAYEY SAND: moderate brown; wet; very dense			RD	
58			C-11	53 92	DR	1.0/1.0 recovery
					RD	
60			J-10	13 30 41	SS	
					RD	
62	SW	61.0-66.0 SAND: moderate brown; moist; very dense	C-12	46 46	DR	1.0/1.0 recovery
					RD	
64		grading to GRAVELLY SAND	J-11	73	SS	refusal at 6"
66	GP	66.0-68.5 SANDY GRAVEL: wet; very dense				
68			C-13	75 75	DR	0.6/1.0 recovery

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GP	66.0-68.5 SANDY GRAVEL: continued			RD	
	SW	68.5-73.5 SAND: moderate yellowish brown; wet; very dense	J-12	62	SS	
70					RD	
72				130	DR	0.6/0.8 recovery
			C-14	60		refusal at 10"
					RD	
74	SM	73.5-83.5 SILTY SAND: moderate brown; wet; very dense	J-13	18 27 48	SS	
76					RD	
				29	DR	1.0/1.0 recovery
78			C-15	35		
					RD	
			J-11	7 11 18	SS	
80						
					RD	
82		scattered fine gravel		29	DR	1.0/1.0 recovery
			C-16	45		
					RD	
84	GW	83.5-88.0 SANDY GRAVEL: wet; very dense	J-15	76	SS	refusal at 6"
					RD	
86						
88	SW	88.0-97.0 SAND: wet; very dense				piezometer set at 100'
90						
92						

Sheet 4 **of** 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SW	88.0- 99.5 SAND: continued			RD	
94						
96						
	GW	grading gravelly				
98						
				32	DR	
	CL	99.5-100.0 SANDY CLAY: moderate yellowish brown; wet; very stiff	C-17	68		
100		B.O.H. 100'				following completion of drilling, prior to installation of piezometer, fluid level dropped to near T.D. = 100'
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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Converse Consultants, Inc.
Earth Sciences Associates
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BORING LOG 28-6

Proj: DESIGN UNIT A350 **Date Drilled** 11-16-83 **Ground Elev.** 385.5'

Drill Rig Failing 1500 **Logged By** P. Moon **Total Depth** 82.5'

Hole Diameter 4 7/8" **Hammer Weight & Fall** SS 140 lb 30"; DR: 320 lbs. @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.75 CONCRETE 0.75-1.0 BASE ROCK			GB	
	CL	ALLUVIUM 1.0-3.5 SANDY CLAY: dark yellowish brown; moist; stiff				
2				3	DR	0.6/1.0 recovery
			C-1	6		
					AD	
4	SC	3.5-6.5 CLAYEY SAND: dark yellowish brown; moist; medium dense				
			J-1	4 4 7	SS	
6					RD	
	CL	6.5-16.0 SANDY CLAY: dark yellowish brown; moist; stiff		6	DR	0.8/1.0 recovery
8			C-2	9		
					RD	
			J-2	4 6 8	SS	
10		becoming clayey			RD	
12				9	DR	1.0/1.0 recovery
			C-3	11		
					RD	
		trace of gravel				
14			J-3	3 5 7	SS	
					RD	
16	SW	16.0-18.5 SAND: moderate yellowish brown; moist; medium dense				
				12	DR	1.0/1.0 recovery
			C-4	16		
18					RD	
	SC	18.5-22.0 CLAYEY SAND: moderate yellowish				
20			J-4	5 10	SS	

Sheet 1 of 4

Project DESIGN UNIT A350 **Date Drilled** 11-16-83 **Hole No.** 28-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SC	18.5-22.0 CLAYEY SAND: continued brown; moist; medium dense	J-4	9	SS	
					RD	
22	CL	22.0-25.5 SANDY CLAY: moderate yellowish brown; moist; medium dense		12	DR	1.0/1.0 recovery
			C-5	13		
					RD	
24			J-5	11 13 19	SS	
					RD	
26	SC	25.5-27.5 CLAYEY SAND: moderate yellowish brown; moist; wet; medium dense to dense				
				22	DR	1.0/1.0 recovery
	SP	27.5-31.5 SAND: moderate yellowish brown; moist; dense	C-6	29		
28					RD	
			J-6	9 22 23	SS	
30						
					RD	
32	SC	31.5-51.5 CLAYEY SAND: moderate yellowish brown; moist; medium dense; with sandy clay		28	DR	1.0/1.0 recovery
			C-7	38		
					RD	
	SC & SM	with silty sand lenses				
34			J-7	9 14 15	SS	
36					RD	
				17	DR	1.0/1.0 recovery
38			C-8	36		
					RD	
			J-8	7 10 12	SS	
40						
					RD	
42				22	DR	1.0/1.0 recovery
			C-9	37		
					RD	
44						

Sheet 2 of 4

Project DESIGN UNIT A350 **Date Drilled** 11-16-83 **Hole No.** 28-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	31.5-51.5 CLAYEY SAND: continued moderate yellowish brown; wet; medium dense to dense; trace of gravel	J-9	7 14 16	SS	
					RD	
46	SC & SM	with silty sand lenses				
			C-10	21 36	DR	1.0/1.0 recovery
48					RD	
			J-10	16 20 21	SS	
50					RD	
52	SW/SM	51.5-53.5 GRAVELLY SAND: moderate yellowish brown;wet; very dense; silty in places	C-11	24 50	DR	refusal at 11"
					RD	
54	GW	53.5-56.0 SANDY GRAVEL: brown; wet; very dense	J-11	50	SS	
					RD	
56	SC	56.0-61.5 CLAYEY SAND: moderate brown; wet; very dense		110	DR	no recovery refusal at 4"
58						
			J-12	30 50	SS	11/16/83 11/17/83
60					RD	
62	SW	61.5-67.0 GRAVELLY SAND: moderate brown; wet; very dense		110 50	DR	no recovery refusal at 9"
					RD	
64			J-13	39 50	SS	refusal at 10"
					RD	no recovery
66						
68	SC	67.0-81.0 CLAYEY SAND:		115	DR	refusal at 9"

Sheet 3 of 4

Project DESIGN UNIT A350 Date Drilled 11-17-83 Hole No. 28-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SC	67.0-81.0 CLAYEY SAND: continued moderate yellowish brown; wet; very dense			RD	
			J-14	12 20 32	SS	
70						
					RD	
72				44 34		
			P-1			
74						
			J-15	29 33 37		
76						
		scattered fine to coarse gravel				
78						
			C-12	183		refusal at 4"
80						
	SW	81.0-82.5 GRAVELLY SAND: wet; very dense				
82			J-16	63		refusal at 6"
		B.O.H. 82.5'				following sample J-16, boring caved to 58' Attempted to redrill to 82'. Boring continued to cave. Grout seal placed.
84						
86						
88						
90						
92						

Sheet 4 of 4

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Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-7

Proj: DESIGN UNIT A350
Date Drilled 11-19-20-83
Ground Elev. 382.5'
Drill Rig FAILING 750
Logged By St. Slaff
Total Depth 99.9'
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lbs., 30"SS,320 lb, 18" DR

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT			AD	
		0.4-0.7 BASE ROCK				
	CL	0.7-2.8 SANDY CLAY: moderate brown; dry; soft; micaceous				
2		2.5 becoming moist; stiff				
				2	DR	1.0/1.0 recovery
	SM	2.8-9.4 SILTY SAND :moderate brown; moist; loose; minor steel debris; trace of fine gravel				
4			C-1	3		
					AD	
			J-1	3	SS	1.5/1.5 recovery
				4		
6				4		
					RD	
8		8.8 decreasing fines; moderate yellowish brown		4	DR	0.4/1.0 recovery
			C-2	5		
					RD	
10	CL	ALLUVIUM				
		9.4-32.5 SANDY CLAY: moderate yellowish brown; moist; stiff; trace of gravel	J-2	2	SS	
				5		
				6		
12					RD	
		13.0 becoming more sandy				
				3	DR	0.9/1.5 recovery
14			C-3	5		
					RD	
		becoming less sandy	J-3	2	SS	1.3/1.5 recovery
16				4		
				5		
					RD	
18			PB-1		PB	
						0.2/2.5 recovery
20						

Sheet 1 of 5

Project DESIGN UNIT A 350 **Date Drilled** 11-19-20-83 **Hole No.** 28-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	9.4-32.5 SANDY CLAY: (continued)			PB	
		increasing sand with depth	J-4	3 6 6	SS	0.8/1.5 recovery
22					RD	
		occasional gravel		9	DR	0.6/1.0 recovery
			C-4	42		
24					RD	
			J-5	5 6 8	SS	0.5/1.5 recovery
26					RD	slight rig chatter
28				6	DR	
			C-5	8		
					RD	
30		becoming very stiff	lost	5 8 11	SS	0.0/1.5 recovery
32					RD	11-19-83 11-20-83 ground water level 15.2'
	SM	32.5-39.6 SILTY SAND: moderate yellowish brown; trace of gravel; moist medium dense		13	DR	0.9/1.0 recovery
34			C-6	16		
					RD	minor rig chatter
			J-6	7 10 10	SS	
36					RD	minor rig chatter
38			PB-2		PB	1.6/2.5 recovery
40	CL	39.6-53.0 SANDY CLAY:moderate yellowish brown; moist; stiff	J-7	5 6 11	SS	
42					RD	0.7/1.5 recovery
		increasing sand with depth		9	DR	0.8/1.0 recovery
44			C-7	17		

Sheet 2 of 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-20-83 **Hole No.** 28-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	39.6-53.0 SANDY CLAY: (continued)			RD	
			J-8	5 5 7	SS	0.5/1.5 recovery
46		becoming very stiff			RD	
48				10	DR	0.0/1.0 recovery
			lost	14		
					RD	
50			J-9	7 11 18	SS	0.9/1.5 recovery
52					RD	
	SM	53.0-54.6 SILTY SAND: moderate yellowish brown; moist; very dense; with gravel	C-8	27 50	DR	0.9-0.9 recovery refusal at 11" rig chatter
54						
	ML/ SM	54.6-84-5 SANDY SILT/SILTY SAND: moderate yellowish brown; wet; very stiff to hard, medium dense to very dense; occasional gravel	J-10	8 12 9	SS	0.8/1.5 recovery heavy rig chatter
56					RD	
58			PB-3		PB	rig chatter 0.5/2.5 recovery tube damaged/sample disturbed
60			J-11	37 40 27	SS	0.4/1.5 recovery
62					RD	
	GP	63.8-64.0 gravel lens	C-9	22 16	DR	
64					RD	0.5/1.0 recovery
			J-12	27 37 19		0.6/1.5 recovery
66					RD	
68						

Sheet 3 **of** 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-20-83 **Hole No.** 28.7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	ML/ SM	54.6-84.5 SANDY SILT/SILTY SAND: (continued)	C-10	23 33	DR	1.0/1.0 recovery
					RD	
70			J-13	20 29 17	SS	0.6/1.5 recovery
					RD	
72	GW	72.0-72.8 sandy gravel lens				rig chatter
			C-11	32 47	DR	0.8/1.0 recovery
74					RD	
			J-14	27 47 40	SS	0.9/1.5 recovery
76					RD	
78			PB-4		PB	
80			J-15	27 50	SS	0.3/0.75 recovery
					RD	
82						violent rig chatter
			C-12	60 50	DR	0.75/0.75 recovery refusal at 9"
84					RD	violent rig chatter
	CL	84.5-87.0 SANDY CLAY: mottled-moderate brown; greyish green and dark grey; moist; hard				
			J-16	4 14 18	SS	0.7/1.5 recovery
86					RD	
	CL	87.0-92.2 SILTY CLAY: mottled moderate brown; light brown; moderate yellowish brown; moist, hard; trace of sand				rig chatter
88			C-13	35 32	DR	1.0/1.0 recovery
					RD	
90			J-17	13 19 20	SS	
92					RD	

Sheet 4 **of** 5

Project DESIGN UNIT A310 **Date Drilled** 11-19-20-83 **Hole No.** 28-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	CL SM	87.0-92.2 SILTY CLAY: (continued) 92.2-99.3 SILTY SAND: moderate brown; wet; very dense; trace of gravel			RD	
			C-14	44 50	DR	0.9/0.9 recovery
94					RD	refusal at 11"
			J-18	19 27 20	SS	0.5/1.5 recovery
96					RD	
98		99.3-99.9 SAND/SILTY SAND: moderate brown; wet; very dense				
	SP/SM		C-15	32 50	DR	0.9/0.9 recovery 11-20-83
100		B.H. 99.9' terminate hole				11-21-83 groundwater level 36.3 hole filled with 3 sack cement grout, cleaned site, covered hole with steel street cover 11-29-83 removed steel cover capped hole with concrete
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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BORING LOG 28-8

Proj: DESIGN UNIT A350
Date Drilled 2-21-84
Ground Elev.
Drill Rig FAILING 1500
Logged By M. Schluter
Total Depth 100.0'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS: 325 lbs. @ 18", 140 lbs @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.2 A.C.PAVEMENT			C	
	SM	0.2-2.4 SILTY SAND - FILL: moderate yellowish brown; trace of gravel; loose; moist				
					A	
				12	DR	
2			C-1	42		
		2.4-3.0 CONCRETE SLURRY				
					A	
4		4.0-9.0 SILTY SAND: moderate yellowish brown; very loose to loose; moist; trace of gravel; concrete & brick fragments				
				2	SS	
6				3	140	1.3/1.5 recovery
			J-1	4		Rotary wash
					RD	
8		ALLUVIUM				
		9.0-15.0 SAND: moderate yellowish brown; trace of fines; loose to medium dense; moist;				
10				8	DR	
			C-2	10	325	0.0-/2.5 recovery
12						
		gravel lenses				
14					PB	no recovery
	SM/SW	15.0-21.0 SILTY SAND/SAND moderate brown; moist; trace to little gravel		9	DR	
16			C-3	10	325	
					RD	
18						slight rig chatter
				9	SS	
				8	140	1.0.1.5 recovery
			J-2	9		
20						

Sheet 1 of 5

Project DESIGN UNIT A350 Date Drilled 2-21-84 Hole No. 28-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM/ SW	15.0-21.0 SILTY SAND/SAND:(continued)			RD	
	SW	21.0-28.0 SAND/GRAVELLY SAND: dark yellowish brown; medium dense to dense; moist				
22			C-4	24 27	DR 325	
					RD	
24			PB-1		PB	1-7-2.3 recovery
26		fine to coarse gravel	C-5	30 36	DR 325	
					RD	
28	SM/ SP	28.0-33.0 SILTY SAND/GRAVELLY SAND: dark yellowish brown; medium dense to dense				
			J-3	14 14 18	SS 140	0.9/1.5 recovery
30					RD	
32			C-6	16 17	DR 325	
	SW	33.0-35.0 SAND: dark yellowish brown; trace of fines & gravel; medium dense			RD	
34			PB-2		PB	1.9-2.5 recovery
	SM	35.0-36.5 SILTY SAND: medium dense				
36			C-7	14 16	DR 325	
	SW/ SP	36.5-44.0 SAND: moderate yellowish brown trace to little gravel; medium dense			RD	
38						rig chatter
			J-4	11 16 24	SS 140	
40					RD	
	SW	gravel lenses				rig chatter
42				20 12	DR 325	no recovery rig chatter
					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A 350 Date Drilled 2-21/22-84 Hole No. 28-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW/ GW	44.0-51.0 SAND/SANDY GRAVEL: moderate yellowish brown; medium dense to dense	PB-3		PB	2.2/2.5 recovery
46						
			C-8	26 27	DR 325	
					RD	
48						
			J-5	15 12 15	SS	1.0/1.5 recovery
50					RD	
		51.0-56.5 CLAYEY SILT: moderate yellowish brown; moist; stiff; trace gravel;				
52	ML		C-9	14 15	DR 325	
					RD	2.5/1.5 recovery
54			PB-4		PB	
56		BEDROCK 56.5-100.0 SANDSTONE/SILTSTONE: pale yellowish brown and light brown; soft to moderately hard; very thinly laminated; slightly moist to moist				
			C-10	47 73	DR 325	
					RD	
58						
	30-40					
60			C-11	29 75	DR 325	refusal at 11"
					RD	
62						
64			PB-5		PB	2.4/2.5 recovery
66					RD	
68						

Sheet 3 of 5

Project DESIGN UNIT A350 **Date Drilled** 2-22-84 **Hole No.** 28-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		56.5-100.0 SANDSTONE/SILTSTONE: (continued) with interbedded claystone; medium grey sandstone; dark grey;siltstone; trace of petroleum			RD	
70				47	DR	
			C-12	79	325	
					RD	slight oil film developing on drilling fluid tub -petroleum slight rig chatter at 74'
72						
74						
			PB-6		PB	
76						
					RD	
78						
80				80	DR	refusal @ 9"
			C-13	50	325	
					RD	
82						
84						
	40-45°	moderately hard	PB-7		PB	2.2/2.2 recovery
86					RD	rig chatter
88						
90		cemented		35	DR	
			C-14	108	325	
					RD	
92						

Sheet 4 **of** 5

Project DESIGN UNIT A350 Date Drilled 2-22-84 Hole No. 28-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92					RD	
						0.3/0.3 recovery
94		94.0-96.0 hard; well cemented sandstone; medium light grey; fractured; thinly bedded; little weathered	PB-8		PB	rig chatter (94'-96')
96						
98						variable rig chatter
			C-15	150	DR 325	refusal @ 4" disturbed sample
100	B.H.	100.0' Terminated Hole				2-22-84 installed 100' piezometer, perforated from 80-100'; backfilled with pea gravel 2-24-84, water level 40.2' below street level, installed casing and cover
102						
104						
106						
108						
110						
112						
114						
116						

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 29

Proj: DESIGN UNIT A-350
Date Drilled 1/19-23/81
Ground Elev. 417'
Drill Rig Mobile B-40
Logged By D. Gillette
Total Depth 209.8'
Hole Diameter 3", 6"
Hammer Weight & Fall 140 lbs @ 15-18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		FILL 0.0-0.4 CONCRETE			AD	
	SM	ALLUVIUM 0.4-16.5 SILTY SAND: moderate brown; trace of gravel; occasional thin silty clay lenses; moist; medium dense			RD	
2						
4						
6						
8						
10						
			J-1	6 10 10	SS	1.0/1.5 recovery
12					RD	
14		14.5-15.0 gravel				rig chatter
16						
	GP	16.5-20.5 SANDY GRAVEL and COBBLES: light brown and brownish gray; trace of fines; moist; dense				rig chatter
18						
20					PB	

Sheet 1 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	GP	16.5-20.5 SANDY GRAVEL and COBBLES: (cont)			PB	no recovery
		WEATHERED TOPANGA FORMATION 20.5-36.0 SANDY CLAYSTONE: mottled gray inclusions and streaks; moist; firm to very stiff				
22			J-2	5 8 10	SS	0.9/1.5 recovery
		Physical Condition: massive or little fractured; soft to friable hardness; friable strength; deep weathered			RD	
24						
26						
28						
		29.0-29.6 cobbles or boulders				
30					PB	cobble pushed through clay no recovery
32			J-3	7 10 13	SS	1.2/1.5 recovery
34					RD	
36		36.0-42.0 SANDY CLAYSTONE/SILTSTONE: moderate yellow brown and medium gray; thin sandy claystone beds with siltstone interbeds				
38		38.2-39.8 fracture zone				
		Physical Condition: closely to moderately fractured; friable to low hardness; weak to moderate strength; deep to moderate weathering				
40	10°		Box 1		PB	2.0/2.0 recovery
	55°					
42	35°	42.0-45.0 SILTY CLAYSTONE: light brown Physical Condition: massive or little fractured; soft; plastic; moderate weathered	J-4	11 30 45	SS	1.5/1.5 recovery pocket penetrometer 3.5 tsf (broke apart) 2/
44			Box 1		PB	

Sheet 2 **of** 9

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		42.0-45.0 <u>SILTY CLAYSTONE</u>: (continued)	Box 1		PB	2.1/2.1 recovery 1/19/81 1/20/81
		45.0-209.8 <u>SANDSTONE</u>: moderate yellowish brown and medium light gray; moderate yellowish brown beds with very thin medium light gray clayey siltstone interbeds; contains dark brown organic inclusions; lightly cemented;				
46					RD	
48	80°		Box 2		C	1.1/1.1 recovery
50	60°	Physical Conditions: moderate fractured; low hardness; weak strength; moderate to little weathered				
52		50.0-209.8 sandstone is medium light gray and grayish black; occurs as thin sandstone beds with very thin grayish black siltstone and claystone interbeds				4.9/4.9 recovery
54						
56						
58						2.8/5.0 recovery
			Box 3			
60	15°	60.0-68.0 claystone beds				
						2.4/3.0 recovery
62	10°					
64						
		65.0-65.2 well cemented sandstone; medium light gray; very dense				
66		66.0 cemented				4.2/5.0 recovery
						4.0/5.0 recovery
68	10°					

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	10°	45.0-209.8 SANDSTONE: (continued)	Box 3		C	
70	65°	70.2-70.5 cemented (as at 65')				
			Box 4			4.0/5.0 recovery
72						
	10°					
74						
						4.5/4.5 recovery
76						
78	10°					
80						5.0/5.0 recovery
			Box 5			
82						
84						
	50°					4.9/5.0 recovery
86						
88	36°					
90						4.8/5.0 recovery
			Box 6			
92						

Sheet 4 of 9

Project DESIGN UNIT A-350 Date Drilled 1/19-23/81 Hole No. 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	40° 30°	45.0-209.8 <u>SANDSTONE</u>: (continued) 92.0-92.5 cemented sandstone	Box 6		C	
94	60°					5.0/5.0 recovery
96	40°	Physical Condition: as previously described				
98	39° 40°	99.0' cemented sandstone (as at 65.0')				
100	45°		Box 7			5.0/5.0 recovery 1/20/81 1/21/81
102	70° 65°					
104	80° 45°					3.5/3.5 recovery
106	55° 55° 45°					
108			Box 8			5.0/5.0 recovery
110	50° 65° 70°					gas test: O_2 - 21% combustibles - 0%
112	82°	113.0-118.0' brecciated sandstone				4.8/4.8 recovery
114						
116						

Sheet 5 of 9

Project DESIGN UNIT A-350 Date Drilled 1/19-23/81 Hole No. 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		45.0-209.8 SANDSTONE: (continued)	Box 8		C	
			Box 9			
118	50°					
		119.0-119.2 cemented sandstone (as at 65.0')				5.0/5.0 recovery
120	30°	120.0-123.5 slate: very hard; claystone interbed				
122	10°					2.6/3.2 recovery
		123.5-132.2 medium light gray with thin slate interbeds				
124						2.0/2.0 recovery
126						
			Box 10			
128	70°	Physical Condition: closely to intensely fractured; moderate hard to hard; moderate strong; little weathered				4.5/4.5 recovery
130						
						3.0/3.0 recovery
132		132.2-209.8' sandstone as at 50.0'				
134	70°					
	75°					
136						4.5/5.0 recovery
	45°		Box 11			
138	45°					5.0/5.0 recovery
	30°					
140	85°					

Sheet 6 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	45° 30°	45.0-209.8 SANDSTONE: (continued)	Box 11		C	5.0/5.0 recovery
142						
144						4.0/4.0 recovery
146	20°		Box 12			
148	50° 50°	Physical Condition: 146-151' intensely fractured; friable; weak; little weathered				
150						4.7/5.0 recovery
152						
154	55° 45°		Box 13			3.5/3.5 recovery 1/21/81 1/22/81
156	55° 60°					
158	50°					5.0/5.0 recovery
160	60°					
162	50°					5.0/5.0 recovery
164	65°					

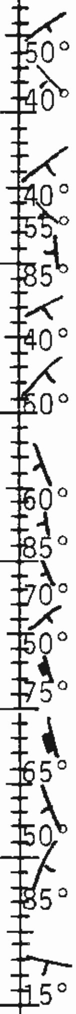
Sheet 7 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	60° 80°	45.0-209.8 SANDSTONE: (continued) Physical Condition: 165-175' intensely fractured; low to moderate hardness; moderate to weak strength; little to fresh weathering	Box 14		C	
166	70° 85° 30°					
168						5.0/5.0 recovery
170						
172						5.0/5.0 recovery
174	45°		Box 15			
176	45°					
178	15°					4.3/4.3 recovery 1.5/1.5 recovery
180	30°					
182	65° 50°	182.0-209.8' brecciated shear zone	Box 16			
184	52° 50°					5.0/5.0 recovery
186	60° 80°					
188						

Sheet 8 **of** 9

Project DESIGN UNIT A-350 **Date Drilled** 1/19-23/81 **Hole No.** 29



DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		45.0-209.8 SANDSTONE: (continued)	Box 16		C	5.0/5.0 recovery
		182-209.8 brecciated shear zone continued				
190		Physical Condition: 182-209.8' closely to intensely fractured; low hardness; moderate strong; little to fresh weathering				
192			Box 17			
194						5.0/5.0 recovery
196						
198						5.0/5.0 recovery
200						
202			Box 18			
204						5.0/5.0 recovery
206						
208		208.0 cemented sandstone				
						5.0/5.0 recovery
210		B.H. 209.8' Terminated hole water sampled 2/25/81				
212						Sheet 9 of 9

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 29A

Proj: DESIGN UNIT A-350
Date Drilled 2/11-14/83
Ground Elev. 418'
Drill Rig Maynew 1000
Logged By G. Halbert
Total Depth 69'
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb @ 30", 340 lb @ 24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 A.C. PAVEMENT			C	
	ML	FILL 0.5-4.0 CLAYEY SILT: contains sand			RD	
2						
4	GW	ALLUVIUM 4.0-18.0 SAND:				
6		6.0-7.0' gravelly sand layer				moderate chatter
8						
10		fine gravelly sand				moderate to heavy chatter continuous light to moderate chatter
12		occasional siltysand lens				
14						drill rate: 1'/min.
16						
18	ML	18.0-36.0 SANDY SILT: moderate yellowish brown; very stiff; very moist				
20						

Sheet 1 of 4

Project DESIGN UNIT A-350 **Date Drilled** 2/11-14/83 **Hole No.** 29A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	ML	18.0-36.0 SANDY SILT: (continued) with gravelly sand lenses; moderate yellowish brown; very moist; very stiff to hard			RD	
22						drill rate 0.8'/min.
24						
	(SP)	25.0-25.8' sand lens				moderate chatter
26						
28	(SP)	28.0-28.8' gravelley sand lens				moderate chatter
30						
32						
34						
		gravelly lens				
36	SM	36.0-46.0 SILTY SAND: moderate yellowish brown; dense				drill rate 2'/min
38						
40						
42						
44						

Sheet 2 **of** 4

Project DESIGN UNIT A-350 **Date Drilled** 2/11-14/83 **Hole No.** 29A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM	36.0-46.0 SILTY SAND: (continued)			RD	
46	SW	46.0-60.0 SAND: contains fine gravel; trace of fines				
48						
50						drill rate 3'/min.
52						
54						
	(SP)	fine gravelly sand lens				
	(SM)	silty sand with gravel				
56						
	(GW)	gravel lens				heavy chatter
58						
60	SM/ML	60.0-65.0 SILTY SAND/SILT: moderate yellowish orange with dusky yellowish brown silt				
62						
						drill rate 0.4'/min
64						
66		TOPANGA FORMATION - BEDROCK 66.0-69.0 SANDSTONE: medium gray; fine with very thin darker gray silt layers; also dark yellowish orange weathering stains; jointed, otherwise massive;				
68			C-1	18 20	DR	

Sheet 3 of 4

Project DESIGN UNIT A-350 **Date Drilled** 2/11-14/83 **Hole No.** 29A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		66.0-69.0 SANDSTONE: (continued) friable strength; friable to low hardness 66.0-67.0' very hard with possible basalt intrusion	J-1	25 45	SS	
70						installed 2" pvc perforated from 39' to 69'
		B.H. 69.0' Terminated hole				
72						
74						
76						
78						
80						
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
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BORING LOG 29B

Proj: DESIGN UNIT A350
Date Drilled 2/10-11/83
Ground Elev. 421'
Drill Rig Mayhew 1000
Logged By G. Halbert
Total Depth 47'
Hole Diameter 4 7/8"
Hammer Weight & Fall SPT 140lb, 30", C 340lb, 20"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 A.C. PAVEMENT			C	
	CL	FILL 0.5-3.0 CLAYEY SILT: dark yellowish brown; very stiff; low plasticity			RD	
2						
4	SW	ALLUVIUM 3.0-22.6 SAND: mixed white, black, red, brown, dense				
6						
		occasional silty sand lenses				
8						
						light chatter
10						
12						
	SP	fine gravel				moderate chatter
14						
16	GP	15.5-16.6 gravel layer				moderate chatter
						moderate to heavy chatter
18						
20						

Sheet 1 of 3

Project DESIGN UNIT A350 **Date Drilled** 2/10-11/83 **Hole No.** 29B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	3.0-22.6 SAND: (continued)			RD	
						drill rate: 0.5 min.
22						
	SM	22.6-26.5 SILTY SAND: moderate yellow brown; dense to very dense; very moist				
24						
26						
	SW/GP	26.5-36.0 SAND & GRAVEL: mixed black, red brown, pale green, little fines; dense to very dense				light chatter
28						
30						
32						
34						
	SM	34.0-35.0 silty sand lens				
		gravel				drill rate 0.5'/min.
36						
		BEDROCK 36.0-46.5 SANDSTONE: interbedded siltstone mottled dark yellow orange & medium grey; moist; weathered; moderately fractured; friable; weak strength, low hardness				
	35°		C-1	7 11	DR	1.0/1.0 recovery
38						
						penetrometer 4.0 tsf
40						
						drill rate 0.3'/min.
42						
44						

Sheet 2 of 3

Project DESIGN UNIT A350 **Date Drilled** 2/10-11/83 **Hole No.** 29B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		36.0-46.5 SANDSTONE: massive			RD	
			J-1	21 34 46	SS	1.5/1.5 recovery
46	45°					
	B.H. 46.5 Terminated Hole					installed P.V.C. 2" diameter 0-46' perforated from 26' to 46'
48						
50						
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 **of** 3

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 29C

Proj: DESIGN UNIT A350
Date Drilled 2-10-83
Ground Elev. 451'
Drill Rig Mayhew 1000
Logged By G. Halbert
Total Depth 28'
Hole Diameter 4 7/8 "
Hammer Weight & Fall SPT 140 lb., 30", C 340lb., 24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		6" CONCRETE PAVEMENT			RD	
	ML	FILL 0.5-5.0 SANDY SILT: moderate yellowish brown; stiff; moist				
2						
4						
	ML	ALLUVIUM 5.0-7.0 SANDY SILT & GRAVEL:				
6	GW	gravel and cobbles				
	ML	7.0-15.2 SANDY SILT				
8						
10						
		greyish orange; very stiff to hard				
12						
14						
				12	SS	
	30°	TOPANGA FORMATION 15.2-28 BEDROCK	J-1	23		1.5/1.5 recovery
16	faint	SILTSTONE with interbedded SANDSTONE: mottled pale yellow orange; dark yellow; orange & medium grey; moist; moderately weathered; friable strength		30		
18						
		19.0-19.4 cemented zone				moderate chatter @19'
20						

Sheet 1 of 2

Project DESIGN UNIT A350 **Date Drilled** 2-10-83 **Hole No.** 29C

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	5	15.2-28.0 SILTSTONE interbedded with SANDSTONE: (continued)	J-2	18 39 50	SS	1.3/1.3 recovery refusal @ 15" penetrometer 4.0 tsf (broke apart)
22		little weathered			RD	
24						
26						
	19°		C-1	14 25	DR	penetrometer(no penetration)
28		B.H. 28' Terminate Hole				28' P.V.C. casing installed
30						
32						
34						
36						
38						
40						
42						
44						

Sheet 2 **of** 2

Appendix B

Geophysical Exploration

APPENDIX B GEOPHYSICAL EXPLORATION

B.1 DOWNHOLE SURVEY

B.1.1 Summary

Downhole shear wave velocity surveys were performed in Boring CEG-28 for Design Unit A350. Measurements were made at 5-foot intervals from the ground surface to depths of 190 feet. A description of the technique and a summary of the results are attached.

B.1.2 Field Procedure

Shearing energy was generated by using a sledge hammer source on the ends of a 4-by-6-inch timber positioned under the tires of a station wagon, tangential to the borehole. A 12-channel signal enhancement seismograph (Geometrics Model ES1210) allowed the summing of several blows in one direction when necessary to increase the signal-to-noise ratio. Shear waves were identified by recording wave arrivals with opposite first motions on adjacent channels of the seismograph.

B.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure B-1. The timing line shows a 20 millisecond (MS) break at the end of the record, indicating that each vertical line is 10 MS. The time of the first arrivals of compressional shear energy is indicated by P and S, respectively. Wave arrival records similar to Figure B-1 were analyzed to estimate wave travel times and velocities for CEG-28.

B.1.4 Discussion of Results

Estimated velocity structures are summarized in Table B-1. Velocity estimates are based on selection of linear portions of the downhole arrival time curves (see Figures B-2).

The error analysis performed for these surveys involved a least squares fit of these data by estimating the mean of the slope ($\bar{V}$) in Table B-1 and the standard deviation of this estimate of the slope. This estimate of the standard deviation was combined with an estimate of the overall accuracy to produce the best estimated velocity (V^*). V_p^* are the values to be used for studies of the response of these sites. N is the number of data points used for the straight line fit for each velocity estimate.

In general, the near-surface shear wave velocity was found to be approximately 1000 feet per second. To depths of about 190 feet, shear wave velocity estimates generally increased to 1400 feet per second.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed also in Boring CEG-28. The crosshole technique for determining shear wave velocities of in-situ materials was utilized in a three-borehole array. The array consisted of boring CEG-28 and two additional holes drilled approximately 15 feet away. All boreholes were drilled to a depth of 100 feet. Compressional wave and shear wave velocities are presented in Table B-2.

B.2.2 Field Procedure

The shear wave hammer is placed in an end hole of the array, and vertical geophones are placed in the remaining two boreholes. The shear wave generating hammer and the two geophones are lowered to the same depth in all boreholes. The hammer is coupled to the wall of the hole by means of hydraulic jacks, and the geophones are coupled by means of expanding heavy rubber balloons which protrude from one side of the geophone housings. The hammer is then used to create vertically polarized shear waves with either an up or down first motion. A 12-channel signal enhancement seismograph with oscilloscope and electrostatic paper camera is used as a signal storage device. Seismic wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet (see Figures B-3 and B-4).

B.2.3 Data Analysis

For the data analysis actual crosshole distances were determined to within ±0.01 feet. These distances were computed between each of the three boreholes at the elevations of shear measurements. From the crosshole records (seismograms), the travel times for both compressional and shear wave arrivals at each borehole and at each depth were measured. Shear wave arrivals were identified by the reversed first motion on the seismograms. Compression and shear wave estimates were based on the wave arrival records.

B.2.4 Discussion of Results

The shear wave velocity (V_s) is equal to the difference in travel path distance from the shear source to each geophone divided by the difference in shear wave arrival times. The results of the compressional and shear wave velocity analyses are shown in Table B-2. It should be noted that compression wave velocities below the ground water table may be masked by the compression wave response of the water (V_c = 5000 fps) particularly in highly porous materials.

TABLE B-1
DOWNHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
28	15- 55	1579	22	79	9	1580±100	943	87	47	8	940±130
	55- 85	2233	134	112	7	2230±250	1138	200	57	7	1140±260
	85-135	5169	255	258	11	5170±510	1448	39	72	11	1450±110
	135-190	6788	386	339	11	6790±420	1380	114	69	11	1380±180

$\bar{V}p$ = mean estimate of compressional wave velocity.

$\bar{V}s$ = mean estimate of shear wave velocity.

σp = standard deviation of estimated compressional wave velocity.

σs = standard deviation of estimated shear wave velocity.

Ep = estimated accuracy of compressional survey.

Es = estimated accuracy of shear survey.

Np = number of points used for straight line fit of compressional wave.

Vp* = overall accuracy of compressional wave velocity estimate.

Vs* = overall accuracy of shear wave velocity estimate.

Ns = number of points used for straight line fit of shear wave velocity data.

TABLE B-2
CROSSHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
28	10	----					765	17	38	8	770±60
	15	3000				3000±300	834	11	42	12	830±50
	20	2500				2500±250	749	18	37	8	750±60
	25	----					925	44	46	16	930±90
	30	2220				2000±200	973	28	49	16	970±80
	35	2300				2300±200	993	74	50	16	990±120
	40						1039	76	52	12	1040±130
	45	2140				2100±200	1036	36	52	10	1040±90
	50	1880				1900±200	1102	46	55	12	1100±100
	55	2140				2100±200	1123	16	56	16	1120±70
	60	2000				2000±200	1097	8	55	16	1100±60
	65	2100				2100±200	1018	8	51	16	1020±60
	70	2000				2000±200	1274	61	64	12	1270±130
	75	1800				1800±200	1222	38	61	16	1200±100
	80	1800				1800±200	1477	114	74	16	1480±190
	85	2300				2300±200	1863	106	93	16	1860±200
	90	6000				6000±600	1712	476	86	16	1712±560
	95	7500				7500±750	1550	204	77	4	1550±280
	97	7500				7500±750	1730	79	86	12	1710±170

$\bar{V}p$ = mean estimate of compressional wave velocity.
$\bar{V}s$ = mean estimate of shear wave velocity.
σp = standard deviation of estimated compressional wave velocity.
σs = standard deviation of estimated shear wave velocity.
Ep = estimated accuracy of compressional survey.
Es = estimated accuracy of shear survey.
Np = number of points used for straight line fit of compressional wave.
Vp* = overall accuracy of compressional wave velocity estimate.
Vs* = overall accuracy of shear wave velocity estimate.
Ns = number of points used for straight line fit of shear wave velocity data.

Approved for publication 4/84 by JAD

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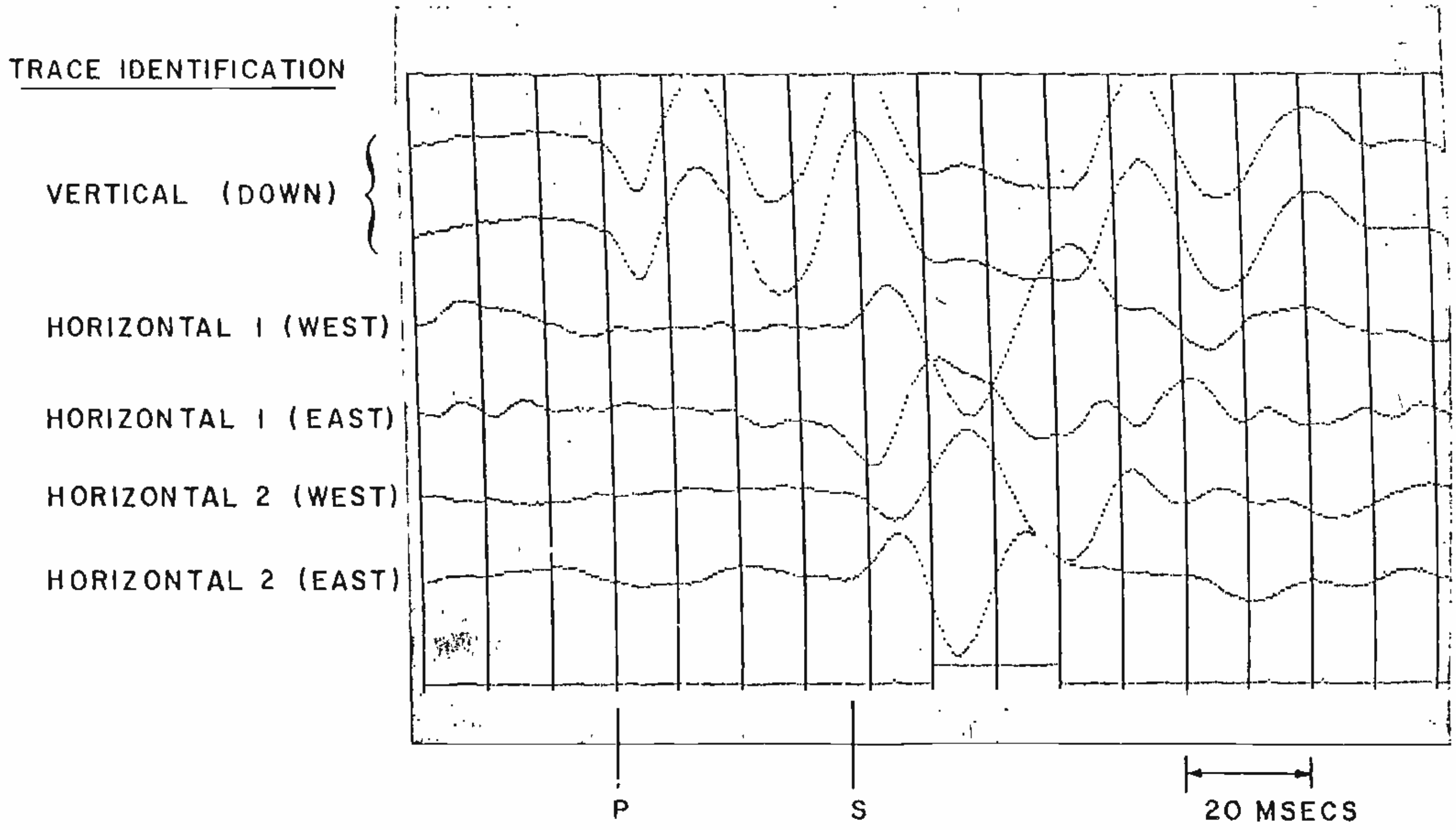
Geotechnical Engineering
and Applied Sciences

Southern California Rapid Transit District
METRO RAIL PROJECT
DESIGN UNIT A350

DOWNHOLE SAMPLE RECORD

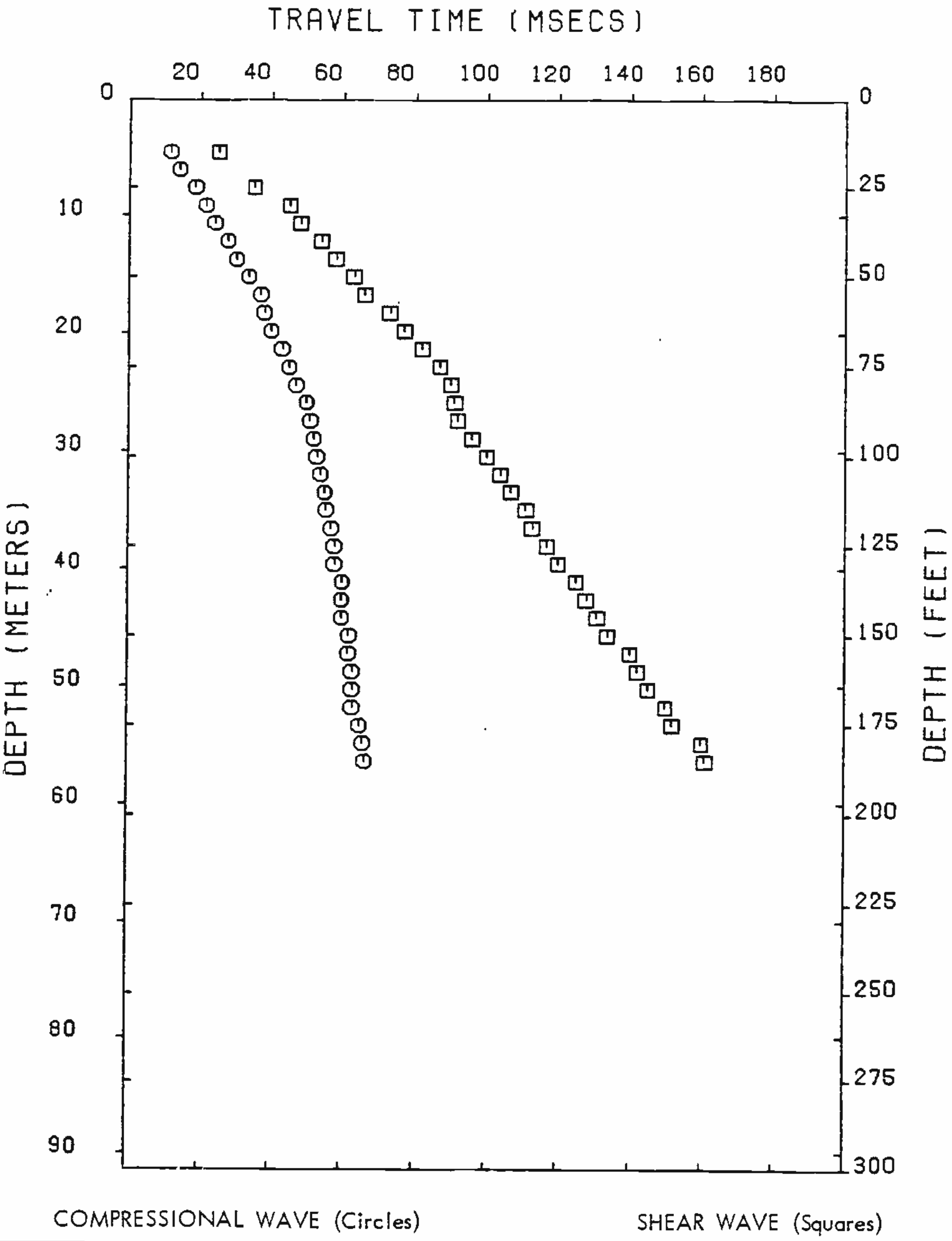
Project No.
83-1140

Figure No.
B-1



BOREHOLE: 13

DEPTH: 70 FT



DOWNHOLE TRAVEL TIME PROFILE - BORING 28

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

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Figure No.
B-2

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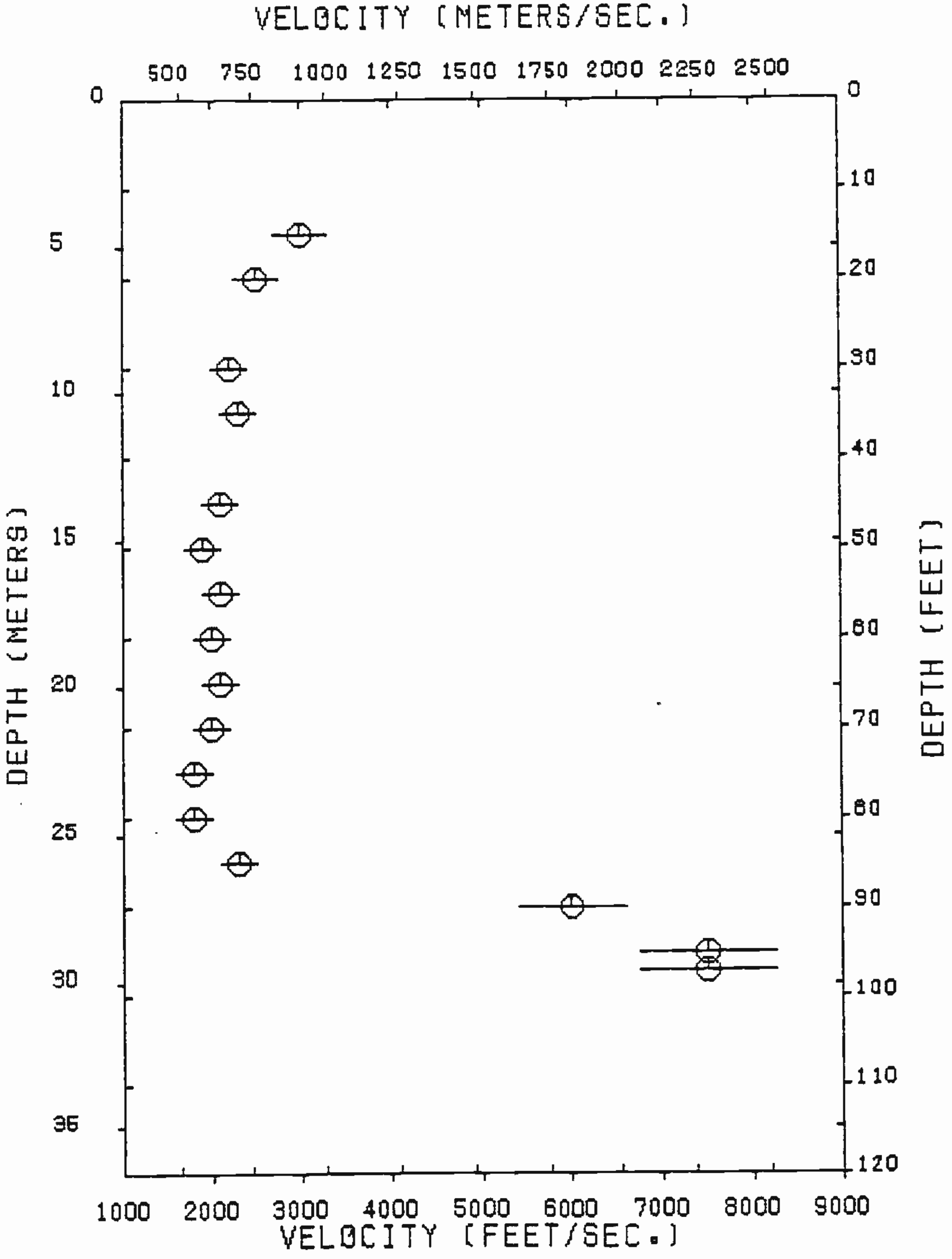

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING 28

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

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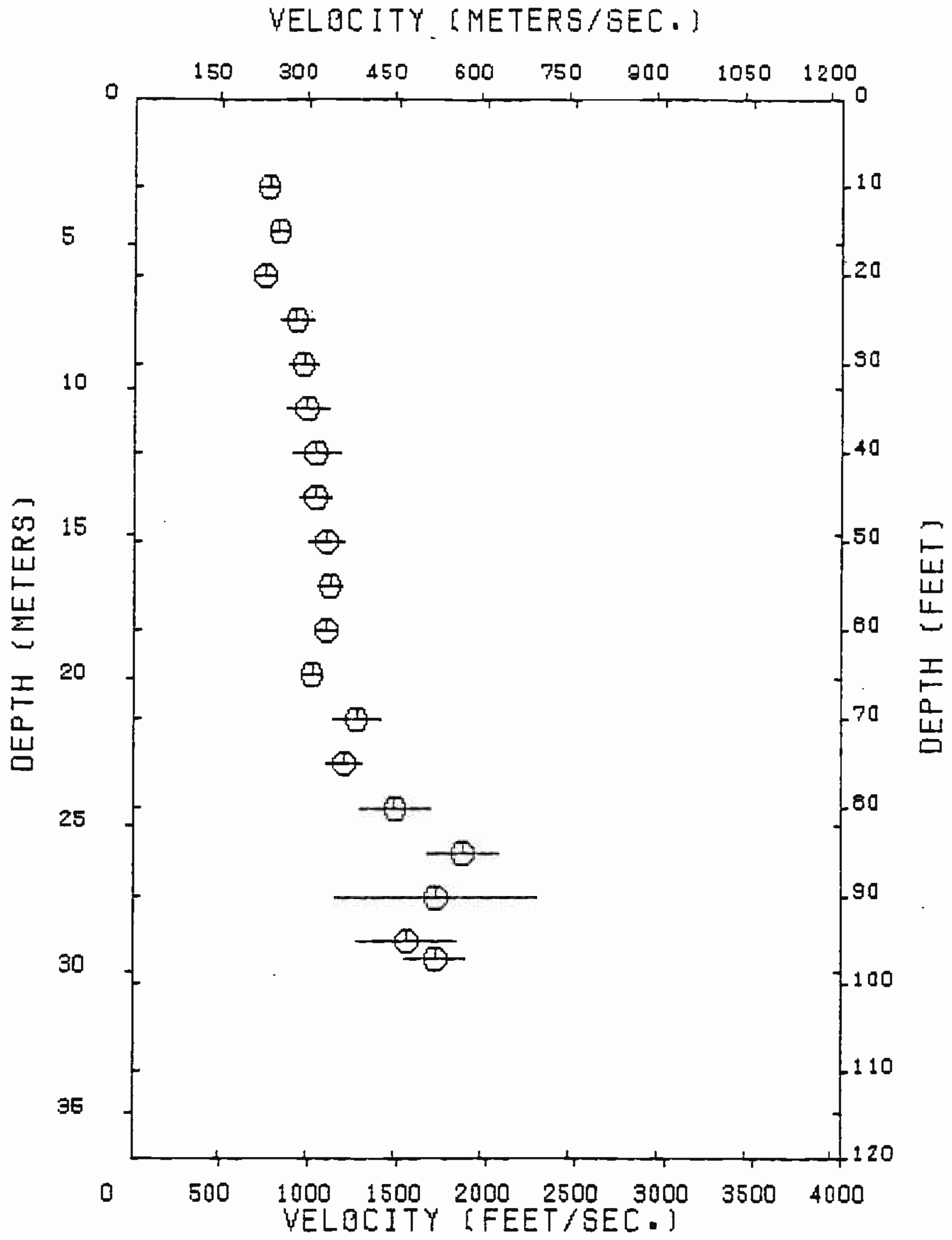
Figure No.
B-3

Approved for publication ______ by ______



SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING 28

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83-1140

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Figure No.
B-4

Approved for publication ________ by ________

Appendix C

Geotechnical Laboratory Testing

APPENDIX C GEOTECHNICAL LABORATORY TESTING

C.1 INTRODUCTION

This appendix presents laboratory geotechnical tests performed on selected soil and bedrock samples obtained from the borings drilled at the Hollywood/ Cahuenga Station site.

The soil tests performed may be classified into two broad categories:

- Index or identification tests which included visual classification, grain-size distribution, Atterberg Limits, moisture content, and unit weight testing;
- Engineering properties testing which included unconfined compression, triaxial compression, direct shear, consolidation, permeability, and resonant column.

The laboratory test data from the present investigation are presented in Table C-1, while data from the 1981 geotechnical investigation are presented in Table C-2. The geologic units listed in these tables are described in Section 5.0 of the report. Figures C-1 through C-11 summarize strength and modulus data for fine-grained alluvium, coarse-grained alluvium, and bedrock at this site.

C.2 INDEX AND IDENTIFICATION

C.2.1 Visual Classification

Field classification was verified in the laboratory by visual examination in accordance with the unified Soil Classification System and ASTM D-2488-69 test method. When necessary to substantiate visual classifications, tests were conducted in accordance with the ASTM D-2478-69 test method.

C.2.2 Grain-Size Distribution

Grain-size distribution tests were performed on representative samples of the geologic units to assist in the soils classification and to correlate test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D-422-63 test method. Combined sieve and hydrometer analyses were performed on selected samples which had a significant percentage of soil particles passing the No. 200 sieve. Results of these analyses are presented in the form of grain-size distribution or gradation curves on Figures C-12 through C-19.

It should be noted that the grain-size distribution tests were performed on samples secured with 2.42- and 2.87-inch ID samplers. Thus, material larger than those dimensions may be present in the natural deposits although not indicated on the gradation curves.

C.2.3 Atterberg Limits

Atterberg Limit Tests were performed on selected soil samples to evaluate their plasticity and to aid in their classification. The testing procedure was in accordance with ASTM D-423-66 and D-424-59 test methods. Test results are presented on Figures C-20 and 21, and Tables C-1 and C-2.

C.2.4 Moisture Content

Moisture content determinations were performed on selected soil samples to assist in their classification and to evaluate ground water location. The testing procedure was the ASTM D-2261 test method. Test results are presented on Tables C-1 and C-2.

C.2.5 Unit Weight

Unit weight determinations were performed on selected undisturbed soil samples to assist in their classification and in the selection of samples for engineering properties testing. Samples were generally the same as those selected for moisture content determinations.

The test procedure entailed measuring specimen dimensions with a precision ruler or micrometer. Weights of the sample were than determined at natural moisture content. Total unit weight was computed directly from data obtained from the two previous steps. Dry density was calculated from the moisture content found in Section C.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables C-1 and C-2.

C.2.6 Specific Gravity and Porosity

A determination of soil particle specific gravity of several representative soil and rock samples was made to allow determination of the soil/rock porosity. Specific gravity was determined in accordance with the ASTM D-854 test method. Soil porosity was determined based on the specific gravity and the dry unit density of the material. Results of these determinations are presented in Table C-1.

C.3 ENGINEERING PROPERTIES: STATIC

C.3.1 Unconfined Compression

Unconfined compression tests were performed on selected samples of cohesive soils and bedrock from the test borings for the purpose of evaluating the undrained, unconfined shear strength of the various fine-grained geologic units. The tests were performed in accordance with the ASTM D-2166 test method. Results of the unconfined compression tests are presented on Tables C-1 and C-2.

C.3.2 Triaxial Compression

Consolidated undrained triaxial compression tests were performed on selected undisturbed soil samples. The tests were conducted in the following manner:

C.3.2.1 Consolidated Undrained (CU) Tests

- The undisturbed test specimen was trimmed to a length to diameter ratio of approximately 2.0.
- The specimen was then covered with a rubber membrane and placed in the triaxial cell.
- The triaxial cell was filled with water and pressurized, and the specimen was saturated using back-pressure.
- When saturation was complete, the specimen was consolidated at the desired effective confining pressure.
- After consolidation, an axial load was applied at a controlled rate of strain. In the case of the undrained test, flow of water from the specimen was not permitted, and the resulting pore water pressure change was measured.
- The specimen was then sheared to failure or until a maximum strain of 15% to 20% was reached.

Some of the tests were performed as progressive tests. The procedure was the same as above except that, when the soil specimen approached but did not reach failure (usually to peak effective stress ratio), the axial load was removed and the specimen was consolidated at a higher confining pressure. The axial load was again applied at a constant rate of strain, and the load was removed before the specimen failed. This process was repeated a third time at a still higher confining pressure, and the sample was loaded until failure occurred.

Results of the triaxial compression tests are presented on Figures C-22 through C-27.

C.3.3 Direct Shear

Direct shear tests were performed on selected undisturbed soil samples using a constant strain rate direct shear machine.

Each test specimen was trimmed, soaked and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until a maximum shear strength was developed. Fine-grained samples were allowed to consolidate prior to shearing. The maximum developed shear strengths are summarized on Tables C-1 and C-2.

Progressive direct shear tests were performed on selected undisturbed samples of coarse-grained material. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. A new normal load was then applied, and the specimen was sheared a second time. This process was repeated for several different normal loads. Results of the progressive direct shear tests are summarized on Tables C-1 and C-2.

C.3.4 Consolidation

Consolidation tests were performed on selected undisturbed soil samples placed in 1 inch high by 2.42-inch diameter brass rings, or 3-inch diameter Shelby tubes trimmed to a 2.42-inch diameter.

Apparatus used for the consolidation test is designed to receive the 1 inch high brass rings directly. Porous stones were placed in contact with both sides of the specimens to permit ready addition or release of water. Loads were applied to the test specimens in several increments, and the resulting settlements recorded.

Results of consolidation tests on the undisturbed samples are presented on Figures C-28 through C-32.

C.3.5 Permeability

Permeability tests were performed on undisturbed specimens selected for testing, or in conjunction with the static and cyclic triaxial tests, using the same selected undisturbed samples of soil. Permeability was measured during back-pressure saturation by applying a differential pressure to the ends of the sample and measuring the resulting flow. Results of the tests are tabulated on Tables C-1 and C-2.

C.4 ENGINEERING PROPERTIES: DYNAMIC

C.4.1 Resonant Column

The resonant column test determines the shear modulus and damping of soil specimens at shear strain values of approximately 10^{-6} to 10^{-4} inches per inch. A solid cylindrical soil specimen is encased in a thin membrane, placed in a pressure cell and subjected to the desired ambient stress conditions. The specimen is caused to vibrate at resonance in torsion by fixing one end and applying sinusoidally varying torque to the free end. The response of the soil specimen is measured using an accelerometer coupled to the free end. Shear modulus and damping values are calculated from the response data.

C.4.1.1 Sample Preparation and Handling

The test apparatus used for this procedure accepts a 1.4-inch-diameter by approximately 3.5-inch-length specimen. Undisturbed samples were prepared by trimming the 1.4-inch-diameter samples from the larger Shelby, Pitcher or Converse ring samples.

C.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures, (σ_{3c}), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressures and progressing to the higher confining pressures. At each confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures C-33 and C-34.

C.4.1.3 Apparatus

The device used in this test program was designed and built by Soil Dynamics Instruments, Inc, of Lexington, Kentucky, and is sometimes referred to as a Hardin Oscillator, after Dr. B.O. Hardin, the designer. Essentially, it consists of the main component groups listed below.

- Pressure Cell and Frame: The unit is made of aluminum with a transparent plexiglass cylinder designed for maximum operating pressures of approximately 150 psi. The bottom specimen end cap is brass and affixed to the base of the unit.

  Pressure lines and fittings are provided to pressurize the cell and for back pressure or sample drainage, if desired. A pneumatic device is also provided to support the weight of the excitation device during specimen setup.

- Excitation Device: This mechanism consists of a torque-producing apparatus mounted on the underside of a hollow stainless steel cylinder. Its mass is very large in comparison to the test specimen. The driving torque is produced by a system of electromagnetic coils attached to the cylinder and permanent magnets coupled to the top specimen load cap through a system of restoring springs. The device is driven by an audio oscillator having a range of approximately 20 Hz to 40 kHz. Because the device is designed to have a large mass in comparison to the specimen, a lever and weight system supports the weight of the device during the test. A strain gauge load cell is built into the excitation device to monitor the axial load applied to the specimen. The driving torque is determined by measuring the voltage drop across a precision resistor in series with the electromagnetic coils.

- Accelerometer and Charge Amplifier: A Columbia Research Labs accelerometer is attached to the excitation device. The accelerometer output is amplified by a charge amplifier, and the system is calibrated to produce output voltage in proportion to the amplitude of angular displacement of the excitation device, and thus of the specimen. Shear strains are calculated from the amplitude of angular displacement.

- Readout Devices: Output voltages produced by the accelerometer, load cell-bridge system, and driving torque are recorded by digital multimeter. Resonance of the specimen is determined using a cathode ray oscilloscope connected to display the Lissajous pattern.

C.4.1.4 Data Reduction

Data obtained from the resonant column tests were reduced in accordance with the ASTM "Suggested Methods of Test for Shear Modulus and Damping of Soils by the Resonant Column"* using a proprietary computer program developed by Converse Consultants, Inc. Graphs of the test results are presented on Figures C-33 and C-34.

C.4.2 Dynamic Triaxial Compression

This test evolved from the static triaxial procedure and is designed to evaluate the stress-strain properties of the soils under dynamic loading conditions. This test differs from the cyclic triaxial test in that it is designed to obtain dynamic stress-strain data at various strain levels, while the cyclic test measures deformation and liquefaction susceptibility at a given level of cyclic stress. Shear strain data is obtained generally in the range of 10^{-4} to 10^{-2} inch/inch.

C.4.2.1 Sample Preparation and Handling: These tests were performed on undisturbed cylindrical samples obtained from rotary borings using a sampler lined with either brass rings or Shelby tubes. Samples from the brass rings were 2.42 inches in diameter by 5 inches in length; those from the Shelby tubes were 2.87 inches in diameter by 6 inches in length. The samples were extruded, weighed and placed in the test cell.

C.4.2.2 Test Conditions and Parameters: Test conditions and parameters may vary in the dynamic triaxial test. The procedures followed for this project were:

- ° Stress controlled: After specimen preparation, the specimens were loaded cyclically at several levels of cyclic stress. Generally, one or two cycles of a relatively low stress were applied, the specimen was reconsolidated and loaded again for one or two additional cycles at a slightly higher stress level. This procedure was repeated until the resulting strain levels became large enough to cause significant permanent strain, precluding further satisfactory data (strain of about 10^{-2} inch/inch or until the maximum cycle stress level possible with the procedure was reached, corresponding to $\sigma_{cyclic}/2\sigma_{3c}$ = 0.5.

- ° Saturation: The specimens were artificially saturated using flushing and back pressure techniques. Typical back pressures of 60 to 100 psi were required to saturate the specimens. The degree of saturation was measured using Skempton's B parameter, $\Delta u/\Delta\sigma_{3c}$. A minimum value of B = 0.95 was obtained for all test specimens which were saturated.

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*ASTM Special Technical Publication 479.

- A few of the test specimens were tested in their in situ moisture condition, without artificial saturation, in order to evaluate the stress-strain properties of unsaturated samples. The tests which were not saturated are identified on the figures.

- Consolidation: Specimens were allowed to consolidate under the specified static ambient stress levels. Consolidation was monitored either by measuring specimen volume changes or by closing the drainage lines and verifying that buildup of pore pressures did not occur. A consolidation ratio ($K_c = \sigma_{1c}/\sigma_{3c}$) of 1.0 was used for this program.

- Waveform and Frequency: A sinusoidal waveform at a frequency of 0.5Hz was used for this test program.

C.4.2.3 Apparatus: The apparatus described below was used for this test. In addition, for the dynamic triaxial tests, an x-y flatbed recorder was utilized to record the hysteretic stress stain curve for each load cycle.

The pneumatic loading system used for these tests was custom-designed and built for Converse Consultants. The device consists of the four main component groups described below.

- Triaxial Chambers and Cyclic Loading Device: The triaxial chambers are comprised of stainless steel and aluminum cells designed for operating pressures up to 400 psi. (Pressures of up to 160 psi were used for this project.) A pneumatic, double-acting piston, capable of applying both static and cyclic loads, is mounted above the triaxial chamber and connected to the specimen load cap by a low-inertia stainless steel rod. The rod passes through the top of the chamber and is held in place by low friction bushings and pressure seals.

- Control Console: This unit contains the various pressure regulators and reservoir systems for controlling cell pressure, back pressures, and sample saturation and drainage. The controls on the console regulate the wave form, frequency, and magnitude of the static and cyclic axial loads.

- Transducer System and Signal Conditioners: The electronic transducers produce electrical voltages in proportion to the key parameters being measured during the test. Parameters monitored and transducer type employed for this program are:

PARAMETER MONITORED	TRANSDUCER TYPE
Axial displacement	- Linear variable differential transformers (LVDT's) mounted internally to the specimen load caps
Soil pore water pressure	- Unbonded wire resistance strain-gauge-type transducers mounted external to the chamber on sample drainage lines
Axial load	- Bonded resistance strain-gauge-type load cell mounted between double-acting piston and rod connected to specimen load cap

Signal conditioners such as power supplies and variable gain amplifiers are used to excite the transducers and amplify the signals to recordable levels.

- Recording Devices: These include (a) a 4-channel continuous strip chart recorder, thermal pens and heat-sensitive paper, frequency response adequate for frequencies normally employed in cyclic triaxial testing, and (b) a cathode ray oscilloscope.

C.4.2.4 Data Reduction: The following methods and definitions were employed in the reduction of test data from the dynamic triaxial tests.

- Axial stress: Given in terms of axial load and the unconsolidated specimen crosssectional area.
- Axial strain: Given in terms of the consolidated specimen length.
- Dynamic axial strain: The peak-to-peak axial strain for any given loading cycle.
- Shear modulus and shear strain conversion: Axial stress, axial strain and Young's modulus, E, were converted to equivalent shear stress, shear strain and shear modulus, G, using a Poisson's ratio of 0.5 (undrained, zero volume change condition) for tests on saturated samples, and an assumed Poisson's ratio of 0.40 for tests on saturated specimens tested at their in situ moisture contents. Shear strain values are the strains on a plane located at 45° to the principal stress plane, which has been shown to be the plane of maximum shear strain during triaxial loading.
- Modulus: Shear modulus values are defined as the equivalent linear modulus corresponding to the straight line connecting the end points of the hysteresis loop of each loading cycle.
- Shear strain: Shear strain values given are the maximum shear strains between the end points of the hysteresis loop for a given cycle. The maximum shear strain is calculated according to the equations of solid body mechanics as 1.5 x the maximum axial strain.

The Dynamic Triaxial test results are shown on Figures C-35 and C-36.

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
28-1	1	2.0	Silty Sand	A	102	11							2.68	38.9				
	2	9.0	Sand	A	103	7					36.5	0.21						
	3	14.5	Silty Sand	A	100	17							2.70	40.9				
	4	24.5	Sand	A	107	12					29.4	0.14						
	5	29.0	-Disturbed-	-	---	--												
	6	34.5	Sand	A	106	13												
	7	39.0	Sand	A	110	10			1.9×10^{-3}		35.0	0.20	2.68	34.3				
	8	44.5	Sand	A	113	11												
	9	49.0	Clayey Silt	A	104	23					15.0	0.61						
	10	59.0	Gravely Sand	A	123	10			1.8×10^{-3}									
	11	64.5	Clayey Silt	A	111	16												
	12	71.0	Sandy Siltstone	C	110	20			1.2×10^{-5}									
	13	78.5	Sandy Siltstone	C	122	16				1.42			2.71	27.9				
	14	86.5	Sandy Siltstone	C	112	17									X	X		X(3)
	15	93.5	Sandy Siltstone	C	122	15											X	
	16	99.5	Sandy Siltstone	C	115	17				3.06					X(2)	X(2)		
28-2	1	3.0	Sand	A	109	6												
	2	7.0	Sand	A	105	13					33.0	0.35					X	

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS'S (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ø, deg	c, ksf						
28-2	3	13.0	Sand	A	113	14					32.0	0.45						
	4	18.0	Silty Sand	A	111	12												
	5	23.0	Clayey Sand	A	103	18	25	6							X	X		X(2)
	6	28.0	Clayey Sand	A	100	16				1.5								
	7	33.0	Clayey Sand	A	97	24												
	8	38.0	Clayey Sand	A	110	16												
	9	43.0	Sand	A	122	14												
	10	48.0	Clayey Sand	A	106	17				0.7								
	11	53.0	Silty Clay	A	97	27	36	15									X	
	12	58.0	Sand	A	99	17									X			
	13	63.0	Sand	A	120	14												
	14	68.0	Sand	A	111	13			$4.4x10^{-3}$		29.0	0.45						
	15	75.0	Sandy Clay	A	121	14					36.0	0.75						
	16	87.0	Clayey Sand	A	120	14												
28-3	1	1.0	Silty Sand	A	102	11												
	2	6.0	-Disturbed-		-	-												
	3	11.0	Clayey Sand	A	113	12												
	4	16.0	Silty Sand (Disturbed)	A	-	-	NP											

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ø, deg	c, ksf						
28-3	5	21.0	Silty Sand	A	113	14					32.0	0.30						
	6	26.0	Clayey Sand	A	117	10												
	7	31.0	Silty Sand	A	103	16												
	8	36.0	-Disturbed-		-	-												
	9	41.0	Clayey Sand	A	89	14												
	10	46.0	Clayey Sand	A	110	13												
	11	51.0	Sandy Clay	A	106	20	30	12								X		X(2)
	12	56.0	Clayey Sand	A	95	18												
	13	61.0	Clayey Sand	A	113	17												
	14	66.0	Sand	A	111	16			7.0×10^{-4}									
	15	71.0	Sand	A	126	11												
	16	76.0	Sand	A	109	16									X			X(3)
	17	31.0	Sand	A	117	14												
28-4	1	2.0	Sand	A	113	15												
	2	7.0	Sand	A	108	8												
	3	12.0	Sandy Clay	A	101	23												
	4	17.0	Clayey Sand	A	108	18	29	13			25.0	0.50						
	5	22.0	Silty Sand	A	111	13												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
28-4	6	27.0	Clayey Sand	A	109	15												
	7	32.0	Clayey Sand	A	112	14				1.0								
	8	37.0	Silty Sand	A	104	16	NP				31.0	0.25				X		
	9	42.0	Sand	A	104	18												
	10	47.0	Sand	A	105	18			3.1x10^{-3}						X			
	11	52.0	Silty Sand	A	118	13												
	12	58.0	Sandy Silt	A	110	17	27	5									X	
	13	63.0	Silty Sand	A	111	18											X	
	14	67.0	Gravely Sand	A	112	13					30.0	0.35			X			
	15	72.0	Gravely Sand	A	117	15												
	16	77.0	Sand	A	126	9												
28-5	1	3.0	Silty Sand	A	111	11											X	
	2	10.0	Clayey Sand	A	108	12												
	3	15.0	Sandy Clay	A	104	18				1.2								
	4	20.0	Clayey Sand	A	112	16	NP									X		
	5	25.0	Clayey Sand	A	111	10					24.0	0.75						
	6	30.0	Sand	A	108	16												
	7	35.0	Sand	A	112	15												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPELIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ø, deg	c, ksf						
28-5	8	40.0	Clayey Sand	A	107	17												
	9	45.0	Silty Sand	A	90	14												
	10	50.0	Silty Sand	A	99	18					26.0	0.30			X			
	11	57.0	Clayey Sand	A	107	12												
	12	63.0	Sand	A	116	11											X	
	13	65.0	-Disturbed-		-	-												
	14	70.0	Sand (Disturbed)	A	76	14												
	15	75.0	Silty Sand	A	116	14					29.0	0.40			X			
	16	80.0	Silty Sand	A	117	14												
	17	85.0	Gravely Sand	A	131	9												
28-6	1	3.0	Sandy Clay	A	103	16											X	
	2	8.0	Sandy Clay	A	105	16					15.0	0.30						
	3	13.0	-Disturbed-		-	-												
	4	18.0	Sand	A	111	14												
	5	24.0	Sandy Clay	A	107	18	28	8								X		X(2)
	6	28.0	Clayey Sand	A	104	17												
	7	33.0	Silty Sand	A	113	14	NP				26.0	0.80				X		
	8	38.0	Clayey Sand	A	113	13												
	9	43.0	Clayey Sand	A	107	17												

TABLE C-1 LABORATORY TEST DATA

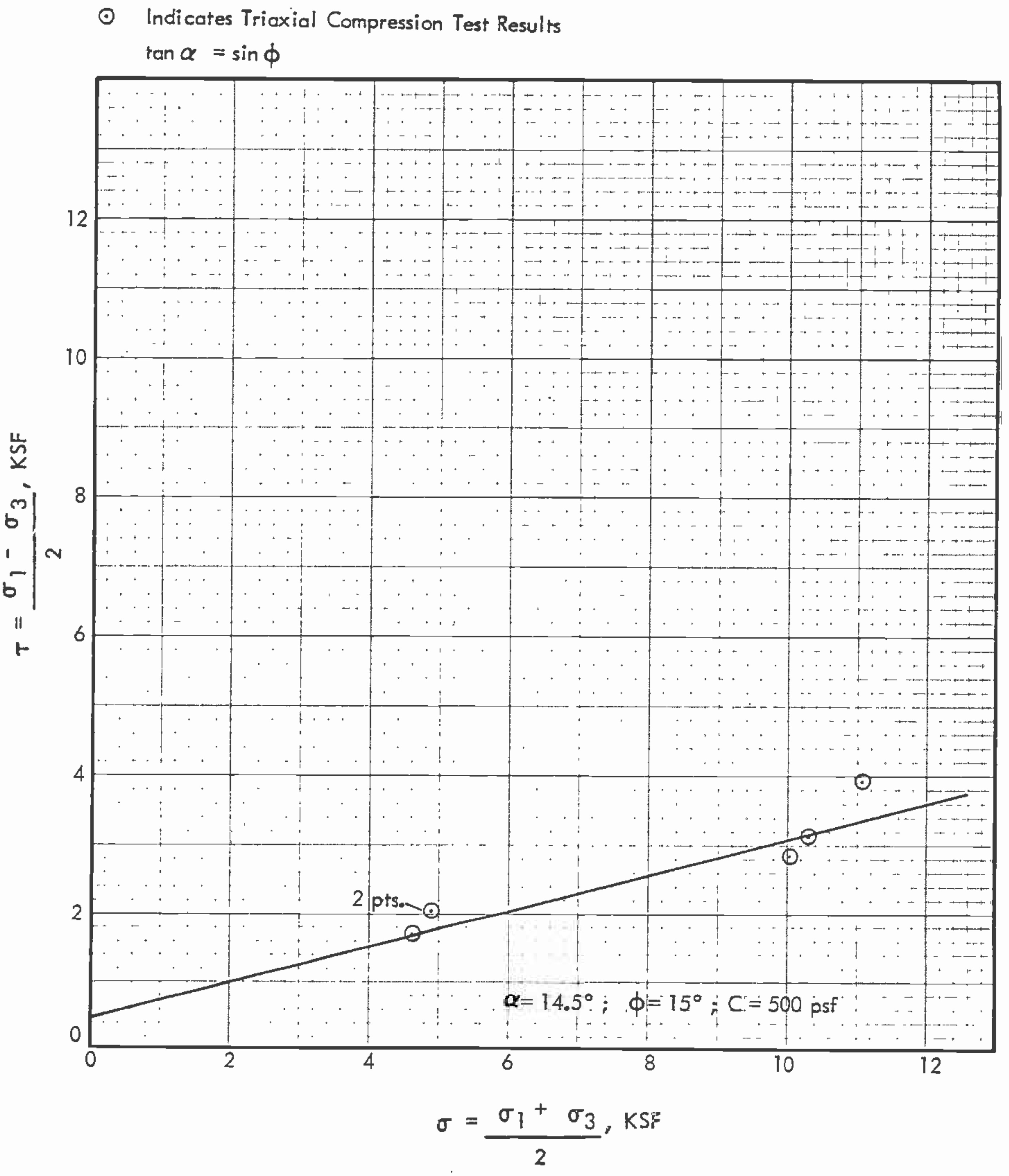
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		K_v, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
28-6	10	48.0	Clayey Sand	A	107	15												
	11	53.0	Sand	A	116	13									X			
	12	79.0	-Disturbed-		-	-												
28-7	1	4.0	Silty Sand	A	99	10											X	
	2	9.0	Silty Sand	A	112	9												
	3	14.0	Sandy Clay	A	110	14					25.0	0.45						
	4	24.0	Sandy Clay	A	114	13												
	5	29.0	Sandy Clay	A	107	19	34	17			26.0	0.40				X		
	6	34.0	Silty Sand	A	113	13												
	7	44.0	Sandy Clay	A	114	15	27	7			26.0	0.50						
	8	54.0	Silty Sand	A	122	8												
	9	64.0	Gravely Sand	A	120	11												
	10	69.0	Silty Sand	A	113	5												
	11	74.0	Silty Sand	A	118	13									X			X(3)
	12	84.0	Silty Sand	A	120	12												
	13	89.0	Silty Clay	A	105	22				5.4								
	14	94.0	Silty Sand	A	110	17												
	15	99.0	Silty Sand	A	118	17												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		SPECIFIC GRAVITY	POROSITY (n)	SIEVE ANALYSIS (Quantity)	HYDROMETER ANALYSIS (Quantity)	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ø, deg	c, ksf						
28-8	1	2.0	Silty Sand	A	118	13												
	2	10.5	Sand	A	113	7					32.8	0.10						
	3	16.0	Silty Sand	A	107	14					28.8	0.25						
	4	22.5	Sand	A	107	12							2.68	35.8				
	5	27.0	Gravely Sand	A	128	6												
	6	32.5	Silty Sand	A	102	15												
	7	37.0	Sand	A	107	13					28.8	0.12						
	8	47.0	Sandy Gravel	A	114	12			$2.2x10^{-4}$				2.74	33.2				
	9	52.5	Clayey Silt	A	101	23					15.0	0.48	2.69	39.6				
	10	57.0	Siltstone	C	110	21												
	11	60.5	Siltstone	C	109	21				1.14								
	12	70.5	Sandy Siltstone	C	115	15			$1.7x10^{-3}$								X	
	13	80.5	Sandy Siltstone	C	114	18												
	14	90.5	Sandy Siltstone	C	125	11				2.53					X(2)	X(2)		
	15	99.5	-Disturbed-	-	-	-												

TABLE C-2 COMPREHENSIVE LIST OF SOILS ENGINEERING PROPERTIES FROM LABORATORY TESTS

CEG BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		PARTICLE SIZE CUMULATIVE % PASSING SIEVE No.			UNCONFINED COMPRESSION STRENGTH (psi)	Kv, COEFFICIENT OF PERMEABILITY (cm/sec	COEFFICIENT OF CONSOLIDATION C_c (in/in per Log Cycle)	SPECIFIC GRAVITY	POROSITY (n)	UNDRAINED QUICK DIRECT SHEAR		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	CYCLIC TRIAXIAL (Liquefaction)	DYNAMIC TRIAXIAL (Stress/Strain)	RESONANT COLUMN	TRIAXIAL COMPRESSION
							LL	PI	4	40	200						ϕ, deg	c, ksf					
28	C1	20	Clayey Silt	A	90	28											20.0	0.18					
	C2	40	Silty Sand	A	110	10											30.5	0.77					
	C3	60	Sandy Clay	A	103	17											28.0	1.12					
	C4	80	Clayey Silty Sand	A	107	13											37.5	0.35					
	C5	100	Sandy Clay	A	101	20											14.5	0.94					
	C6	120	Clayey Sand	A	125	13											34.0	2.58					
	C7	139	Sandy Clay	A	122	13																X	
	C7	140	Sandy Clay	A	120	13							2.6E-7		2.69	28.3							
	C8	160	Silty Sand	A	120	13							8.6E-5			28.3							
	C9	180	Clayey Sand	A	123	12															X		



SUMMARY OF TOTAL STRENGTH DATA – FINE-GRAINED ALLUVIUM

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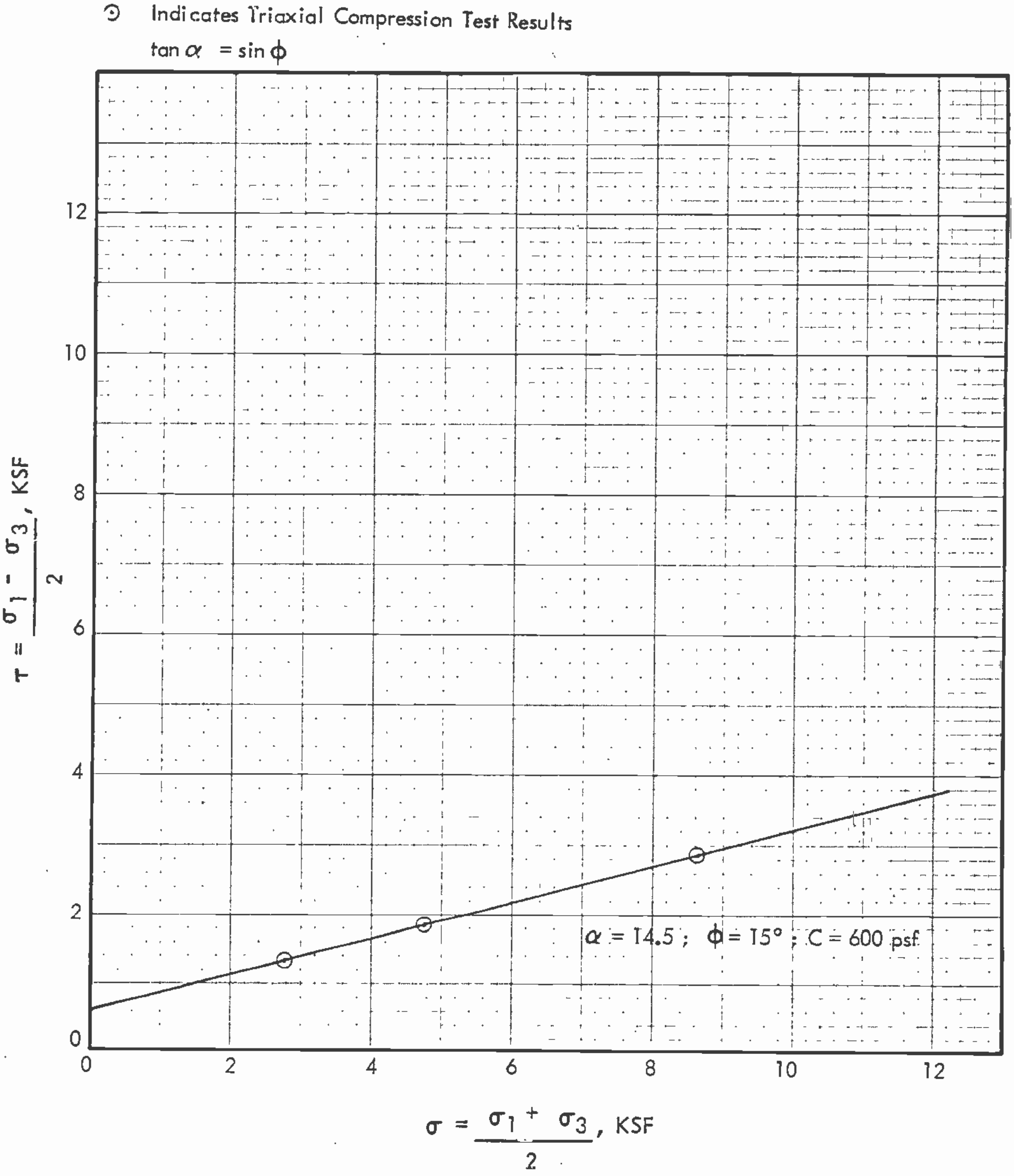
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-1

Approved for publication by 5/84



SUMMARY OF TOTAL STRENGTH DATA - BEDROCK (ALONG BEDDING)

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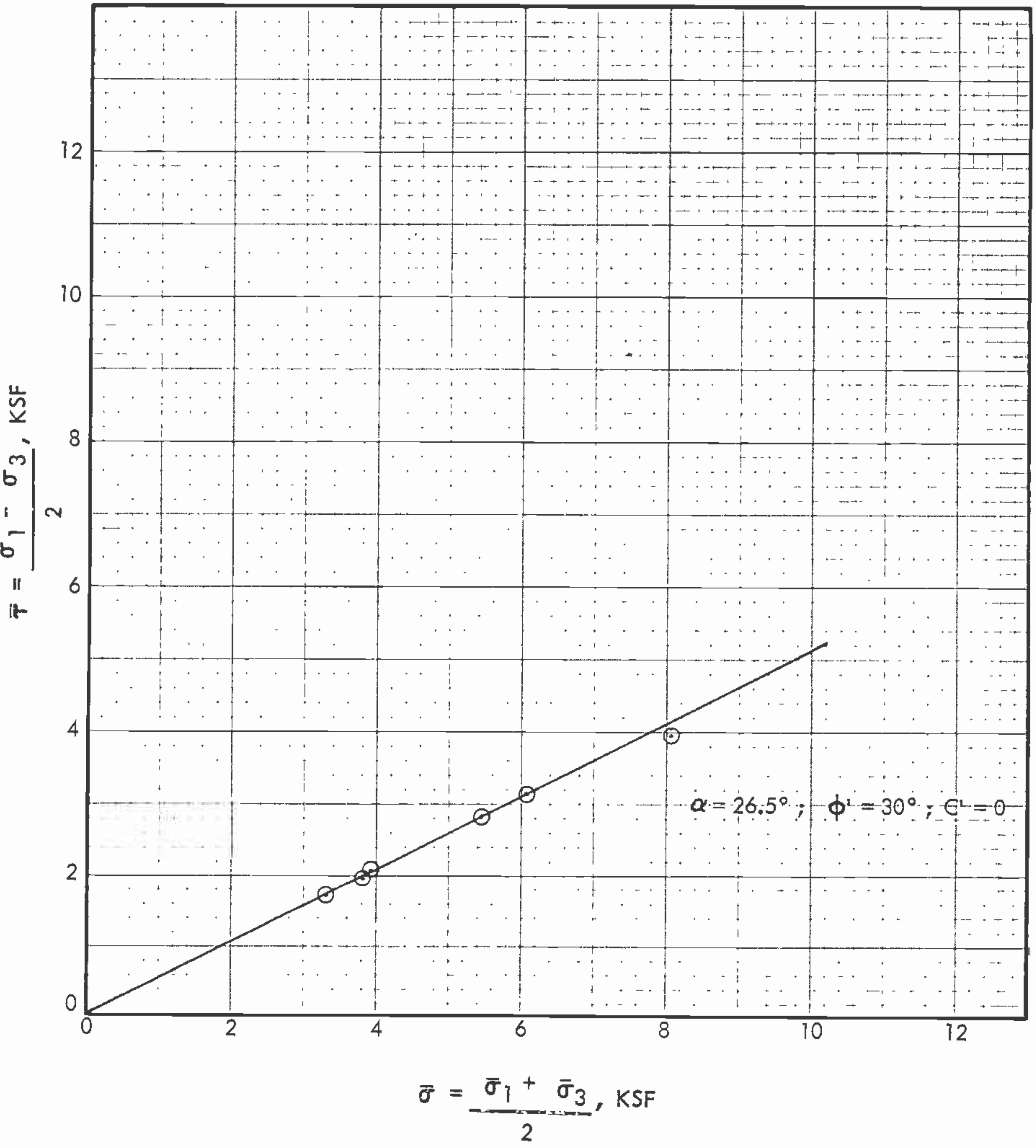
Geotechnical Engineering
and Applied Sciences

Figure No.
C-2

Approved for publication by 5/3/84

⊙ Indicates Triaxial Compression Test Results

$\tan \alpha = \sin \phi'$



SUMMARY OF EFFECTIVE STRENGTH DATA - FINE-GRAINED ALLUVIUM

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Figure No
C-3

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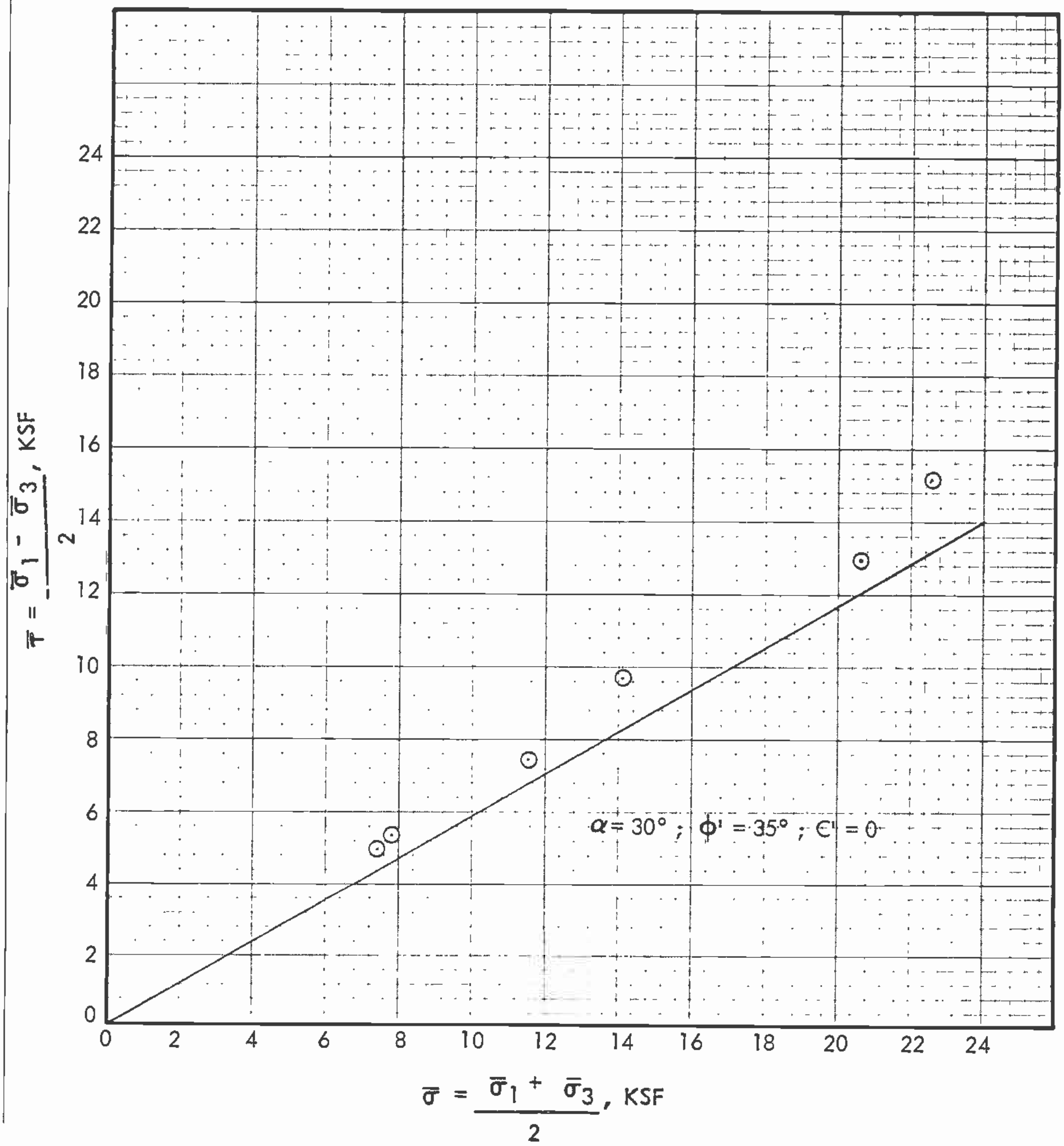
⊙ Indicates Triaxial Compression Test Results

tan α = sin ϕ'



SUMMARY OF EFFECTIVE STRENGTH DATA - GRANULAR ALLUVIUM

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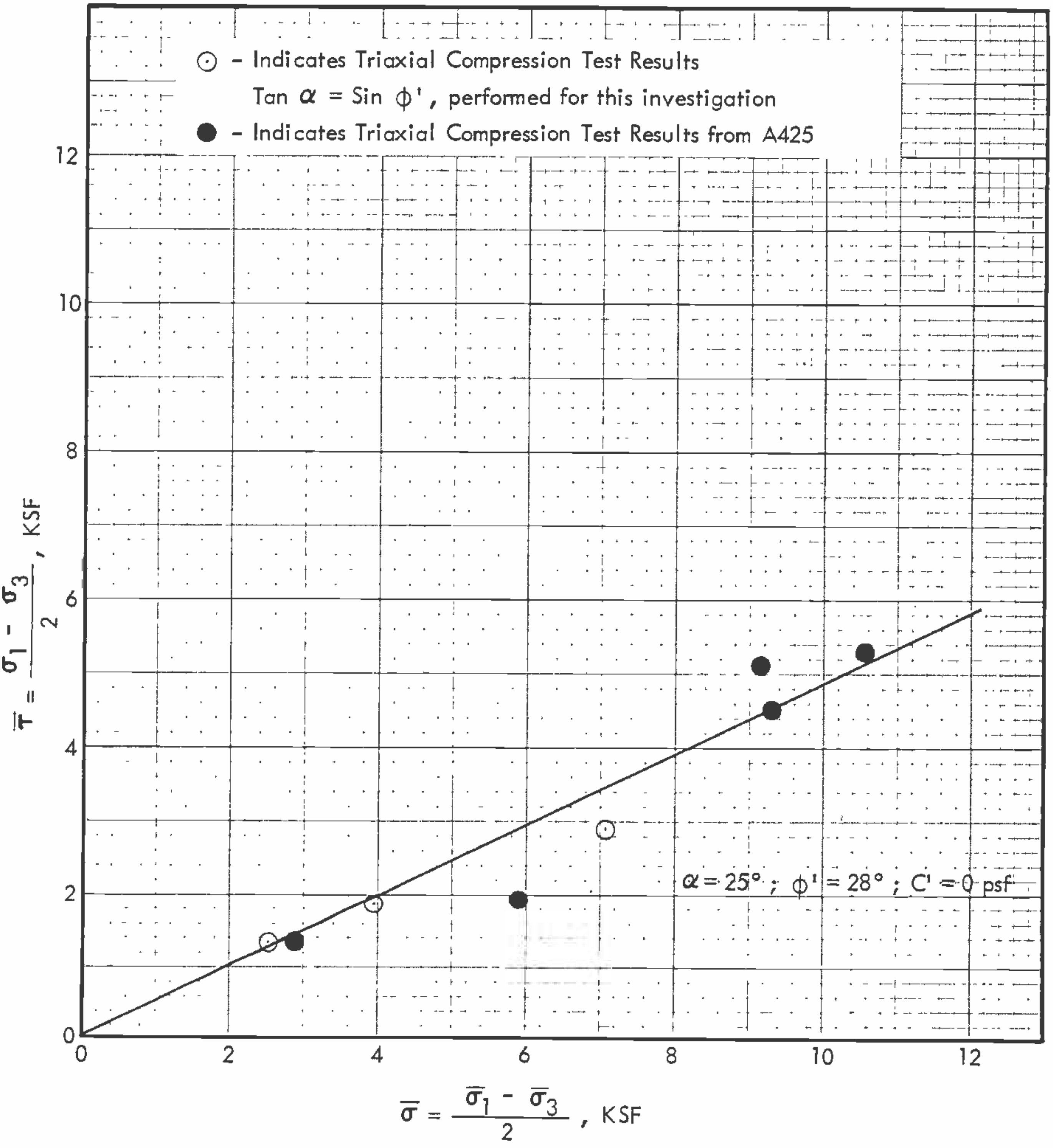
Figure No.
C-4

Approved for publication FL by 5/84



SUMMARY OF EFFECTIVE STRENGTH DATA - BEDROCK (ALONG BEDDING)

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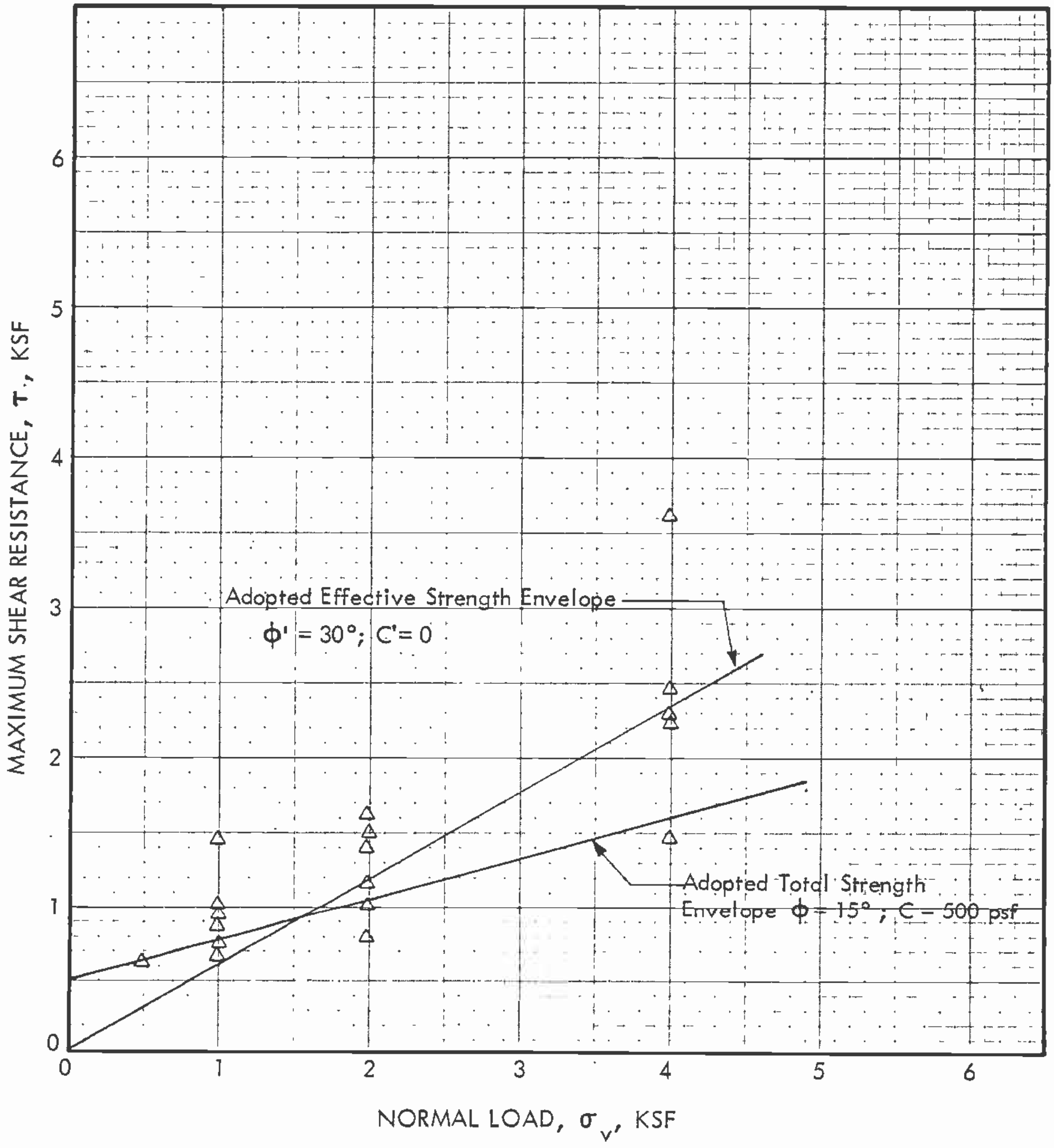
Figure No.
C-5

Approved for publication RC by 5/84



Approved for publication Fi by 5/29

SUMMARY OF DIRECT SHEAR DATA - FINE-GRAINED ALLUVIUM

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83-1140

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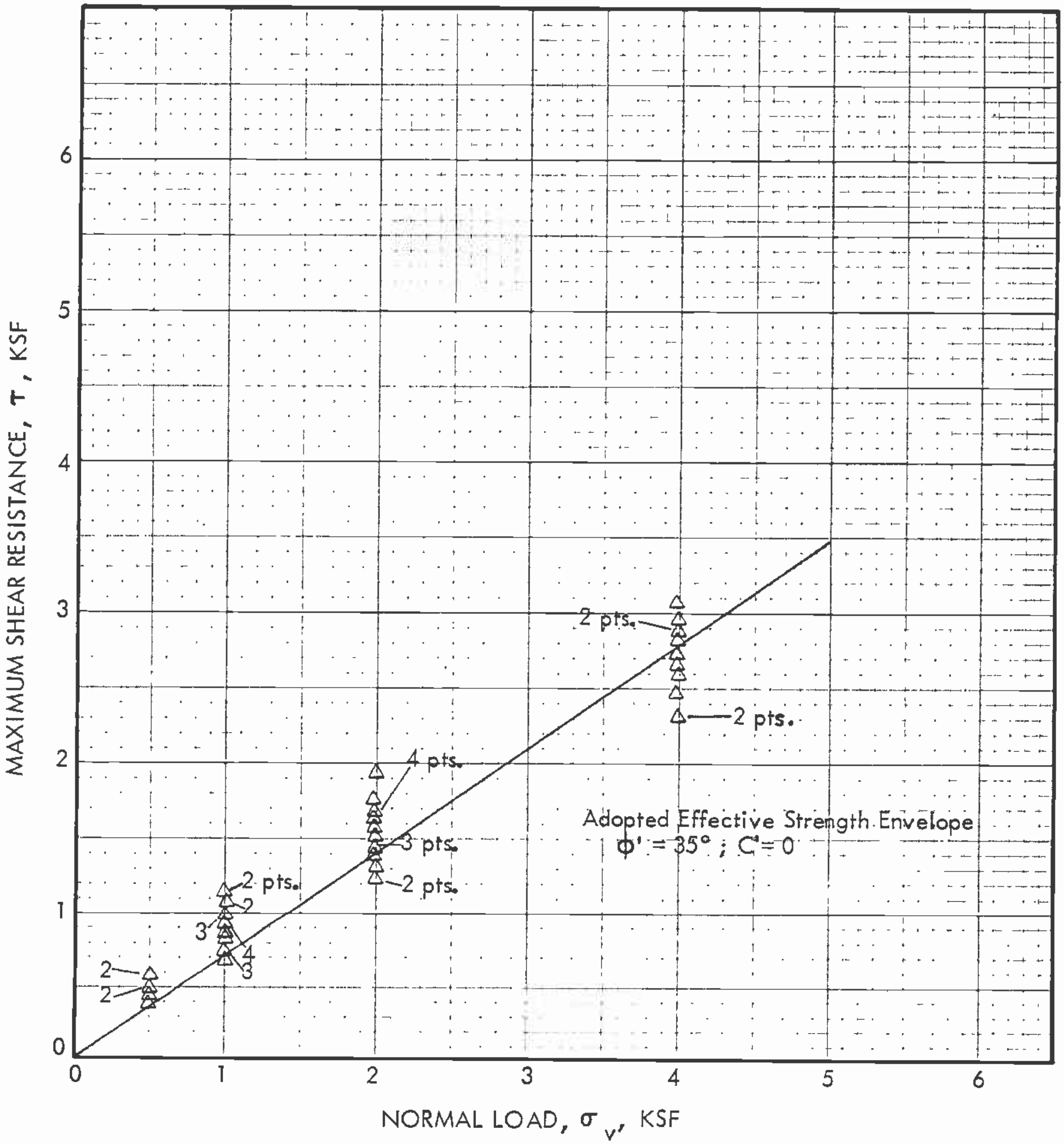
Figure No.
C-6



SUMMARY OF DIRECT SHEAR DATA - GRANULAR ALLUVIUM

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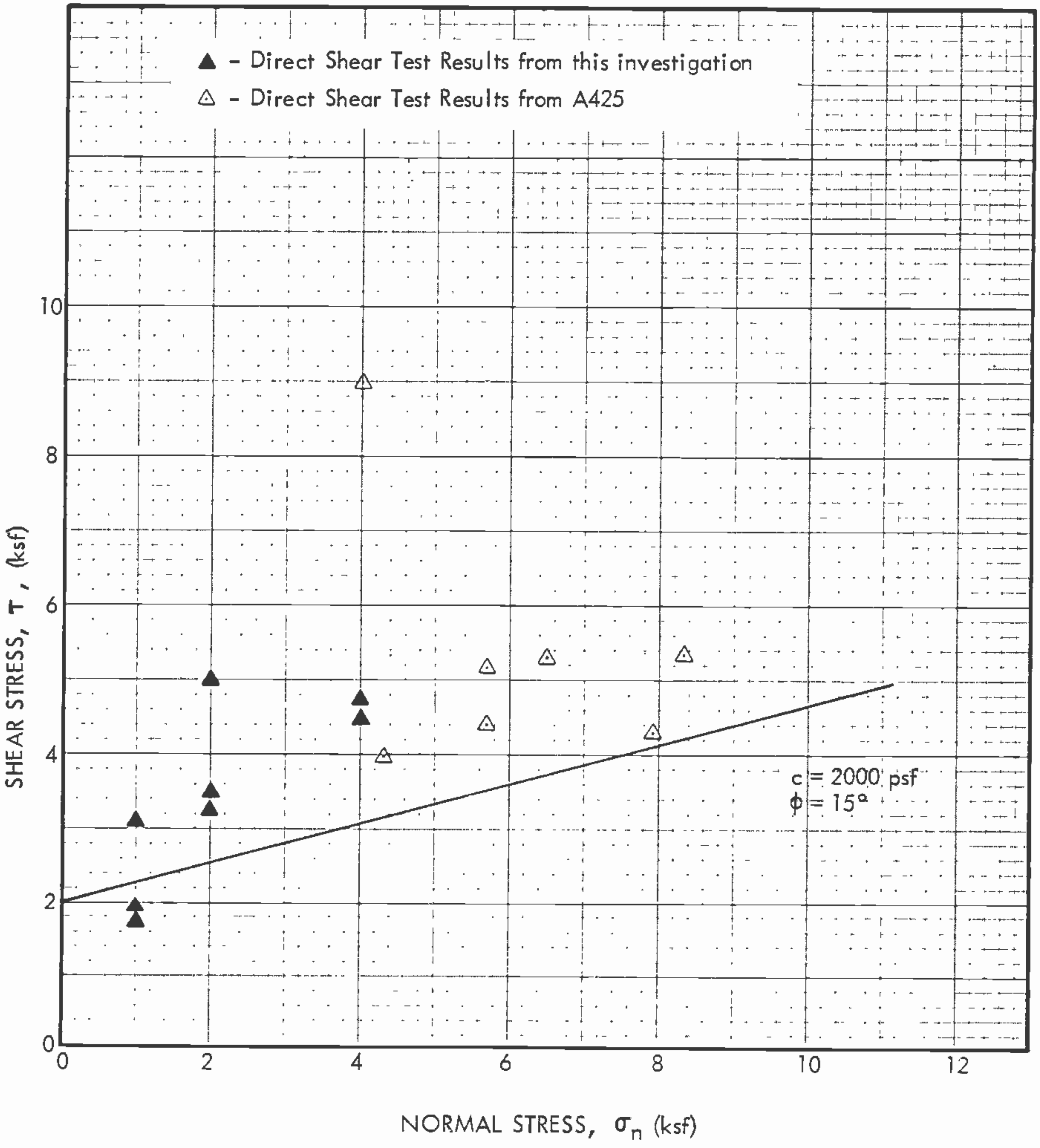
Figure No.
C-7

Approved for publication 5/7/84 by



SUMMARY OF DIRECT SHEAR TEST RESULTS - TOPANGA BEDROCK FORMATION

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METRO RAIL PROJECT

Project No.
83-1140

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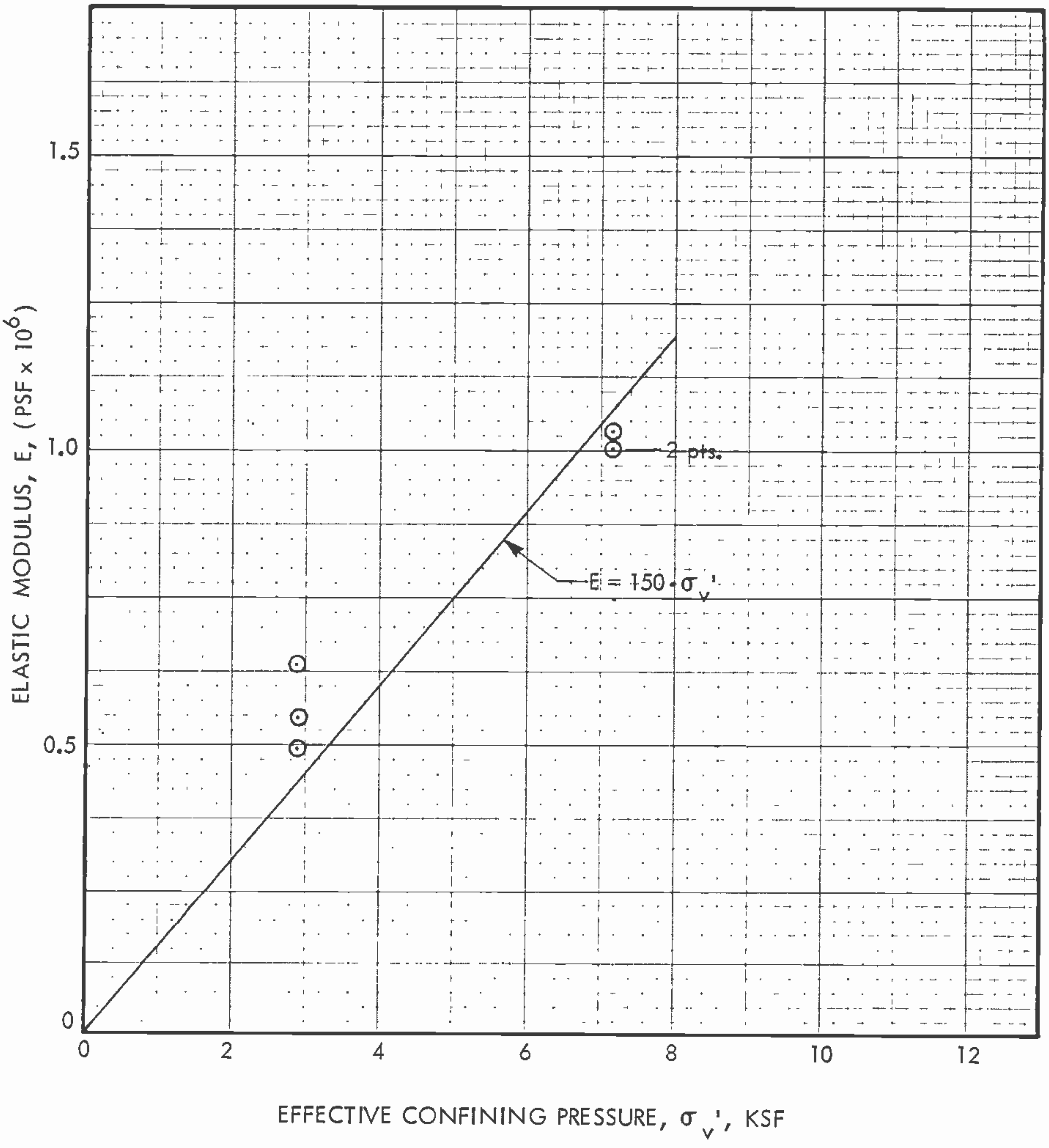
Figure No.
C-8

Approved for publication 5/89 by FC

NOTE: Elastic Modulus values are based on Average Modulus over 0.5% strain.



SUMMARY OF MODULUS DATA - FINE-GRAINED ALLUVIUM

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Project No.
83-1140

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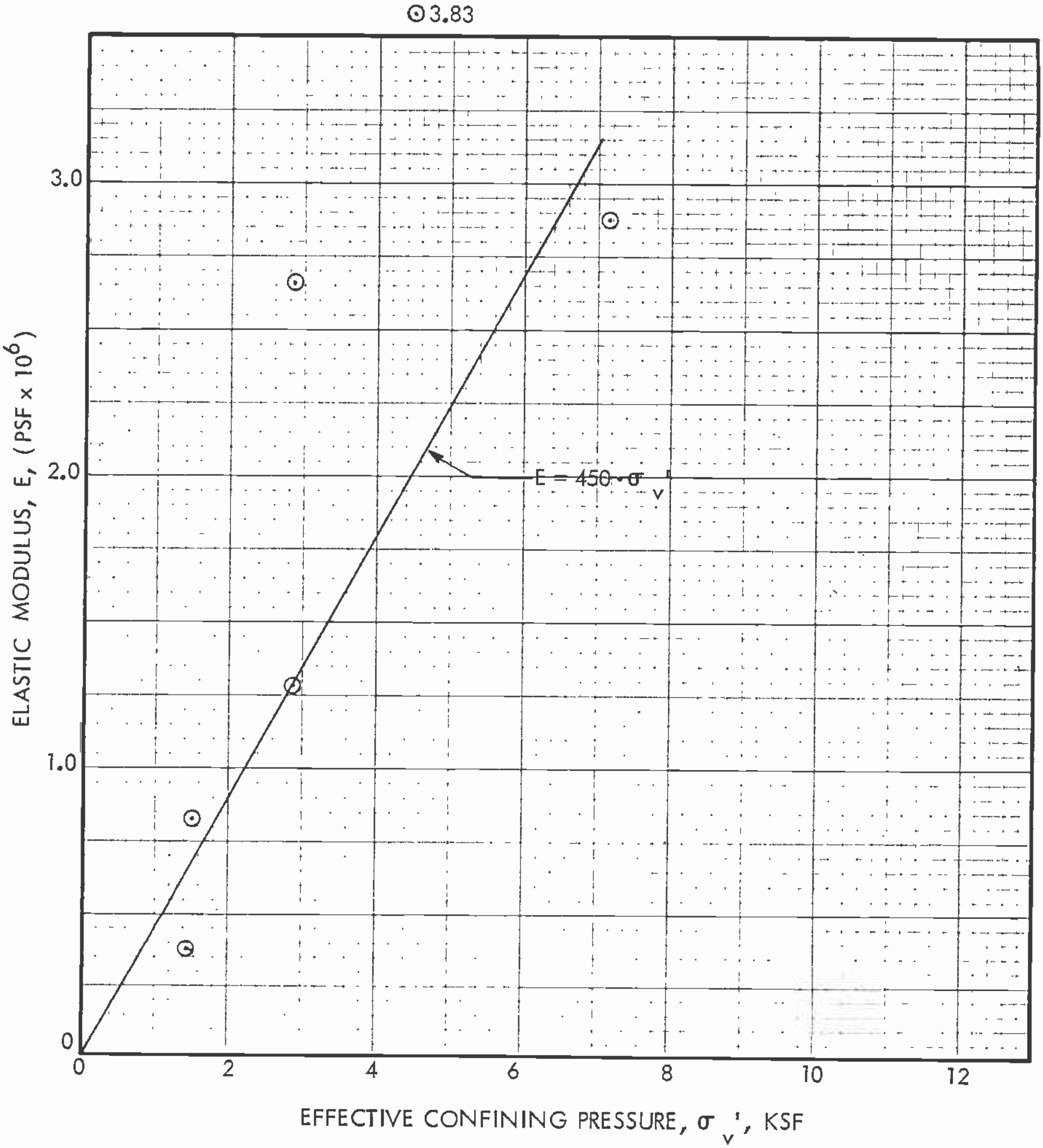
Figure No.
C-9

Approved for publication 5/8/91 by

NOTE: Elastic Modulus values are based on Average Modulus over 0.5% strain.



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SUMMARY OF MODULUS DATA - GRANULAR ALLUVIUM

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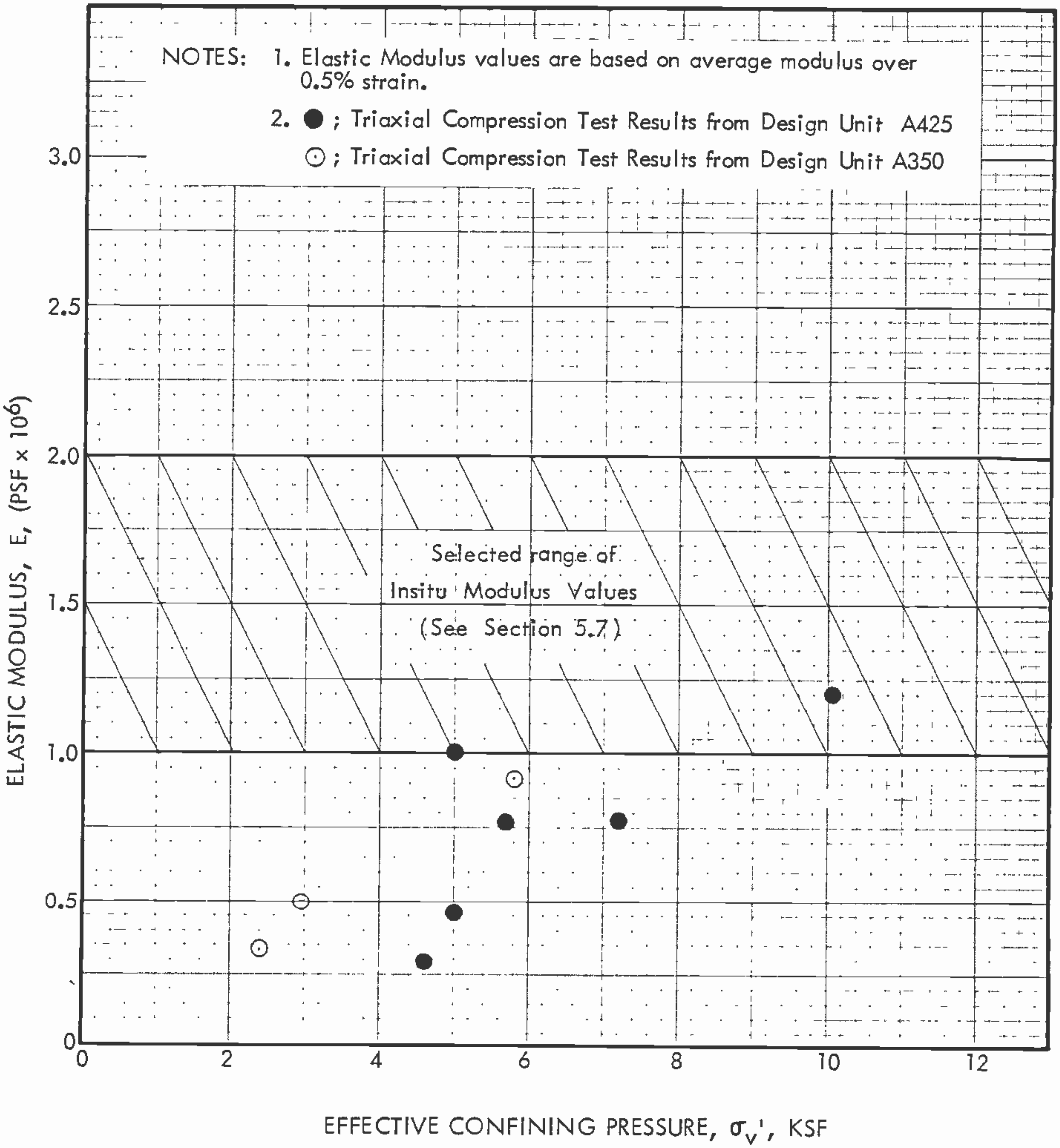
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-10



SUMMARY OF MODULUS DATA - BEDROCK

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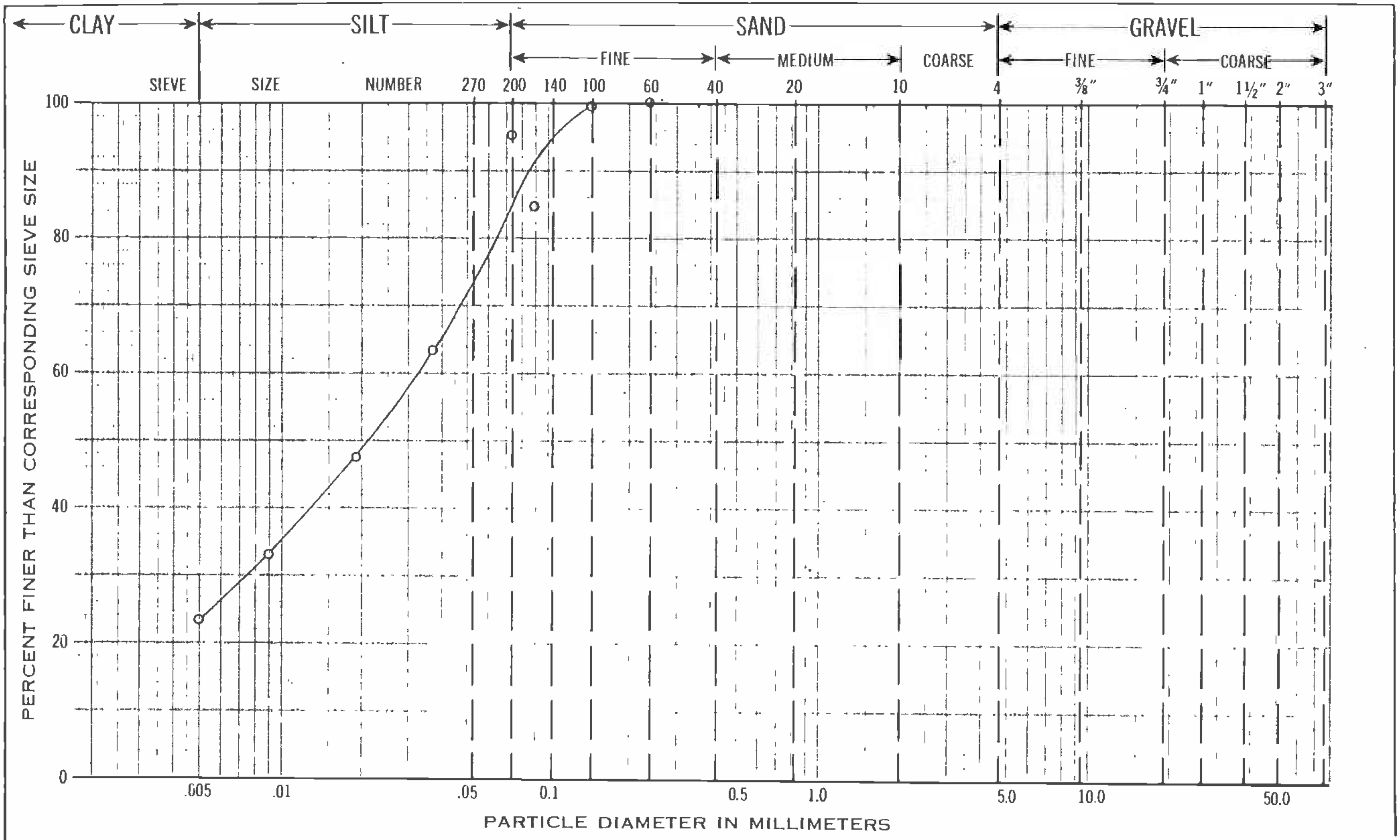
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-11

FORM NO. D-3-10 Approved for publication 5/84 by 7C



GRAIN-SIZE DISTRIBUTION CHART

BORING	SAMPLE	DEPTH
28/1	C-14	86.5'

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Project No.
83-1140

Converse Consultants
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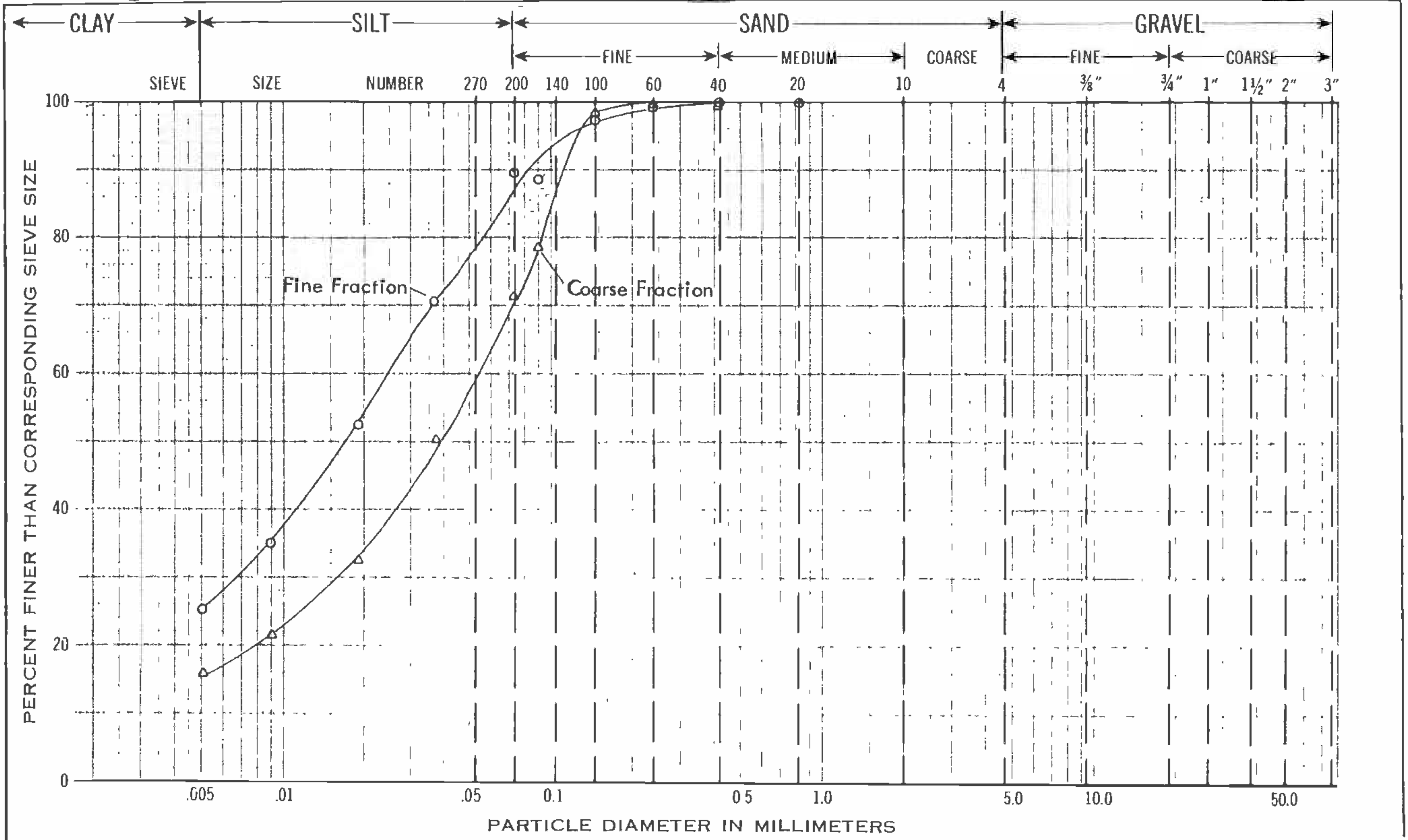
Figure No.
C-12

FORM NO. O-3-10/6 Approved for publication 5/3/4 by FC



SYMBOL	BORING	SAMPLE	DEPTH
—○—	28/1	C-16	100'
—△—	28/1	C-16	100'

GRAIN-SIZE DISTRIBUTION CHART

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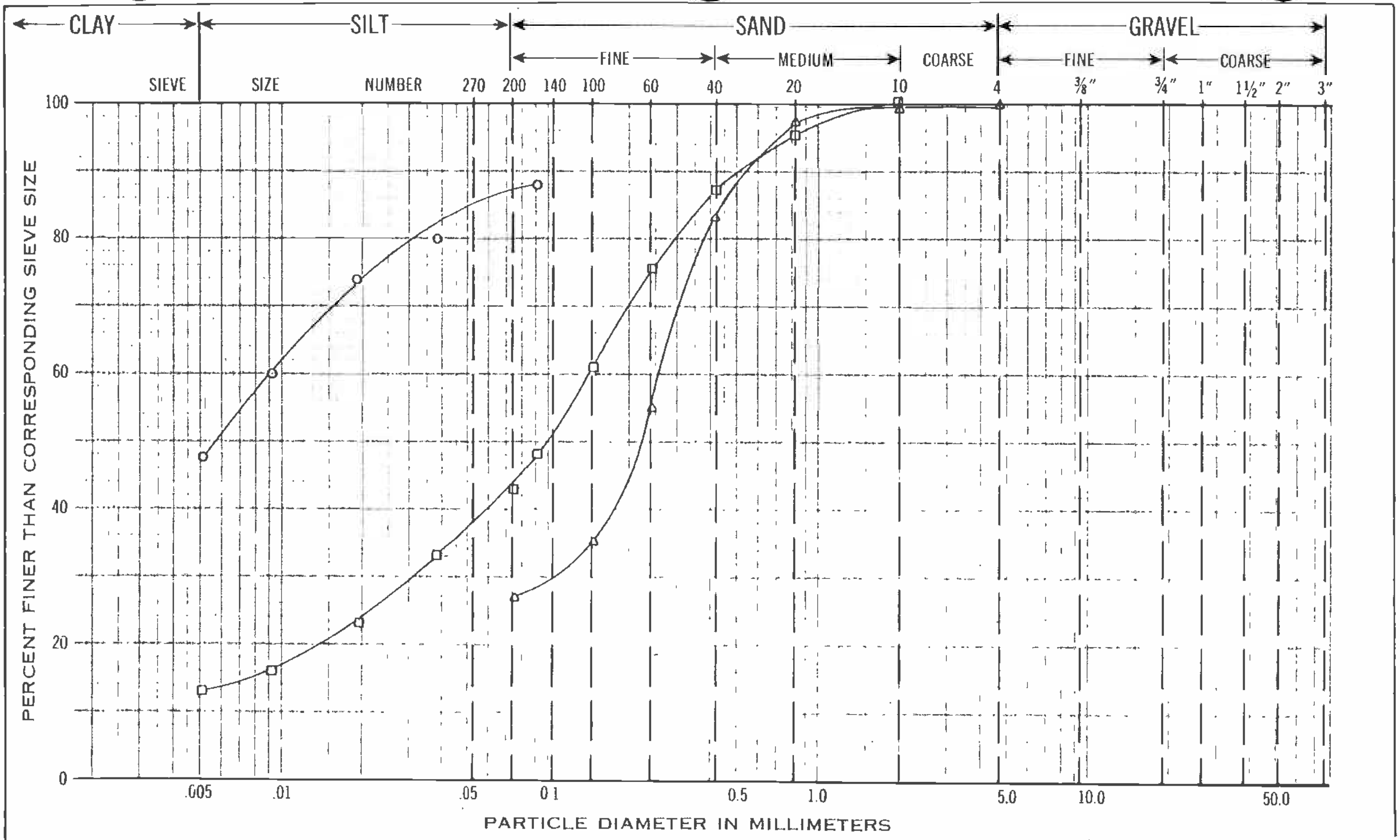
Figure No
C-13

FORM NO. D-3-10/8 Approved for publication 5/84 by FC



SYMBOL	BORING	SAMPLE	DEPTH
─□─	28/2	C-5	23'
─△─	28/2	C-12	58'
─○─	28/3	C-11	52.5'

GRAIN-SIZE DISTRIBUTION CHART

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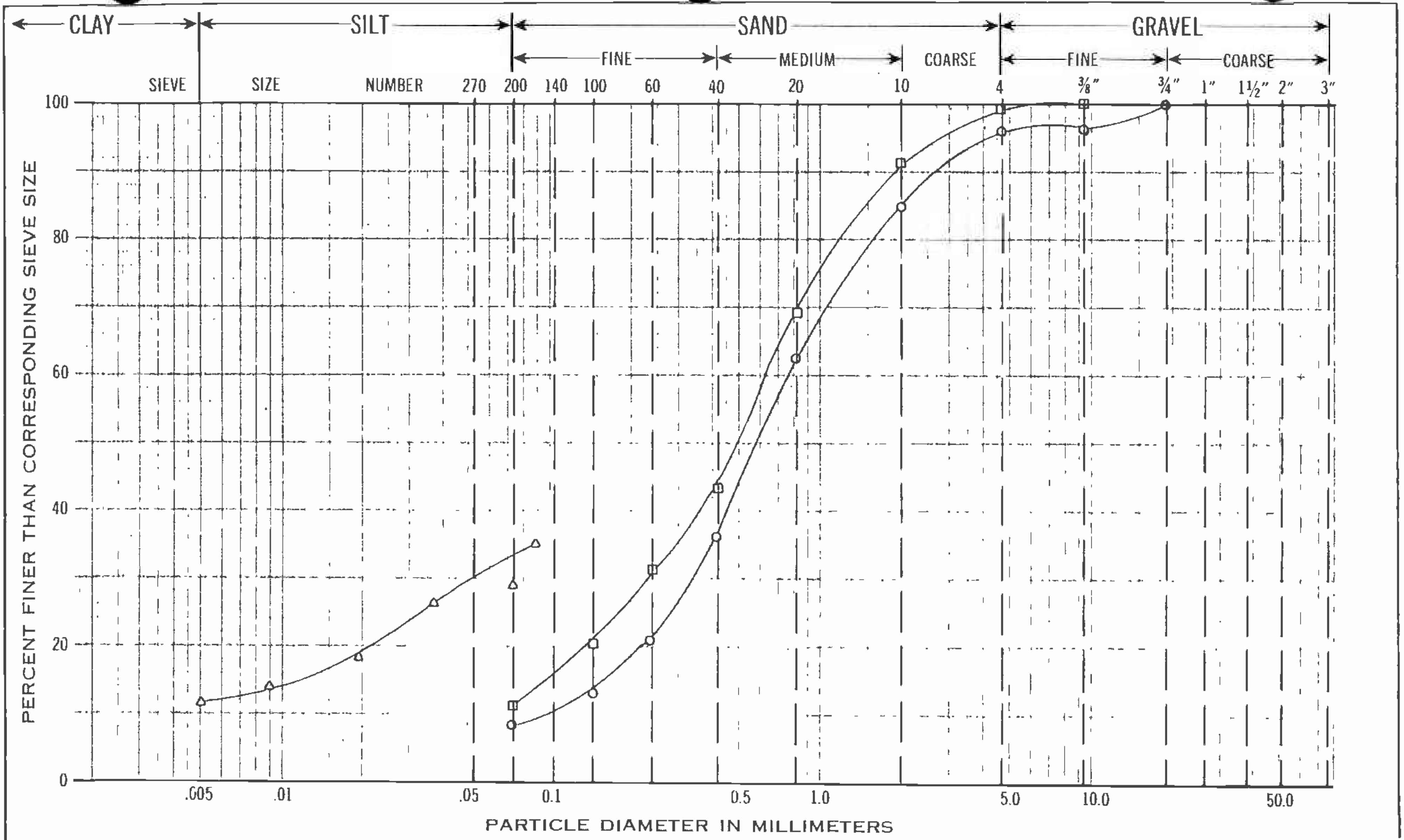
Figure No
C-14

FORM NO. D-3-10/ Approved for publication 5/84 by FC



SYMBOL	BORING	SAMPLE	DEPTH
—□—	28/3	C-16	76'
—△—	28/4	C-8	37'
—○—	28/4	C-10	47'

GRAIN-SIZE DISTRIBUTION CHART

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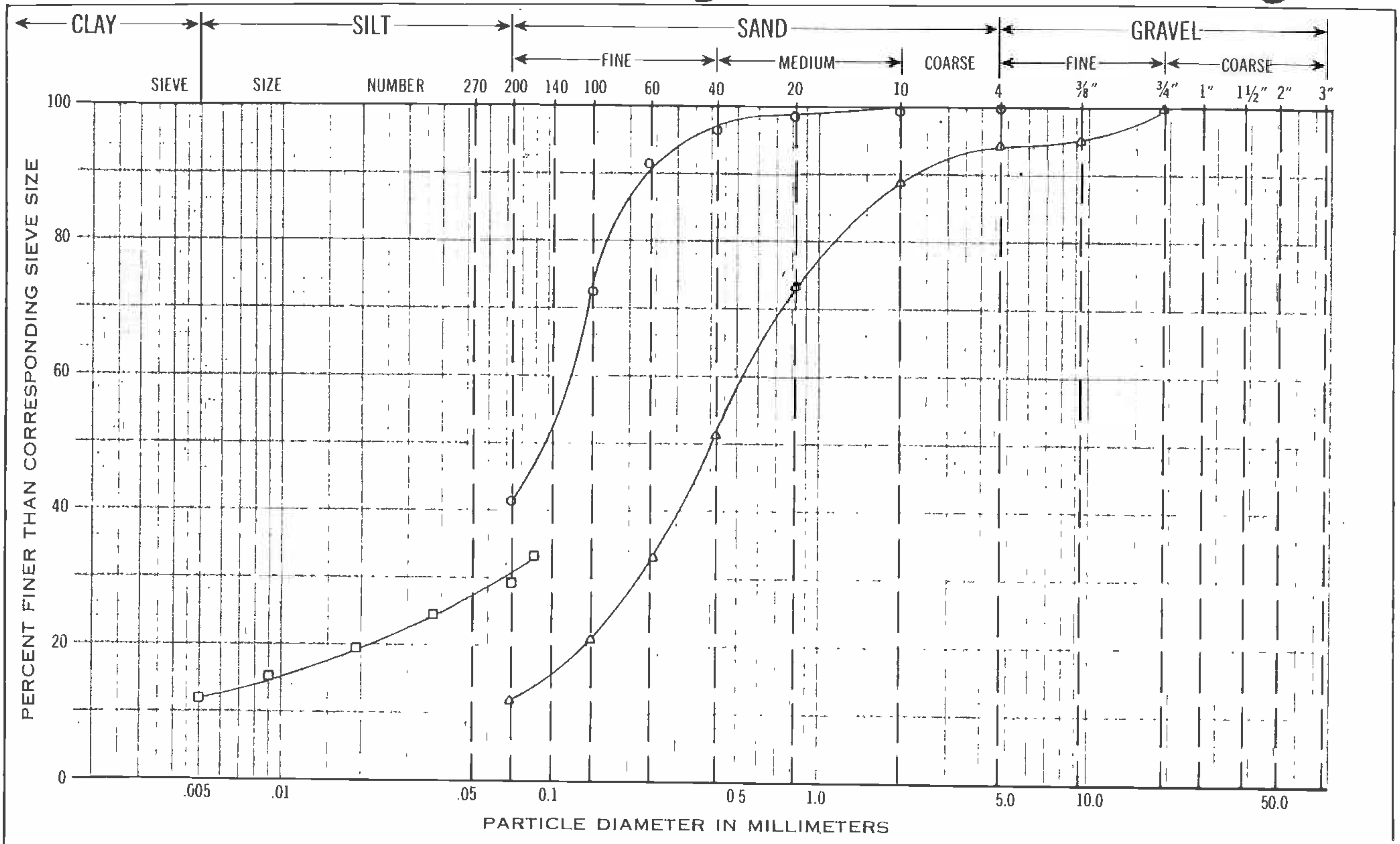
Figure No
C-15

FORM NO. D-3-10 Approved for publication 5/84 by



SYMBOL	BORING	SAMPLE	DEPTH
—△—	28/4	C-14	67'
—□—	28/5	C-4	20'
—○—	28/5	C-10	50'

GRAIN-SIZE DISTRIBUTION CHART

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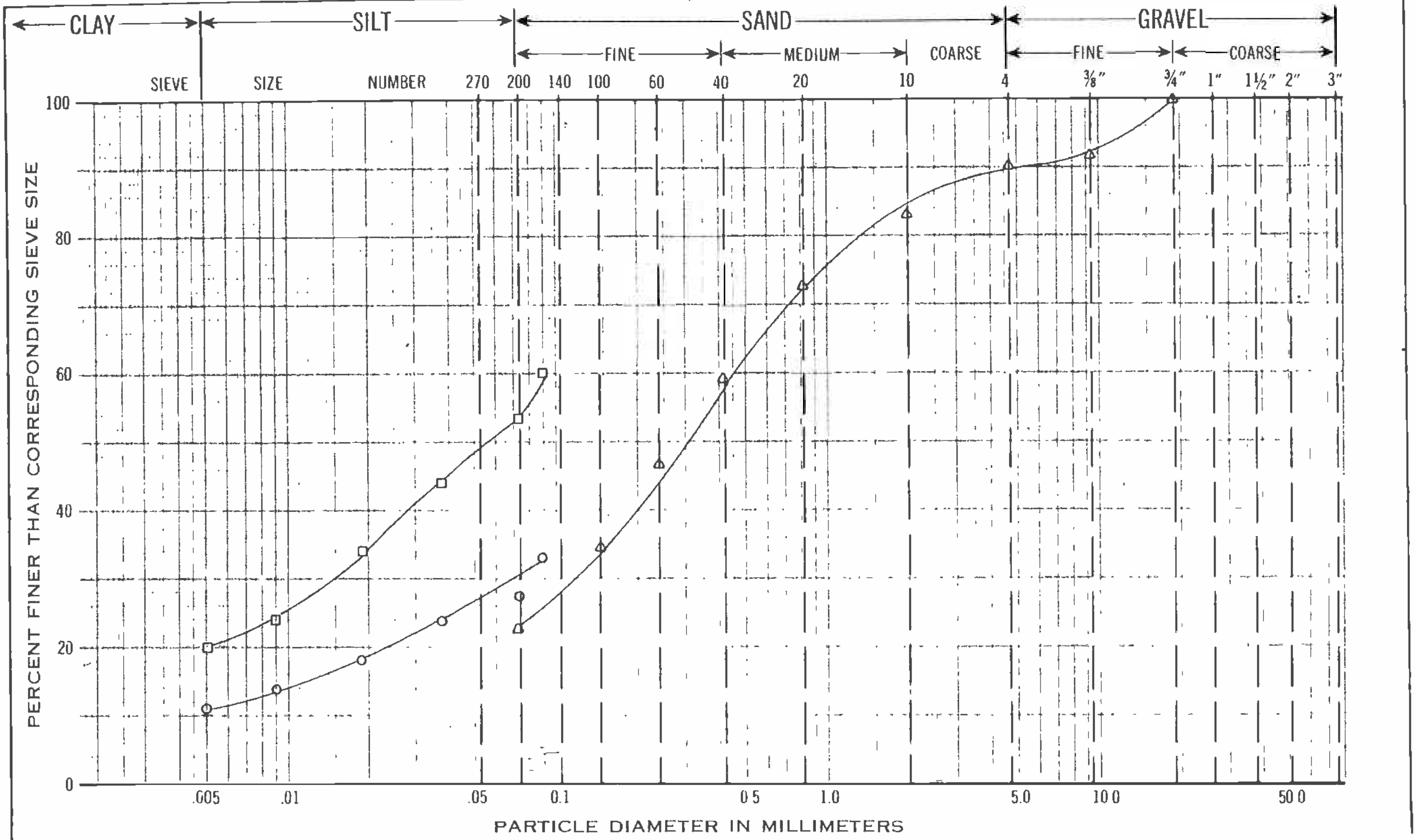
Figure No.
C-16

FORM NO. D-3-1 Approved for publication 5/89 by FL



SYMBOL	BORING	SAMPLE	DEPTH
—△—	28/5	C-15	74'
—□—	28/6	C-5	24'
—○—	28/6	C-7	33'

GRAIN-SIZE DISTRIBUTION CHART

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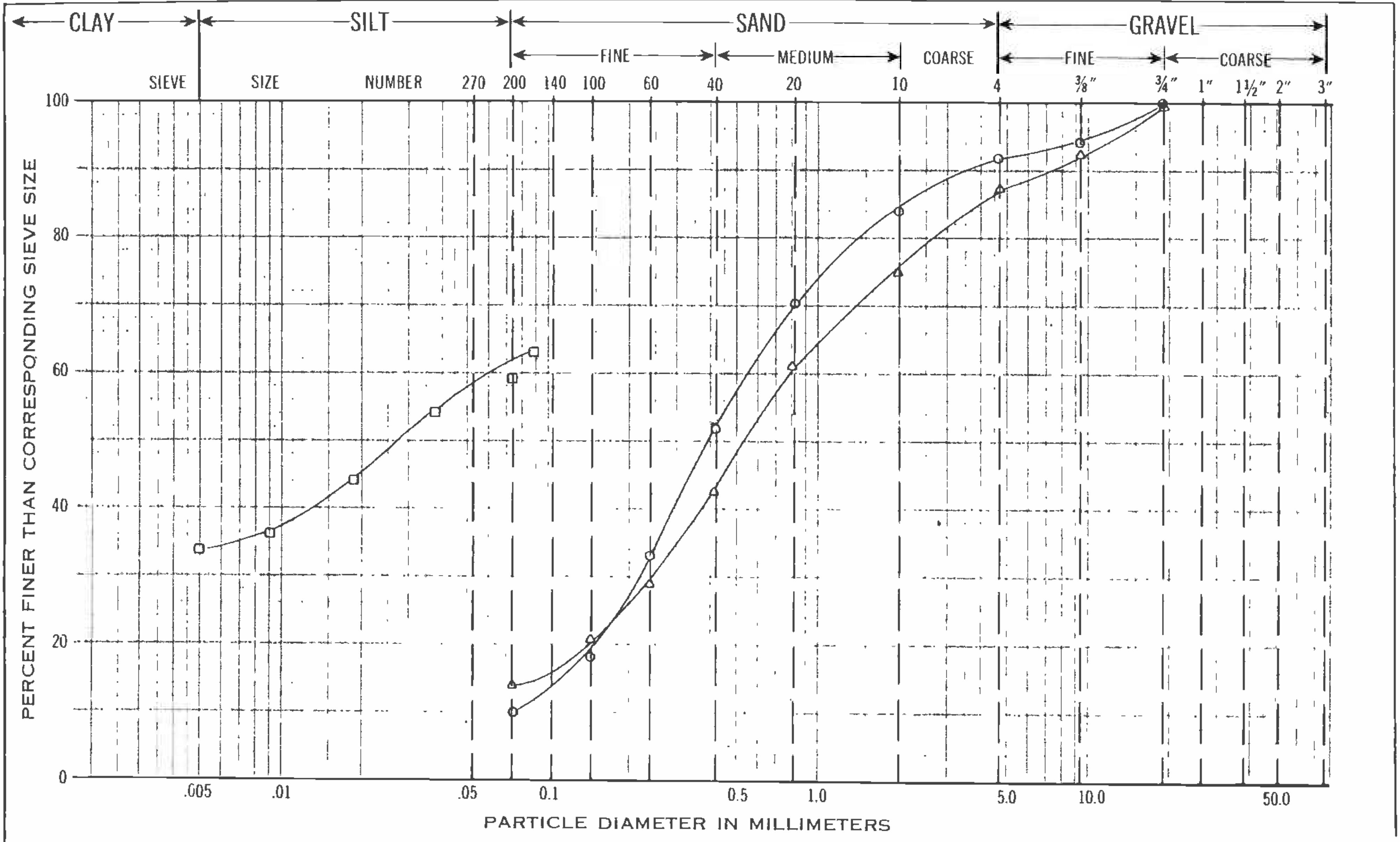
Figure No
C-17

FORM NO. D-3-1 Approved for publication 5/89 by FC



SYMBOL	BORING	SAMPLE	DEPTH
—△—	28/6	C-11	53'
—□—	28/7	C-5	29'
—○—	28/7	C-11	73.5'

GRAIN-SIZE DISTRIBUTION CHART

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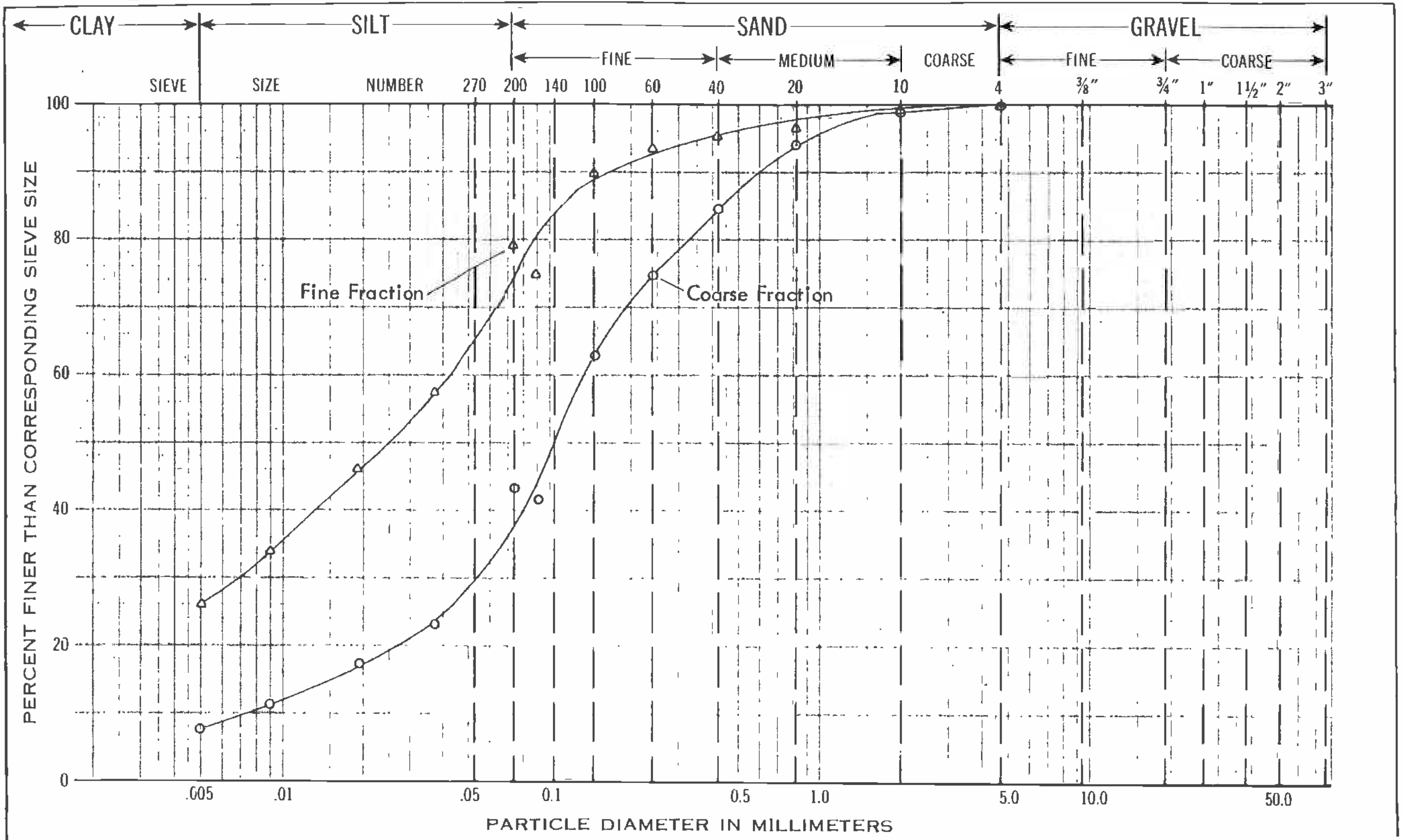
Figure No.
C-18

FORM NO. D-3-10/[illegible] pproved for publication 9/84 by [illegible]



SYMBOL	BORING	SAMPLE	DEPTH
—△—	28/8	C-14	91'
—○—	28/8	C-14	91'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No
C-19

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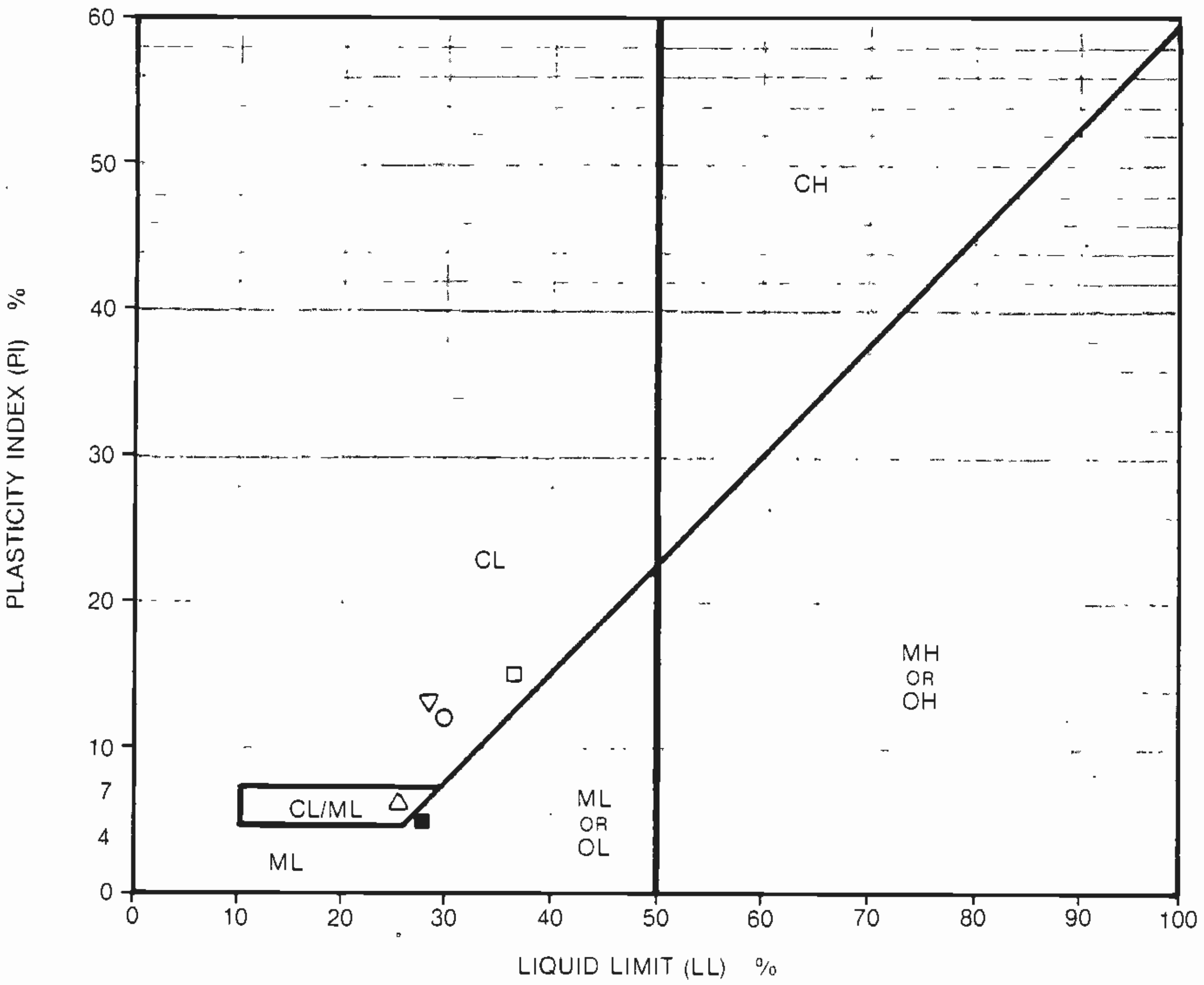

Symbol	Classification and Source				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
△	28/2	C-5	22'-23'	CL/ML	25	19	6	43%
□	28/2	C-11	52'-53'	CL	36	21	15	
	28/3	C-4	16'-17'	NP	-	-	-	
○	28/3	C-11	51'-52'	CL	30	18	12	43%
▽	28/4	C-4	16'-17'	CL	29	16	13	
–	28/4	C-8	37'-38'	NP	-	-	-	29%
■	28/4	C-12	58'-59'	ML	27	22	5	
–	28/5	C-4	19'-20'	NP	-	-	-	29%

PLASTICITY CHART

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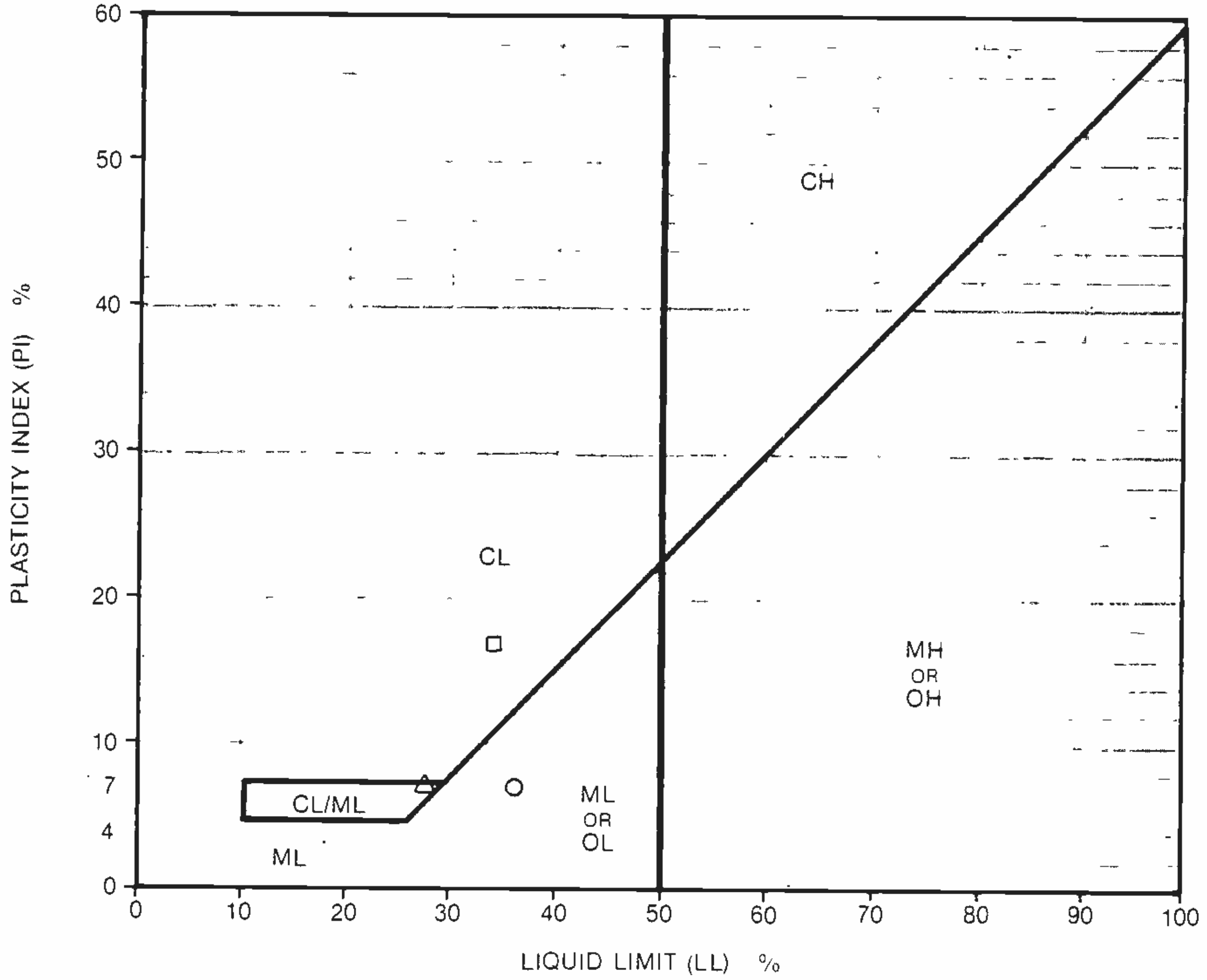
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-20



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
△	28/6 C-5 23'-24' CL/ML	28	20	7	27%
–	28/6 C-7 33'-34' NP	–	–	–	28%
□	28/7 C-5 28'-29' CL	34	17	17	60%
○	28/7 C-7 43'-44' ML/OL	27	20	7	

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PLASTICITY CHART

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METRO RAIL PROJECT

Project No
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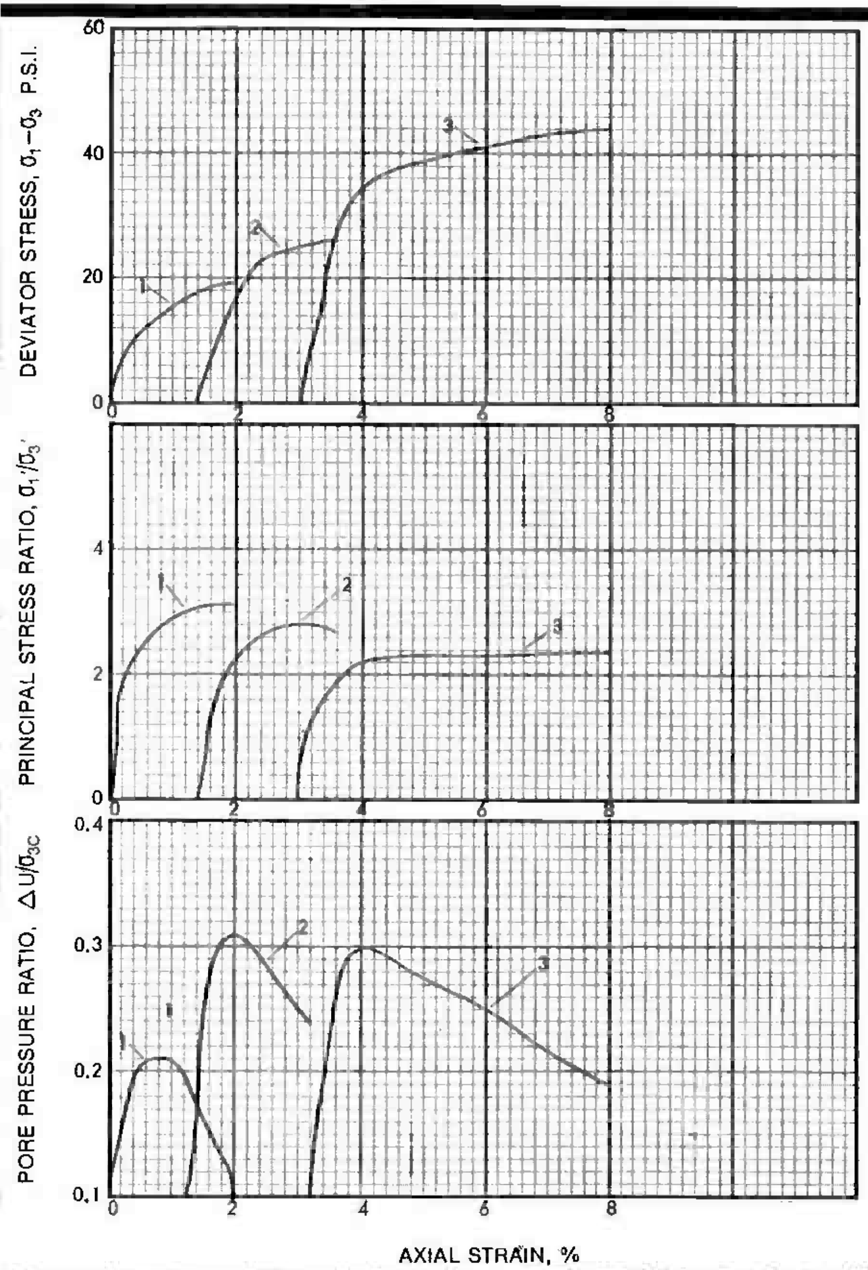
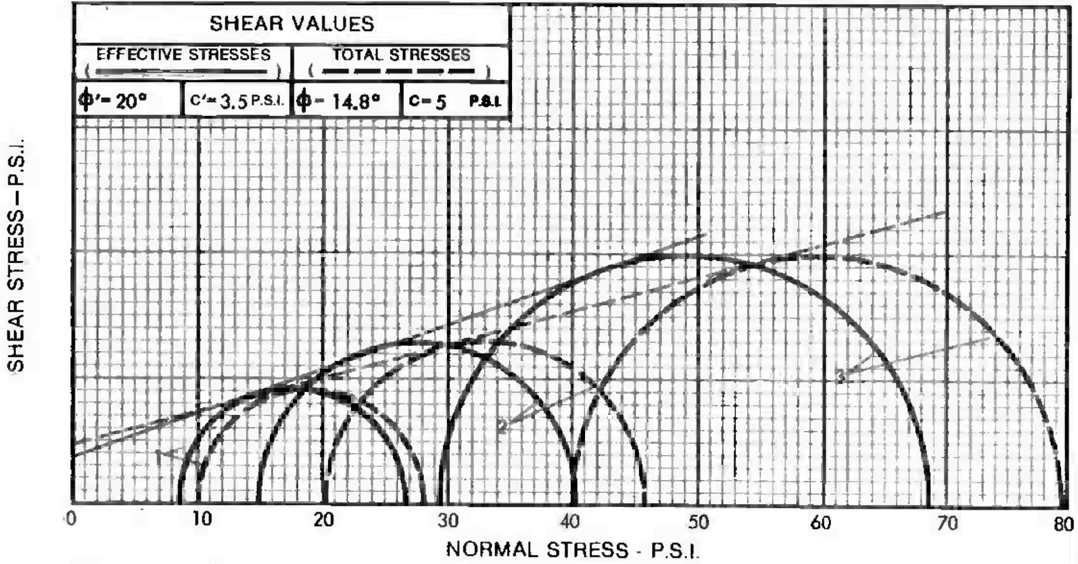
Figure No
C-21

Approved for publication 5/84 by 7C





SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ'= 20°	C'= 3.5 P.S.I.	ϕ= 14.8°	C= 5 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/1	C-14	86.5	ML/CL	5.0	2.42	112.4	17.3	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	10	18.5	1.5	8.5	27.0	TX CUE
	2	20	25.5	5.3	14.7	40.2	PROGRESSIVE
	3	40	39.6	10.7	29.3	68.9	

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

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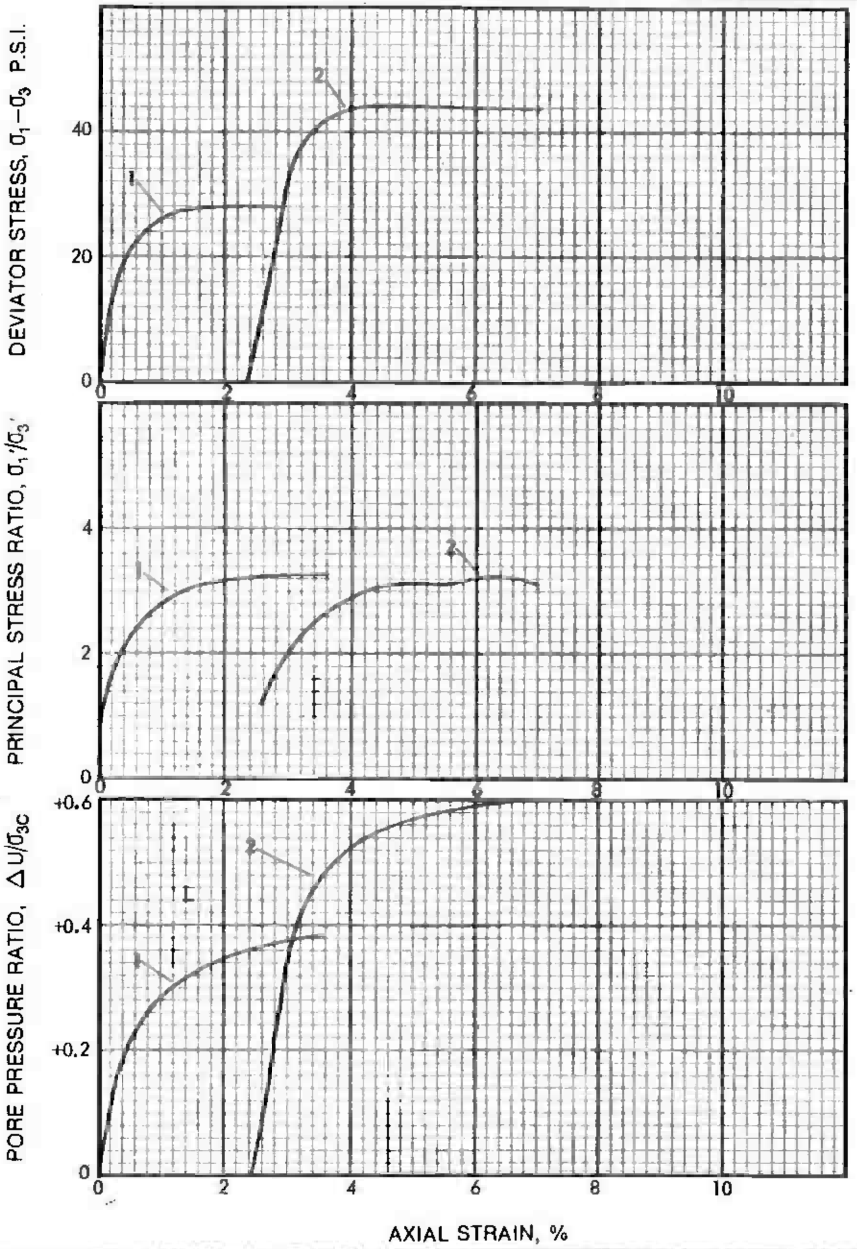
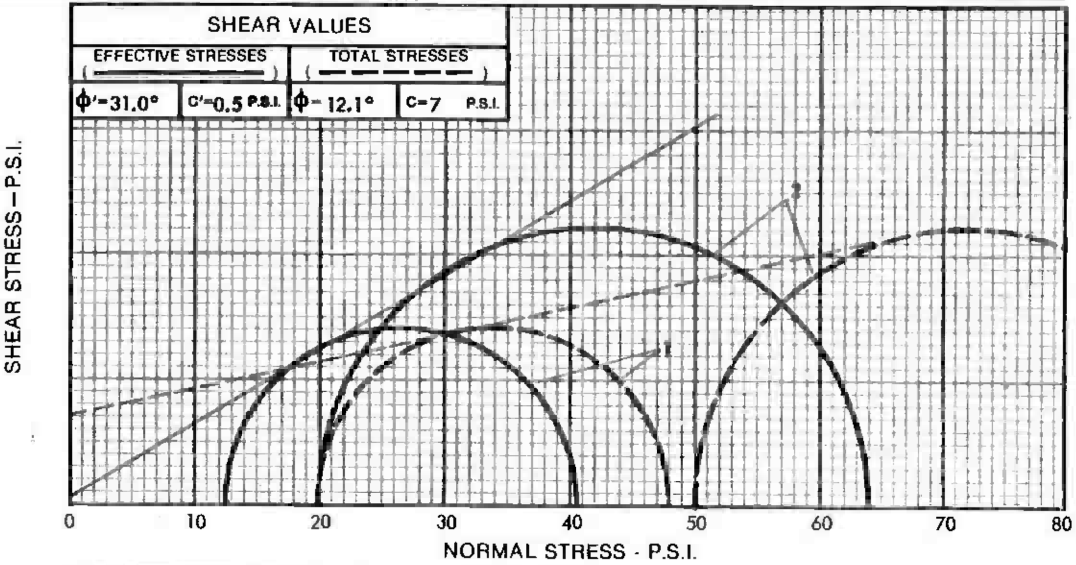
Figure No
C-22

Approved for publication 5/84 by 7C





SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (– – – –)	
ϕ'=31.0°	C'=0.5 P.S.I.	ϕ= 12.1°	C=7 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/2	C-5	22'-23'	CL	5.0	2.42	103.8	16.5	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	20	28.1	7.6	12.4	40.5	TX CUE
	2	50	44.1	30	20.0	64.1	PROGRESSIVE

TRIAXIAL COMPRESSION TESTS

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Figure No.
C-23

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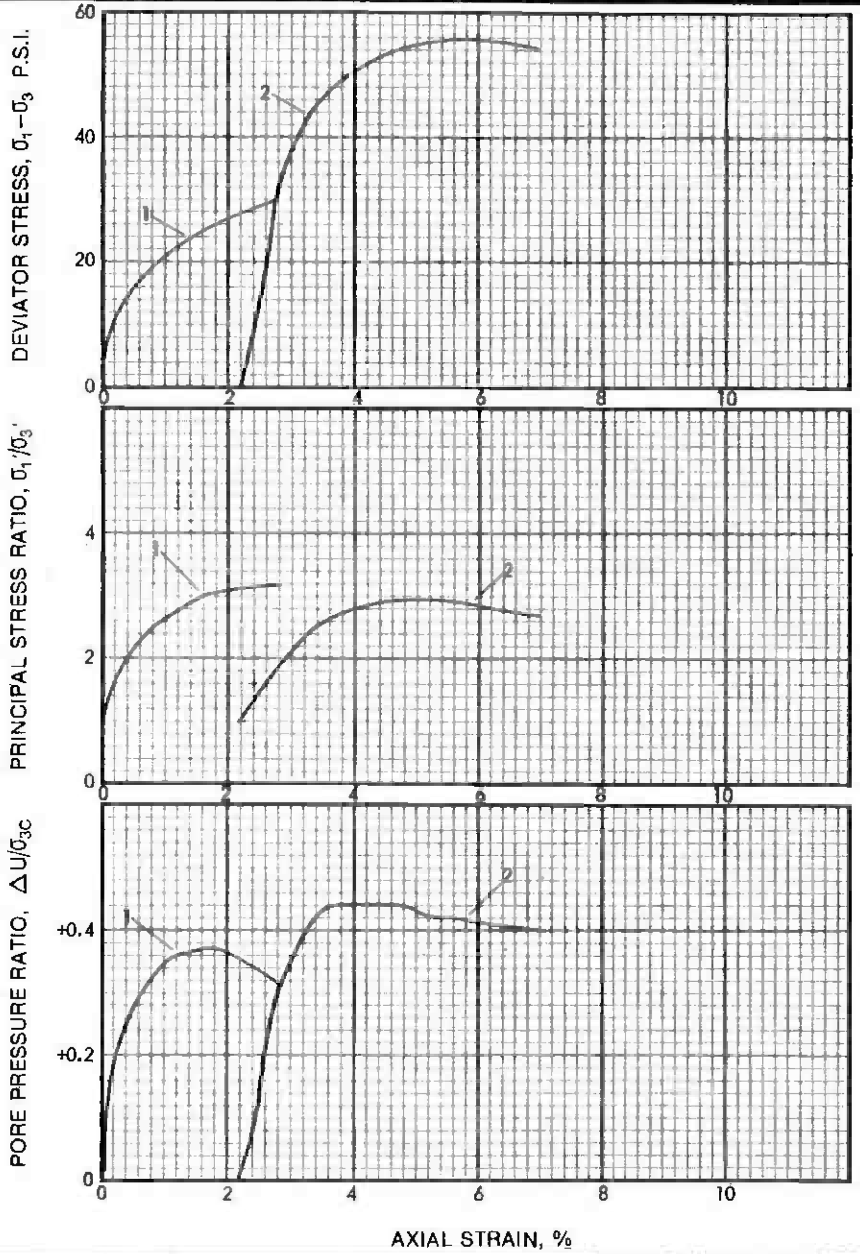

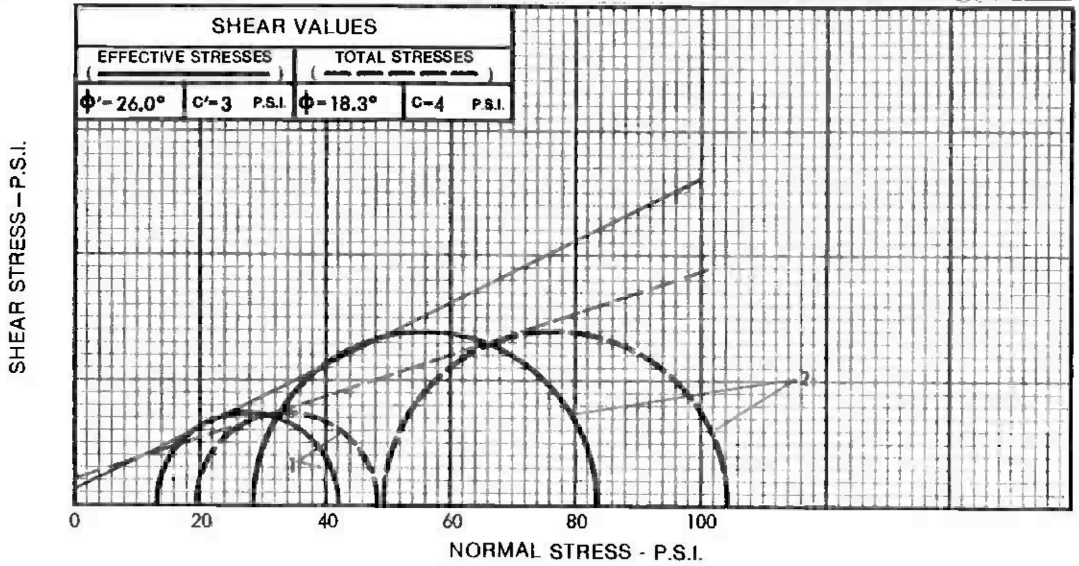

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
$\phi' = 26.0°$	$C' = 3$ P.S.I.	$\phi = 18.3°$	$C = 4$ P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/3	C-11	52'-53'	CL	5.0	2.42	105.9	19.9	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	20	28.8	6.8	13.2	42.0	TX CUE
	2	50	55.2	21.3	28.7	83.9	PROGRESSIVE

TRIAXIAL COMPRESSION TESTS

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Figure No
C-24

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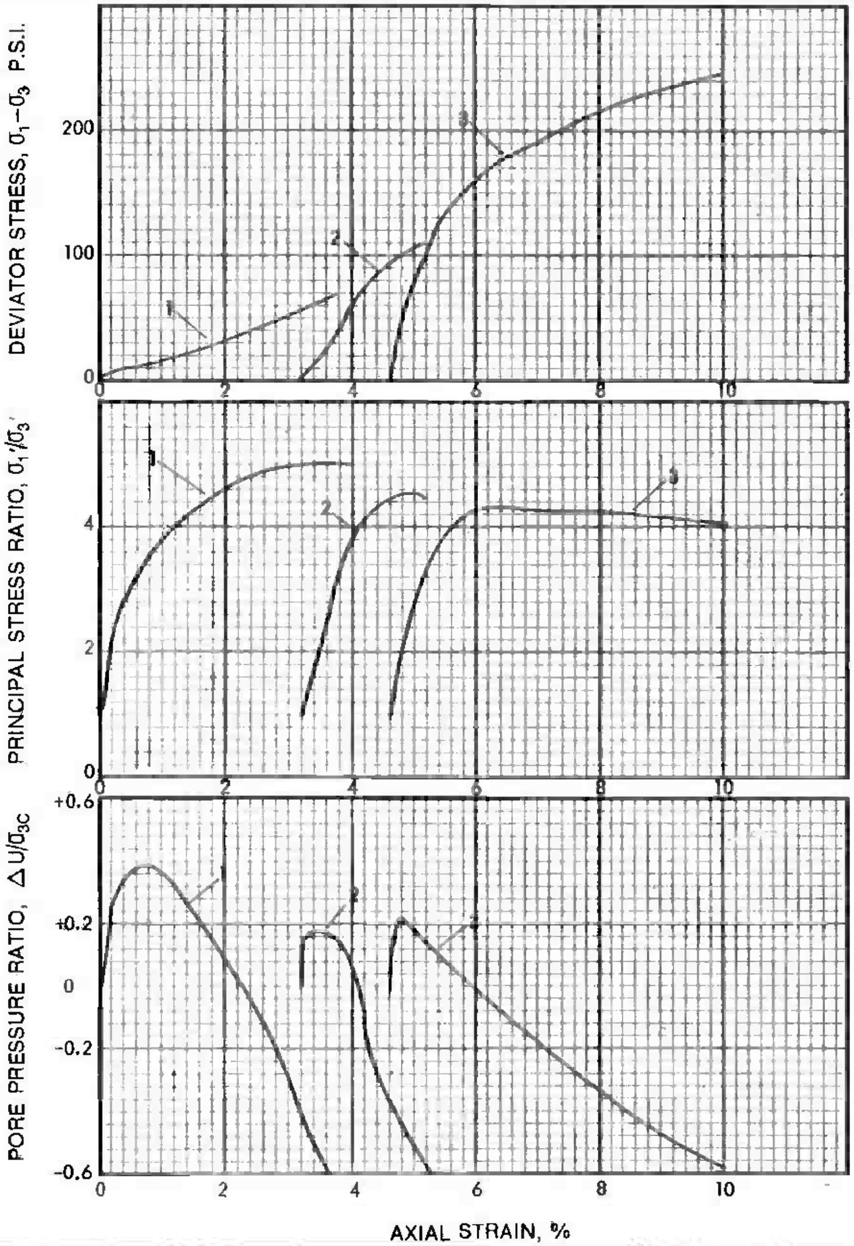

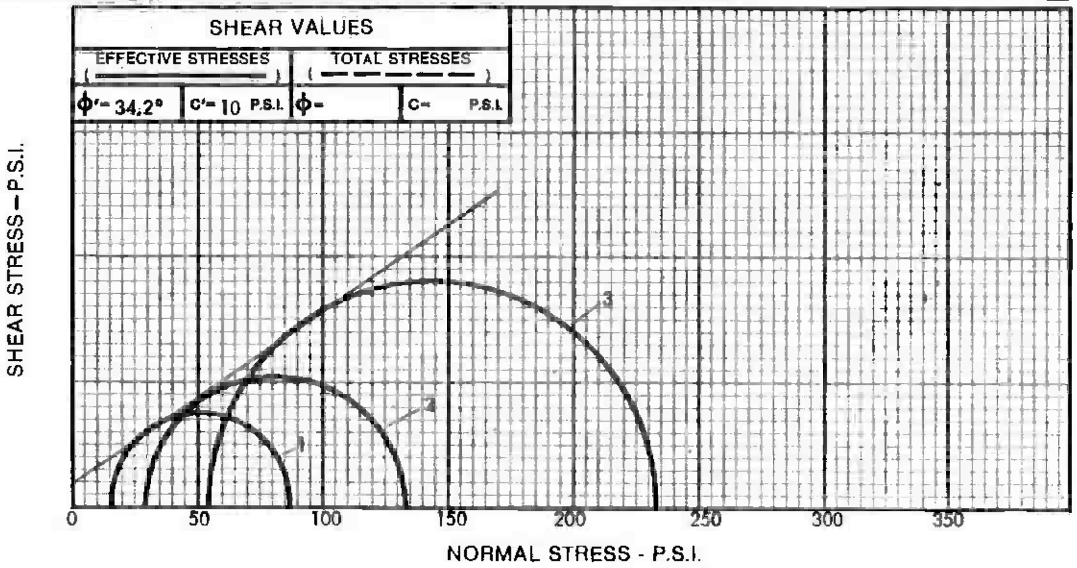

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ'= 34.2°	C'= 10 P.S.I.	ϕ=	C= P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/3	C-16	77'-78'	SM	5.0	2.42	109.0	16.6	5 RING
									CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	10	69.8	-7.2	17.2	87.0	TX CUE
	2	20	103.8	-9.0	29.0	132.8	PROGRESSIVE
	3	50	208.7	-4.0	54.0	133.3	

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

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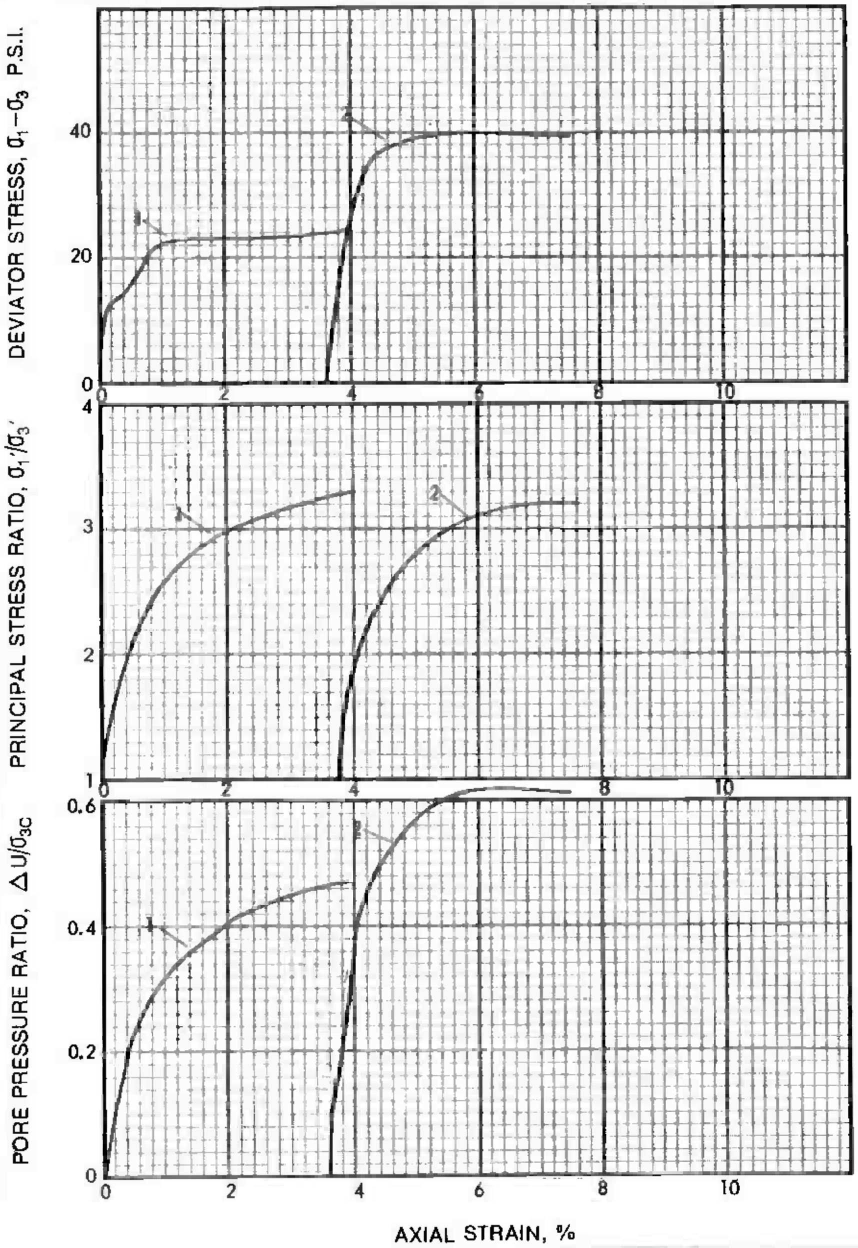
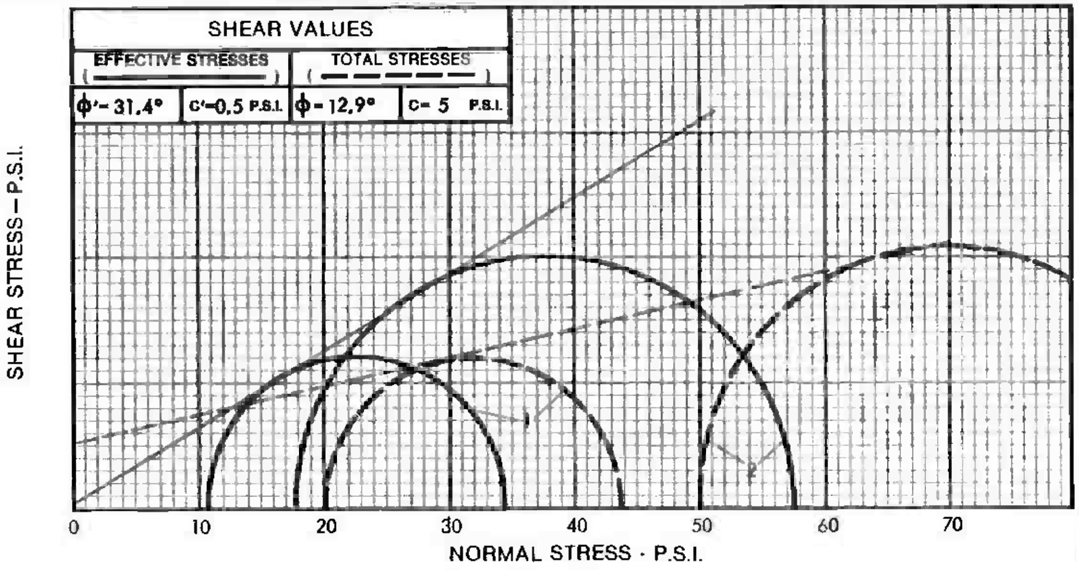
Figure No
C-25

Approved for publication 5/84 by Fc





SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (– – – –)	
ϕ'- 31.4°	c'–0.5 P.S.I.	ϕ - 12.9°	c- 5 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI-FICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/6	C-5	22'-23'	CL	5.0	2.42	106.5	17.6	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	20	24.2	9.4	10.6	34.7	TX CUE
	2	50	39.6	32.0	18.0	57.6	PROGRESSIVE

TRIAXIAL COMPRESSION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

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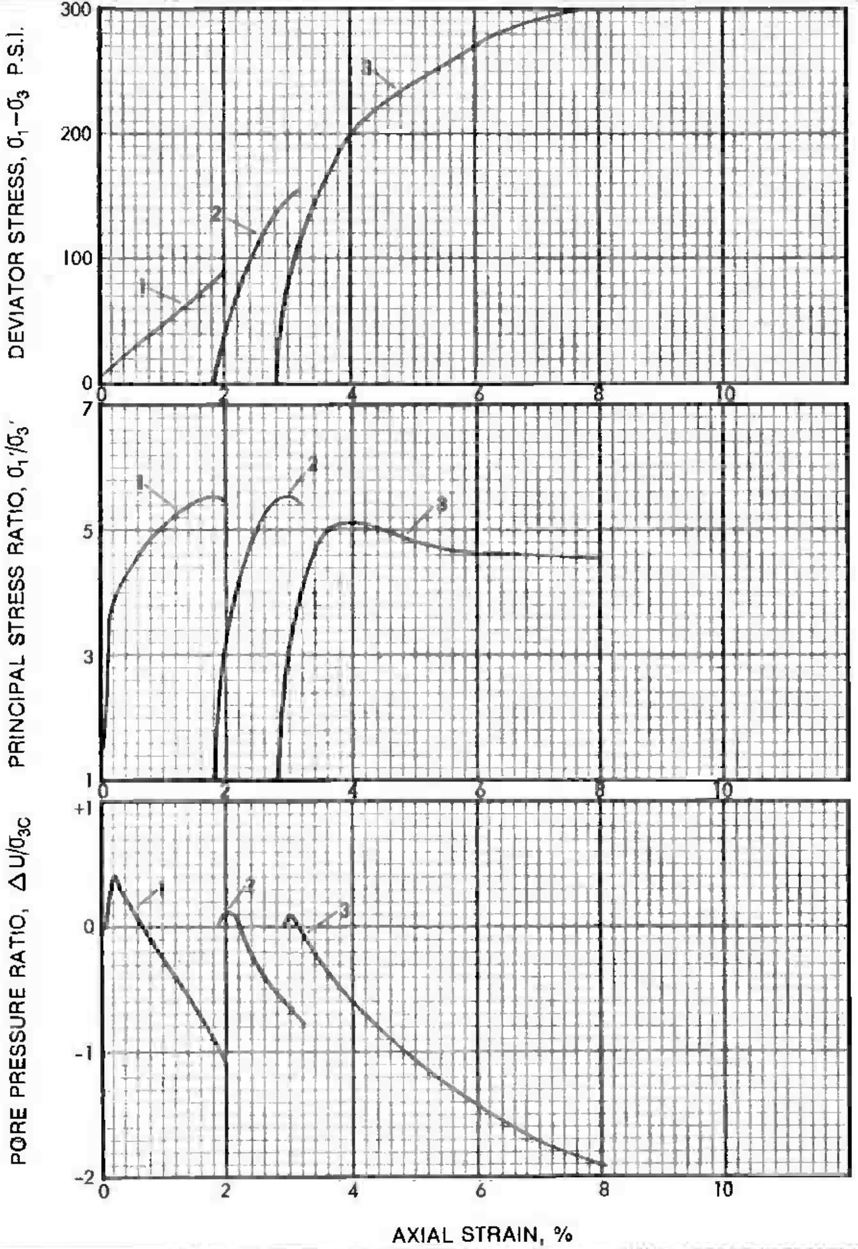
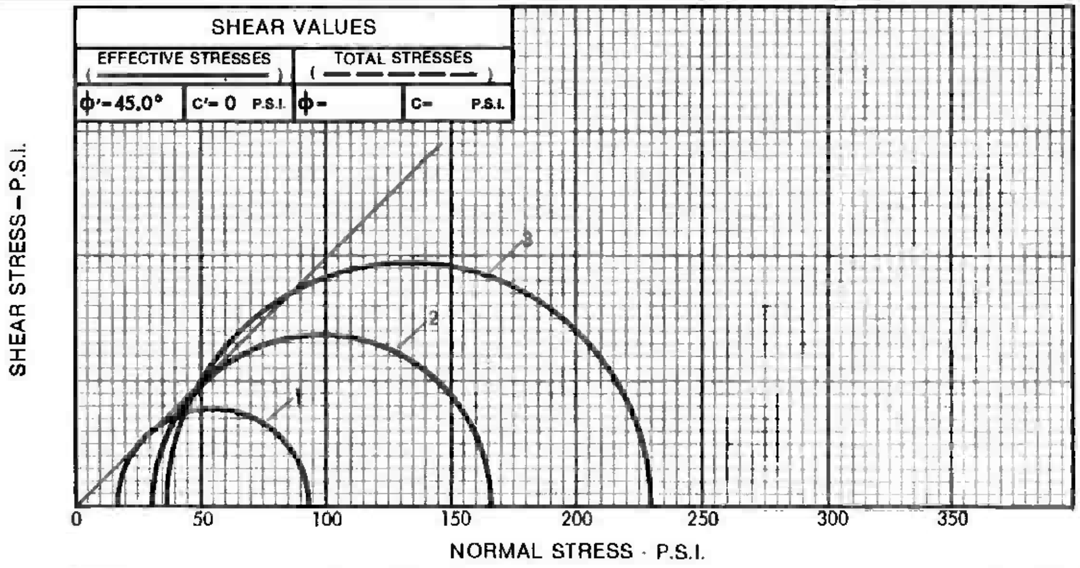
Figure No
C-26

Approved for publication 5/84 by FC





SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'=45.0°	C'= 0 P.S.I.	ϕ=	C= P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
	28/7	C-11	73'-74'	SP	5.0	2.42	117.9	13.3	5 RING
									CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE Δ U (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
	1	10	75.6	-6.8	16.8	92.4	TX CUE
	2	20	136.5	-10.0	30	166.0	PROGRESSIVE
	3	30	186.0	-15.5	35.5	231.5	

TRIAXIAL COMPRESSION TESTS

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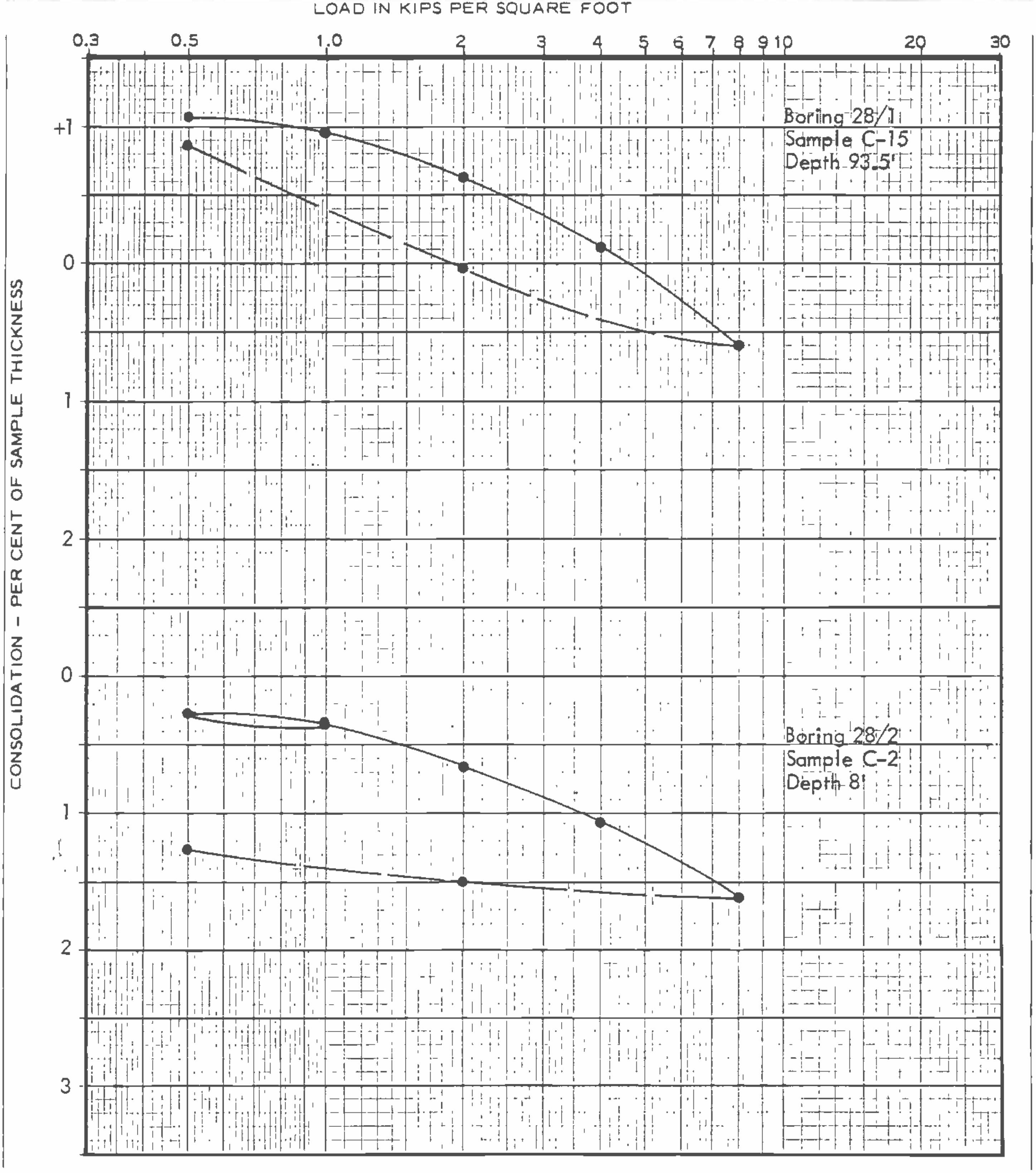
Figure No
C-27



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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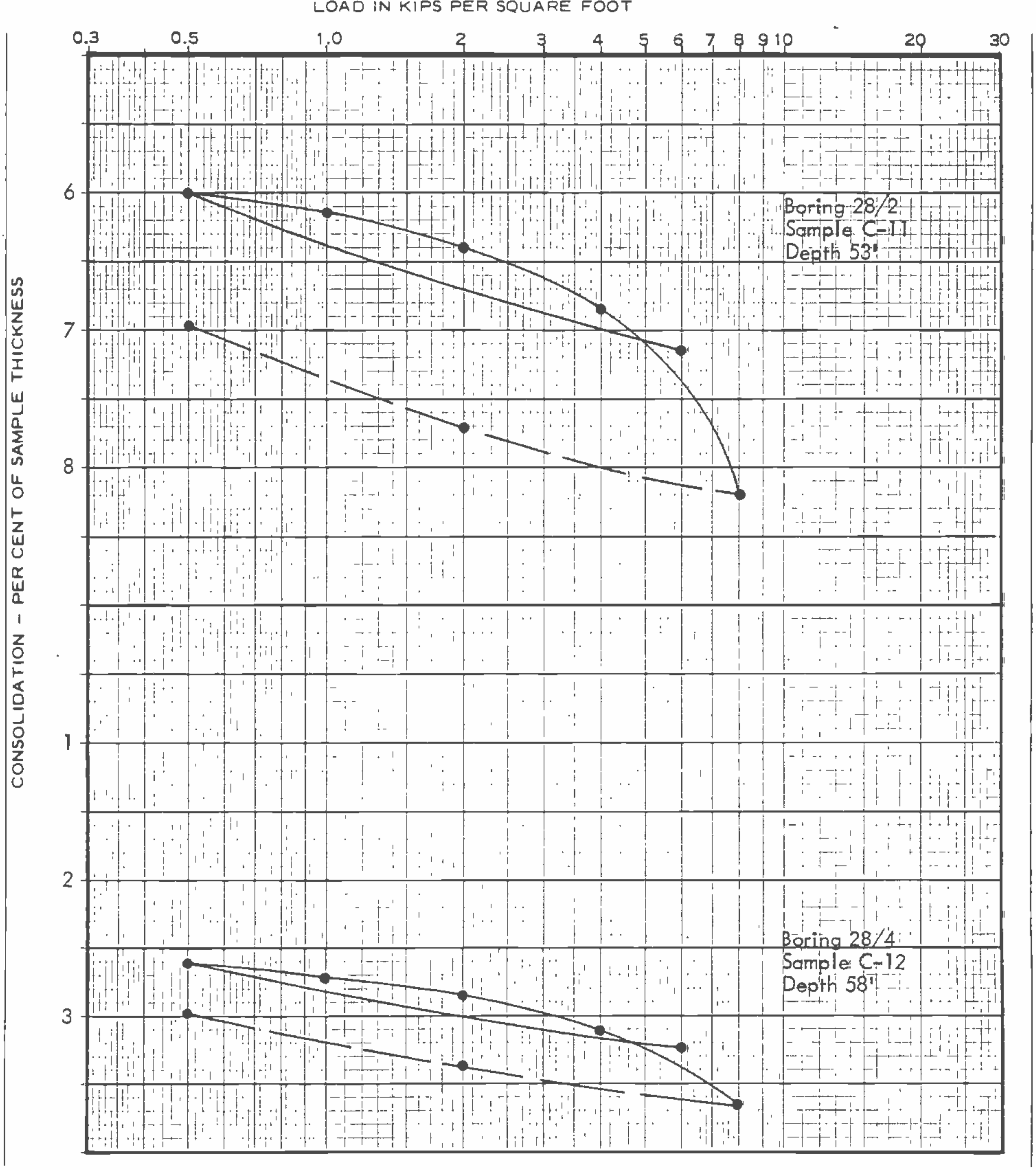
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Figure No
C-28

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 7/84 by FC



● READINGS AFTER SATURATION WITH WATER

Approved for publication 5/8/ by F.C.

CONSOLIDATION TESTS

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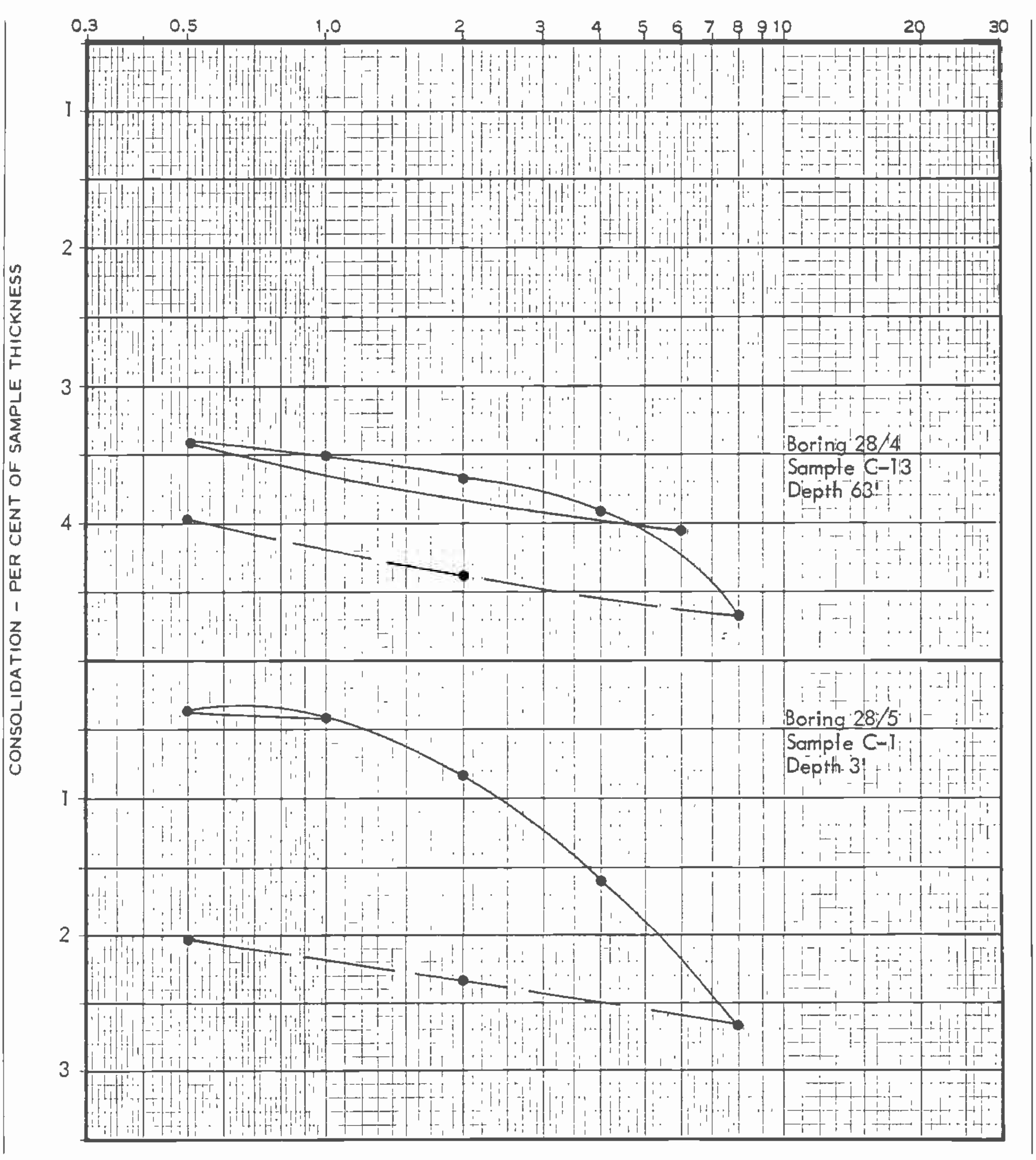
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-29



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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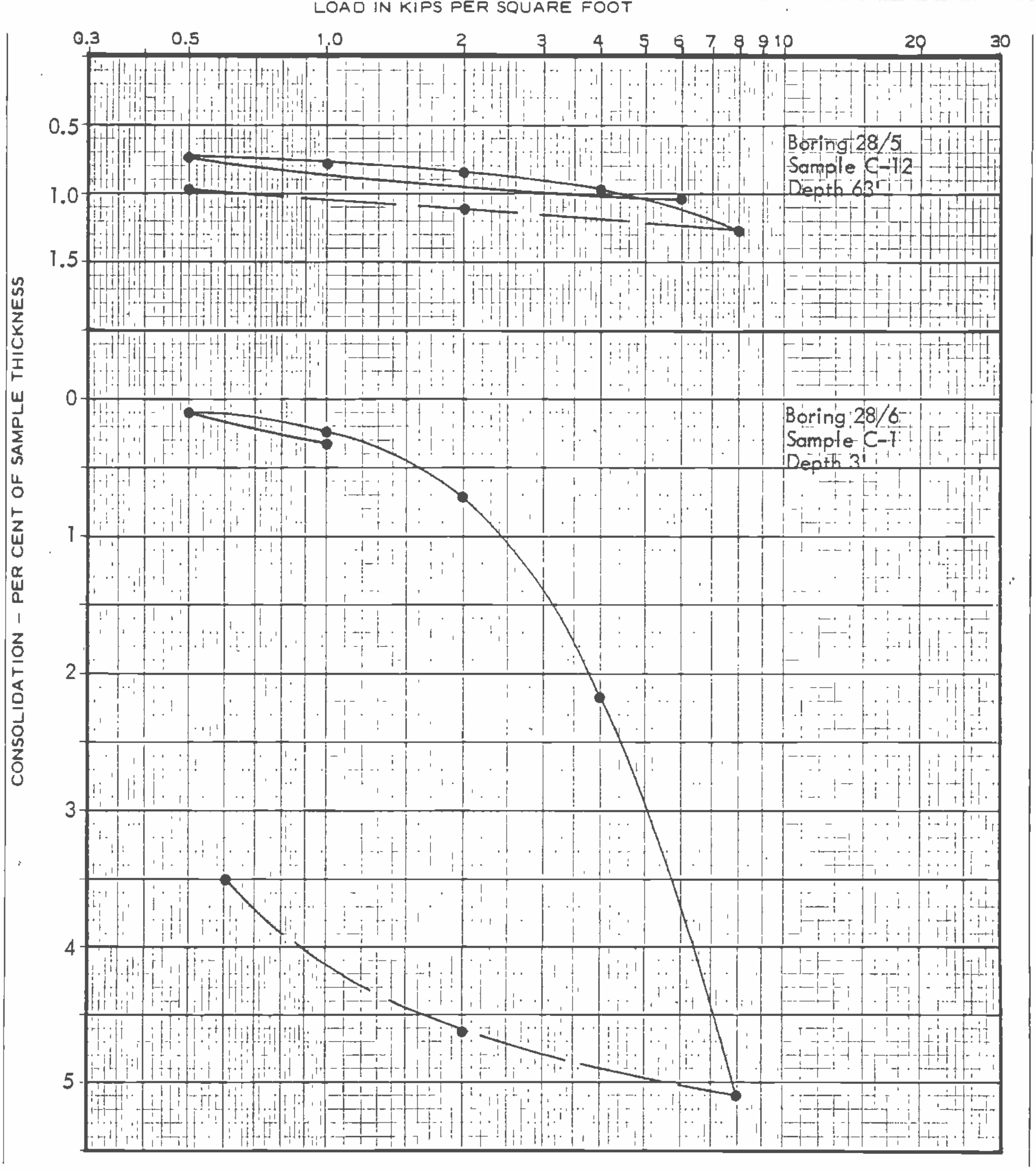
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Figure No.
C-30

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 5/84 by F



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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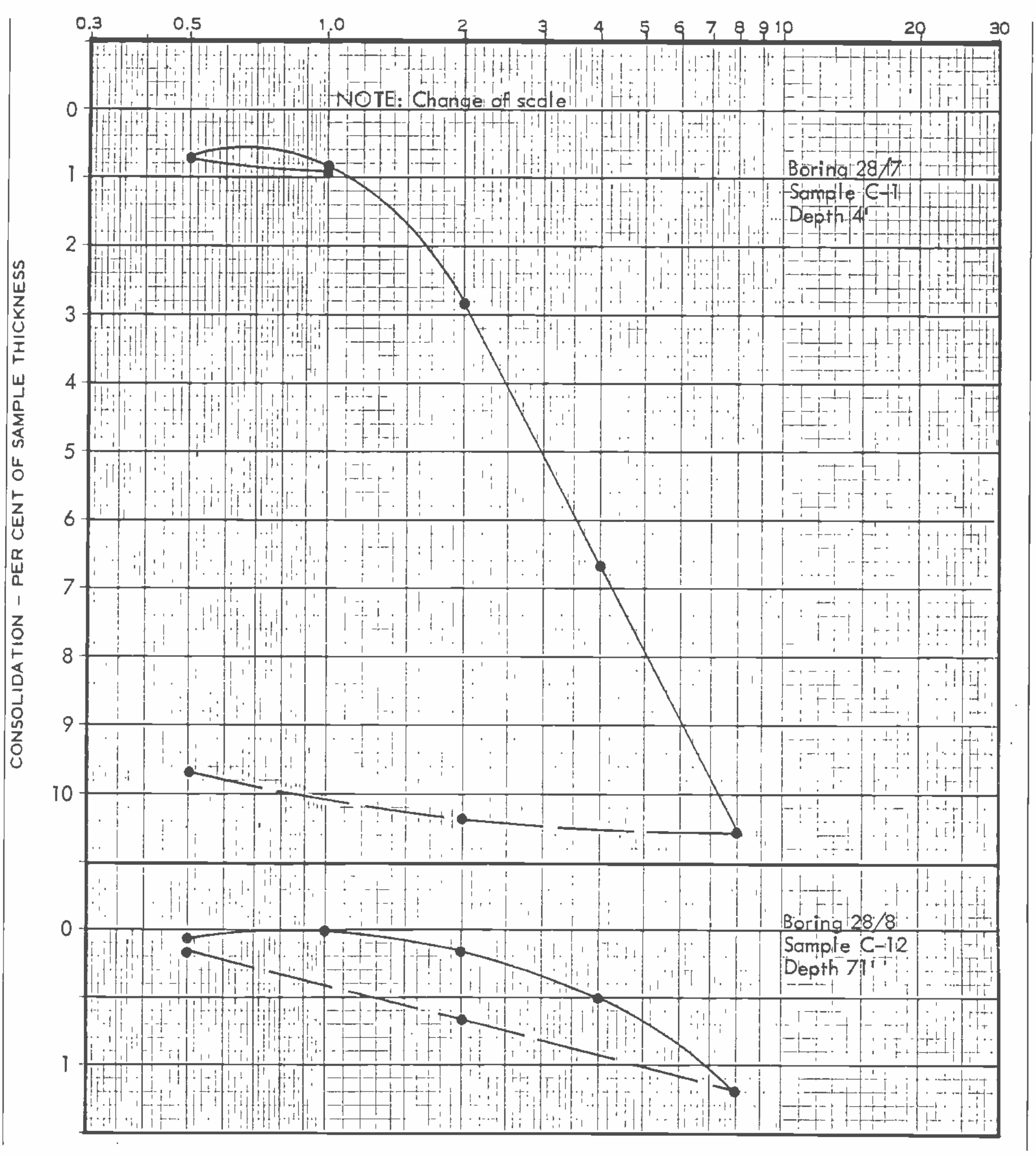
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No
C-31

Approved for publication 5/84 by



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

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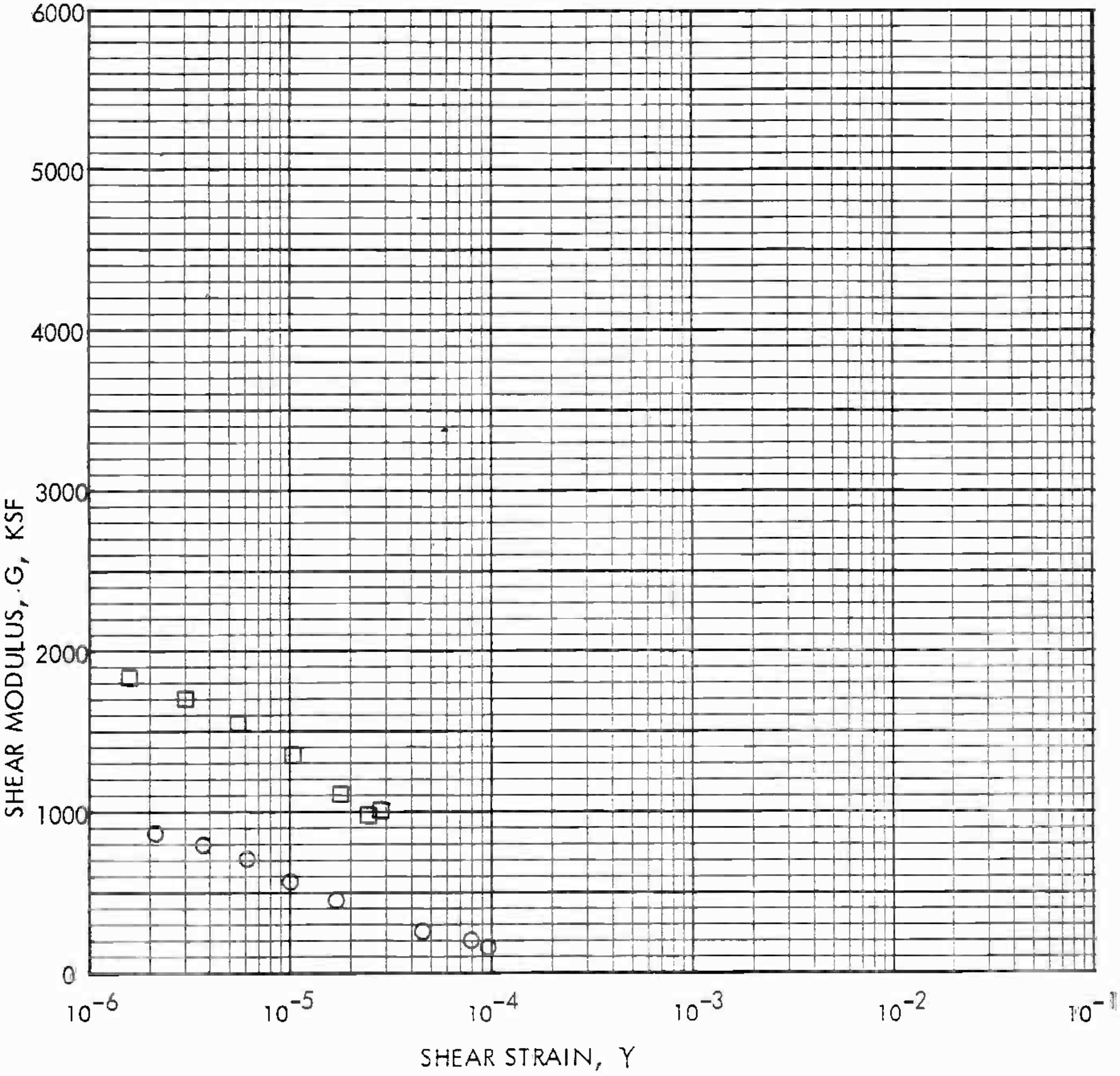
Figure No.
C-32

Approved for publication 5/30 by

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT),	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
28	C-7	139	122	13	15	○
					50	□

Sample Description: Red-Brown Clayey fine to coarse Sand, dense

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RESONANT COLUMN TEST

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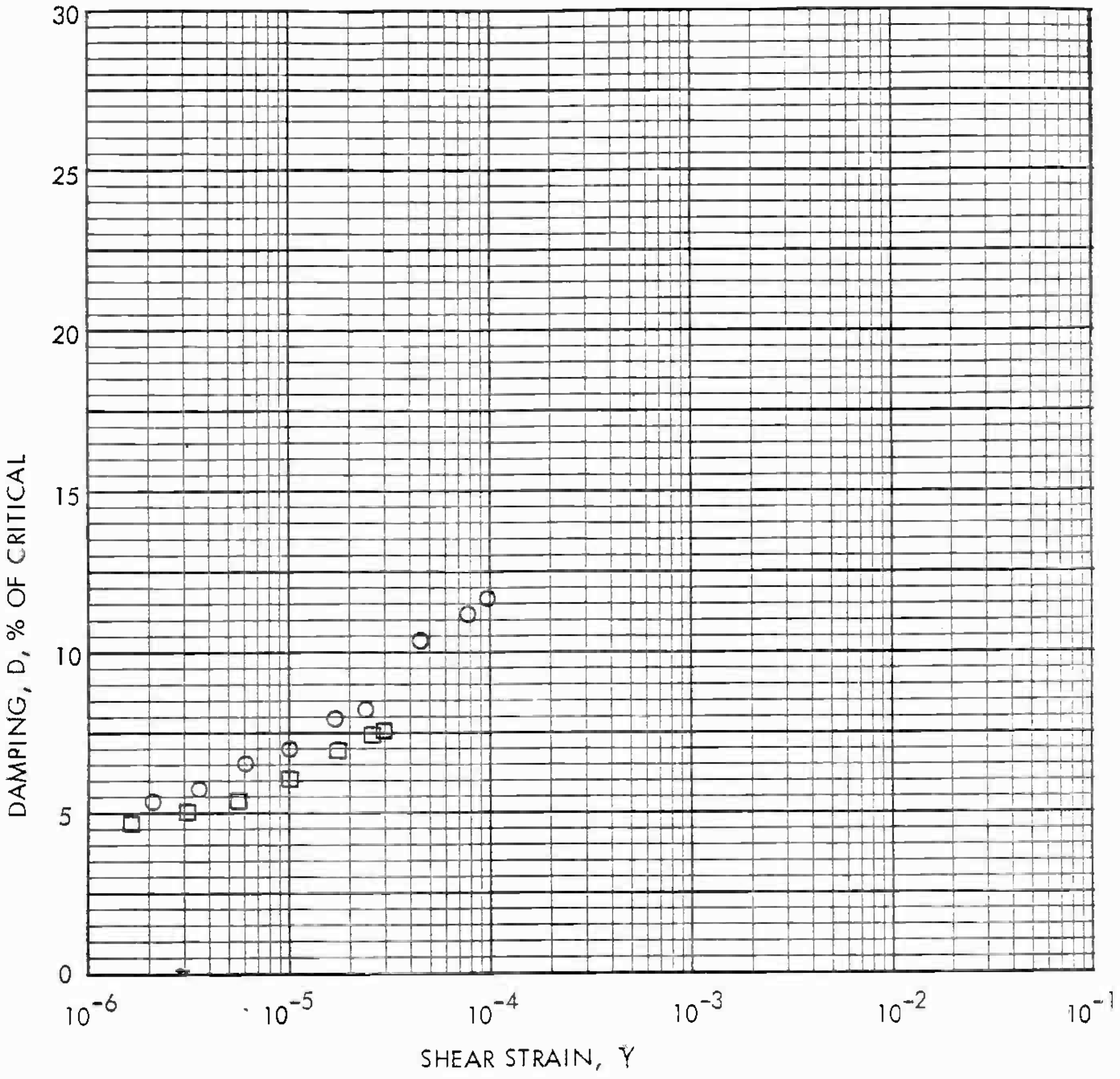
Figure No.
C-33

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(PSI)	$\bar{\sigma}_c$(PSI)	SYMBOL
28	C-7	139	122	13	15	○
					50	□

Sample Description: Red-Brown Clayey fine to coarse Sand, dense

RESONANT COLUMN TEST

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Figure No.
C-34

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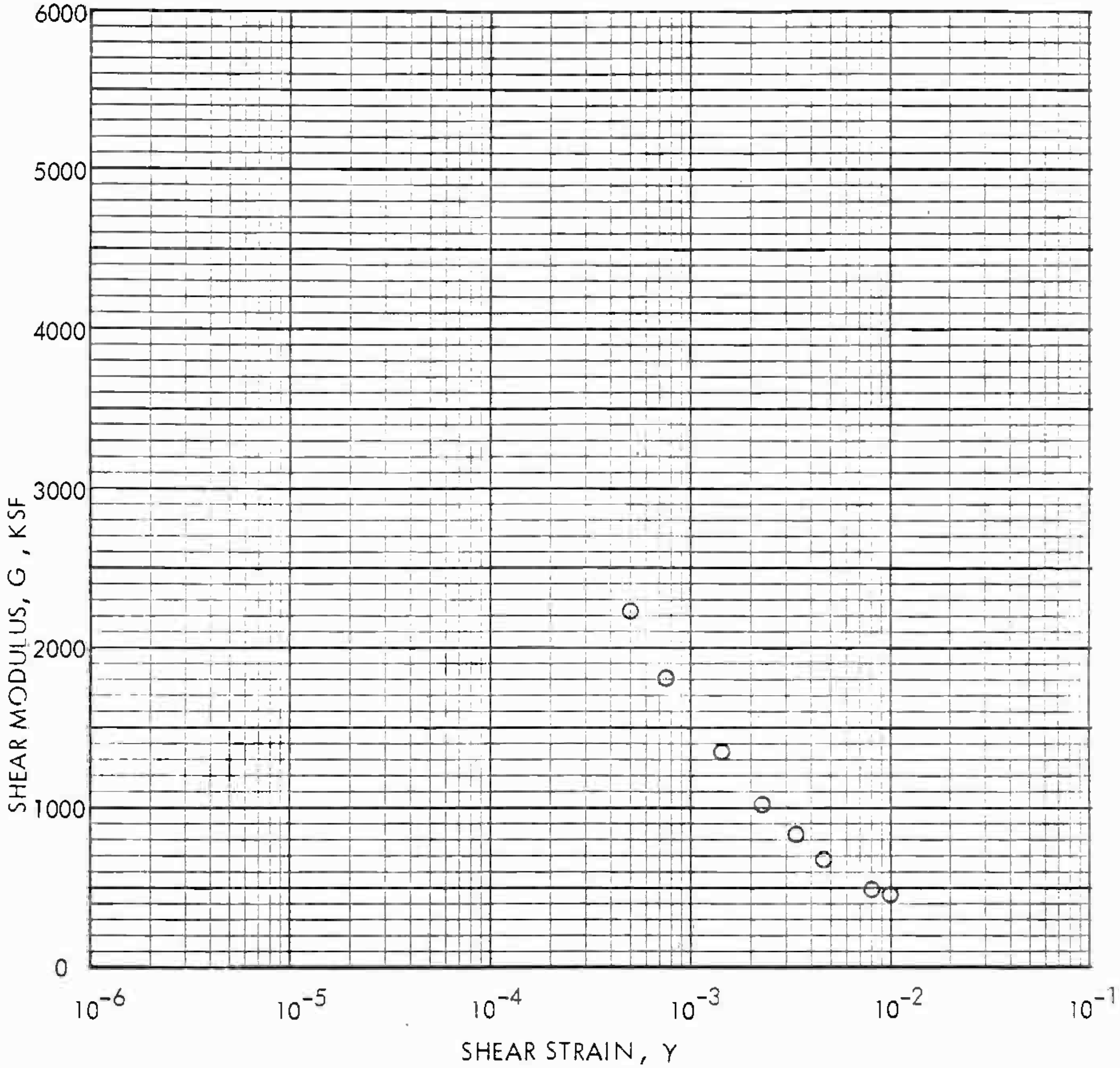
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
28	C-9	180	123	12	30

Sample Description: Brown Clayey Sand; dense, medium graded

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DYNAMIC TRIAXIAL TEST

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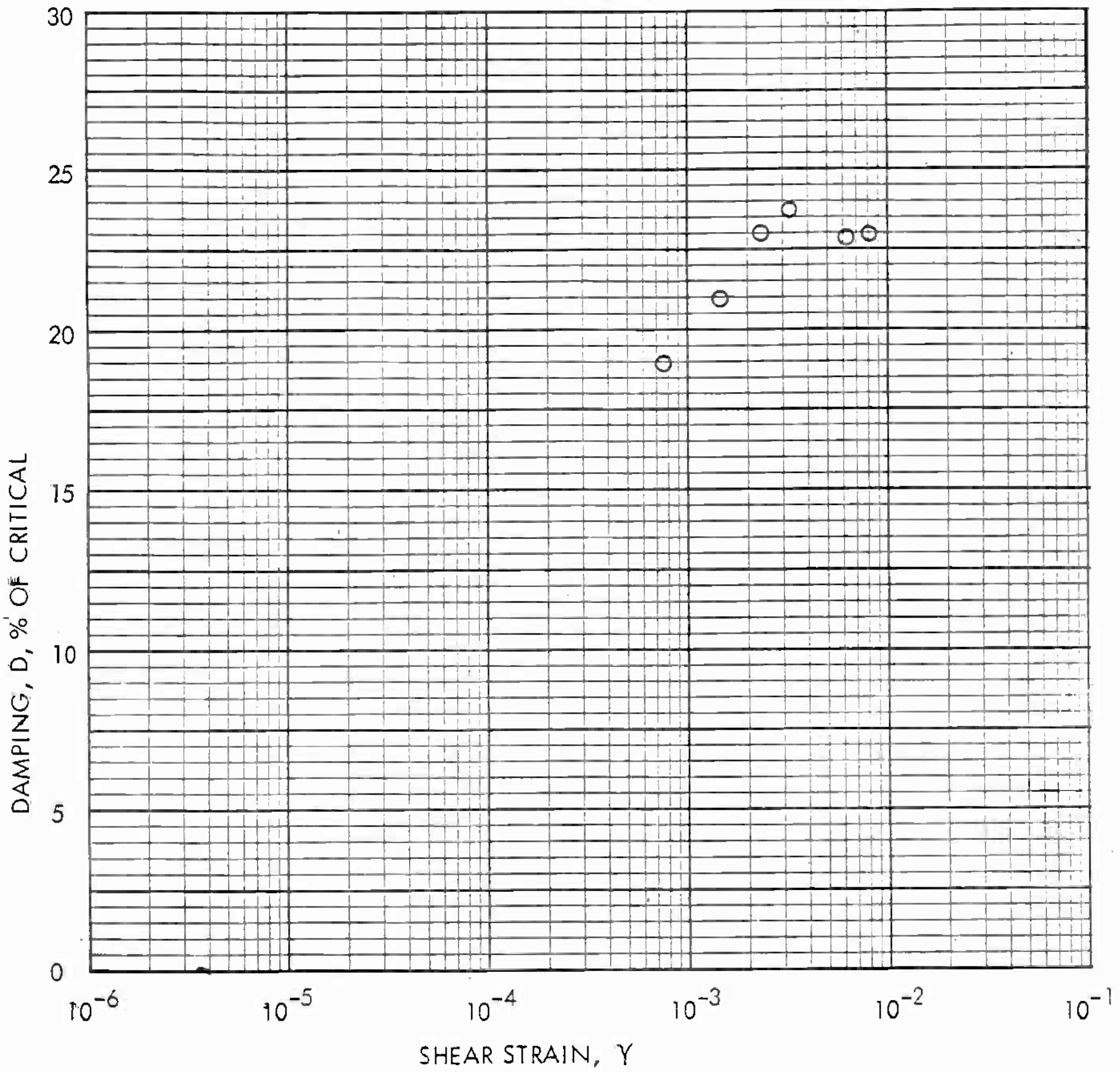
Figure No.
C-35

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
28	C-9	180	123	12	30

Sample Description: Brown Clayey Sand; dense, medium graded

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DYNAMIC TRIAXIAL TEST

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Figure No.
C-36

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Appendix D

Water Quality Analysis

APPENDIX D WATER QUALITY ANALYSIS

D.1 RESULTS

Water samples were taken from Boring CEG-28A during the 1983 investigation. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for future tunneling activities. The chemical constituents tested are attached.

D.2 FIELD PROGRAM

The borehole was flushed and established as piezometers. At a later date (often several weeks) the established piezometer hole was again flushed and cleaned out. Upon achieving a clean hole, water samples were collected with an air-lifting procedure from various depths within the borehole. The water samples were collected in sterilized one-quart glass containers which were properly identified and marked in the field. The water samples were delivered to Brown and Caldwell Consulting Engineers for testing.

Converse Ward Davis Dixon

Lab No. P81-03-152-2

Sample labeled: Hole 28A-2"

No. Samples : 4
Sampled By : Client
Brought By : Client
Date Received: 3-19-81

Conductivity: 920 µ mhos/cm

Turbidity: NTU

pH 7.8 @ 25°C
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)		Milli-equivalents per liter
Cations determined:			
Calcium, Ca	37		1.83
Magnesium, Mg	16.5		1.36
Sodium, Na	224		9.74
Potassium, K	5.8		0.15
		Total	13.08
Anions determined:			
Bicarbonate, as HCO_3	312		5.11
Chloride, Cl	76		2.13
Sulfate, SO_4	272		5.67
Fluoride, F	0.82		0.06
Nitrate, as N	0.39		0.01
		Total	12.98
Carbon dioxide, CO_2, Calc.	7.1		
Hardness, as $CaCO_3$	174		
Silica, SiO_2	12		
Iron, Fe	1.6		
Manganese, Mn	< 0.05		
Boron, B	1.16		
Total Dissolved Minerals, (by addition: HCO_3->CO_3)	805		

Appendix E

Technical Considerations

APPENDIX E TECHNICAL CONSIDERATIONS

E.1 SHORING PRACTICES IN THE LOS ANGELES AREA

E.1.1 General

Deep excavations for building basements in the Los Angeles area are commonly supported with soldier piles with tieback anchors. Three case studies involving deep excavations into materials similar to those anticipated at the proposed site are presented below.

E.1.2 Atlantic Richfield Project (Nelson, 1973)

This project involved three separate shored excavations up to 112 feet in depth in the siltstones of the Fernando Formation. The project is located just north of Boring CEG-9, and the proposed location of the 7th/Flower Station. Key elements of the design and construction included:

- Basic subsurface material was a soft siltstone with a confined compressive strength in the range of 5 to 10 ksf. It contained some very hard layers, seldom more than 2 feet thick. All materials were excavated without ripping, using conventional equipment. Up to 32 feet of silty and sandy alluvium overlaid the siltstone.

- Volume of water inflow was small and excavations were described as typically dry.

- Shoring system consisted of steel, wide flange (WF) soldier piles set in pre-drilled holes, backfilled with structural concrete in the "toe" and a lean concrete mix above. The soldier pile spacing was typically 6 feet.

- Tieback anchors consisted of both belled and high-capacity friction anchors.

- On the side of one of the excavations a 0.66H:1V (horizontal:vertical) unsupported cut, 110 feet in height, was excavated and sprayed with an asphalt emulsion to prevent drying and erosion.

- Timber lagging was not used between the soldier piles in the siltstone unit. However, an asphalt emulsion spray and wire mesh welded to the piles was used.

- The garage excavation (when 65 feet deep) survived the February 9, 1971 San Fernando earthquake (6.4 Richter magnitude) without detectable movement. The excavation is about 20 miles from the epicenter and experienced an acceleration of about 0.1g. The shoring system at the plaza, using belled anchors, moved laterally an average of about 4 inches toward the excavation at the tops of the piles, and surface subsidence was on the order of 1 inch; surface cracks developed on the street, but there was no structural damage to adjacent buildings. Subsequent shoring used high capacity friction anchors and reportedly moved laterally less than 2 inches.

E.1.3 Century City Theme Towers (Crandall, 1977)

This project involved a shored excavation between 70 and 110 feet deep in Old Alluvium deposits. Immediately adjacent to the excavation (about 20 feet away), was a bridge structure supported on piles 60 feet below the ground surface. The project is located about one mile west of Boring CEG-20 and the proposed location of the Fairfax Avenue Station. Key elements of the design and construction included:

- Basic subsurface materials were stiff clays and dense silty sands and sands. The permanent ground water table was below the level of the excavation, although minor seeps from perched ground water were encountered.
- Shoring system consisted of steel WF soldier piles placed in 36-inch diameter drilled holes spaced 6 feet on center.
- As the excavation proceeded, pneumatic concrete was placed incrementally in horizontal strips to create the finished exterior wall. The concrete which was shot against the earth acted as the lagging between soldier piles.
- Tieback anchors consisted of high-capacity 12- and 16-inch diameter friction anchors.
- Actual load imposed on the wall by the adjacent bridge was computed and added to the design wall pressures as a triangular pressure distribution.
- Maximum horizontal deflection at the top of the wall was 3 inches, while the typical deflection was less than 1 inch. Adjacent to the existing bridge, the deflections were essentially zero, with the tops of most of the soldier piles actually moving into the ground due to the high prestress loads in the anchors.
- Survey of the bridge pile caps indicated practically no movement.

E.1.4 St. Vincent's Hospital (Crandall, 1977)

This project involved a shored excavation up to 70 feet deep into the claystones and siltstones of the Puente Formation. Immediately adjacent to the excavation (about 25 feet away) was an existing 8-story hospital building with one basement level supported on spread footings. The project is located about 1/3 mile north of Boring CEG-11 and the proposed location of the Alvarado Street Station. Key elements of the design and construction included:

- Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20° to 40°. Although the permanent ground water level was below the excavation level, perched zones of significant water seepage were encountered.
- Shoring system consisted of steel WF soldier piles placed in 20-inch diameter drilled holes spaced at 6 feet on center.
- Tieback anchors consisted of high-capacity friction anchors.

- Theoretical load imposed on the wall by the adjacent building was computed and added to the design wall pressure. The existing building was not underpinned; thus, the shoring system was relied upon to support the existing building loads.

- Shoring performed well, with maximum lateral wall deflection of about 1 inch and typical deflections less than 1/4 inch. There was no measurable movement of the reference points on the existing building.

E.1.5 Design Lateral Load Practices

Table E-1 summarizes the design lateral loads used for nine shored excavations in the general site vicinity. Based on these projects, the average equivalent uniform pressure for excavations in alluvium is 15.6H-psf (H = depth of the excavation). For excavations in the Puente or Fernando the average value used is 14.5H-psf.

According to Terzaghi and Peck's rules, the design pressure in granular soils would be equal to 0.65 times the active earth pressure. Assuming a friction angle of 37°, the equivalent design pressure should equal about 22H-psf. For hard clays, the recommended value ranges from 0.15 to .30 (equivalent rectangular distribution) times the soils unit weight or at least 18H-psf.

Thus, the local design practices are some 20% less than those indicated by Peck's rules.

TABLE E-1

SHORING LOADS IN LOS ANGELES AREA

PROJECT LOCATION	EXCAVATION DEPTH (ft)	SOIL CONDITIONS	ACTUAL DESIGN PRESSURE (P)
Broadway Plaza Near 7th/Flower Station	15 to 30	Fill over Alluvium Sands	19.0H
500 South Hill	25	Fill over Sands & Gravel	22.0H
Tishman Building Wilshire/Normandie Station	25	Alluvium-Clays, Sand, Silt	19.0H
Equitable Life Wilshire/Mariposa Avenues	55	Alluvium Sand/Siltstone	20.0H
Arco Flower Street/5th to 6th	70 to 90	Alluvium over Claystone	16.0H
Century City	70 to 110	Alluvium-Clays & Sands	18.0H
St. Vincent's Hospital Near 3rd & Alvarado	70	Thin Alluvium over Puente	15.0H
Oxford Plaza Near 7th/Flower	40	Fill & Alluvium over Siltstone	21.0H
Bank Building* 2nd & San Pedro	40	Alluvium (including Sand & Gravel over Siltstone)	20H

* Considerable caving problems were encountered installing tiebacks in dry gravelly deposits in one section of excavation.

Note:

1. All shoring systems were soldier piles.
2. All pressure diagrams were trapezoidal.
3. Equivalent pressure equals a uniform rectangular distribution.

E.2 SEISMICALLY INDUCED EARTH PRESSURES

The increase in lateral earth pressure due to earthquake forces has usually been taken into consideration by using the Monobe-Okabe method which is based on a modification of Coulomb's limit equilibrium earth pressure theory. This simple pseudo-static method has been applied to the design of retaining structures both in the U.S. and in numerous other countries around the world, mainly because it is simple to use. However, just as the use of the pseudo-static method is not really appropriate for evaluating the seismic stability of earth dams, those same shortcomings are also applicable when using the method to evaluate dynamic lateral pressures.

During an earthquake the inertia forces are cyclic in nature and are constantly changing throughout its duration. It is unrealistic to replace these inertia forces by a single horizontal (and/or vertical) force acting only in one direction. In addition, the selection of an appropriate value of the horizontal seismic coefficient is completely arbitrary. Nevertheless, the pseudo-static method is still being used since it provides a simple means for assessing the additional hazard to stability imposed by earthquake loadings.

Monobe-Okabe originally developed an expression for evaluating the magnitude of the total (static plus dynamic) active earth pressure acting on a rigid retaining wall backfilled with a dry cohesionless soil. The method was developed for dry cohesionless materials and based on the assumptions that:

- ° The wall yields sufficiently to produce minimum active pressures.
- ° When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential sliding surface.
- ° The soil behind the wall behaves as a rigid body so that accelerations are uniform throughout the mass.

Monobe-Okabe's method gives only the total force acting on the wall. It does not give the pressure distribution nor its point of application. Their formula for the total active lateral force on the wall, P_{AE}, is as follows:

$$P_{AE} = 1/2\gamma\ H^2(1-k_v)K_{AE}$$

Where:

$$K_{AE} = \frac{\text{COS}^2\ (\phi-\theta-\beta)}{\text{COS}\ \theta\ \text{COS}^2\beta\text{COS}\ (\delta+\beta+\theta)\ \ 1+\left(\frac{\sqrt{\text{SIN}\ (\phi+\delta)\ \text{SIN}\ (\phi-\theta-i)}}{\text{COS}\ (\delta+\beta+\theta)\ \text{COS}\ (i-\beta)}\right)^2}$$

$\theta = \tan^{-1} \frac{Kh}{1-Kv}$

γ = unit weight of soil

ϕ = angle of internal friction of soil

i = angle of soil slope to horizontal

β = angle of wall slope to vertical

k_h = horizontal earthquake coefficient

K_v = vertical earthquake coefficient

δ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes,

$$KAE = \frac{COS^2(\phi-\theta-\beta)}{COS\ \theta\ COS\ (\delta+\theta)\left(1+\frac{\sqrt{SIN\ (\theta+\delta)\ SIN\ (\phi-\theta)}}{COS\ (\theta+\delta)}\right)^2}$$

The seismic component, ΔP_{AE}, of the total lateral load P_{AE} can be determined by the following equation:

$$\Delta P_{AE} = 1/2\ \gamma(total) H^2\ \Delta K_{AE}$$

Where:

$$\Delta K_{AE} = K_{AE}\ (static+seismic) - K_{AE}\ (static)$$

Inspection of actual acceleration time histories recorded during strong motion earthquakes indicates that the accelerations are quite variable both in amplitude and with time. For any given acceleration component the values fluctuate significantly during the entire duration of the record. Statistical analyses of the positive and negative peaks do indicate, however, that when one considers the entire record there are generally an equal number of positive and negative peaks of equal intensity. In the past it has been common practice to use the peak value of acceleration recorded during the earthquake as a value of engineering significance. However, this peak value might occur only once during the entire earthquake duration and is usually not representative of the average acceleration which might be established for the entire duration of shaking.

It has been common practice in the past to ignore the effects of the vertical acceleration and to set the value of the vertical earthquake coefficient, k_v, equal to zero when using Monobe-Okabe's equation. This appears reasonable in the "light" of the above discussion since the vertical acceleration will act in upward direction about as often as it will act in the downward direction. It has also been common practice to set the value of the horizontal seismic coefficient, k_h, equal to the peak ground acceleration.

This is extremely conservative since the peak acceleration acts only on the wall for an instant of time. In addition, for a deep excavation the soil mass behind the wall will not move as a rigid body and will have a seismic coefficient significantly less than the peak ground acceleration (analogous to a horizontal seismic coefficient acting on a failure surface for an earth dam).

For evaluating dynamic earth pressures for this study, we recommend that the value of the horizontal seismic coefficient be taken equal to 65% of the peak ground acceleration and that the vertical seismic coefficient, k_v, be set equal to zero.

In a saturated soil medium the change in water pressure during an earthquake has usually been established on the basis of the method of analysis originally developed by Westergaard (1933). His method of analysis was intended to apply to the hydrodynamic forces acting on the fact of a concrete dam during an earthquake. However, it was used by Matsuo and O'Hara (1960) to determine the dynamic water pressure (due to the pore fluid within the soil) acting on quay walls during earthquakes, and has been used by various other engineers for evaluating dynamic water pressures acting on retaining walls backfilled with saturated soil. Unless the soil is extremely porous, it is difficult to visualize that the pore water can actually move in and out quick enough for it to act independently of the surrounding soil media. For most natural soils, the soil and pore water would move together in phase during the duration of the earthquake such that the dynamic pressure on the wall would be due to the combined effect of the soil and water. Thus, the total weight of the saturated soil should be used in calculating dynamic earth pressure values.

The Allowable Building Code stress increases for seismic loading (33%) translates into an allowable uniform seismic earth pressure on the temporary shoring of about magnitude 6H. This earth pressure corresponds to a seismic coefficient (K_h) of about 0.15g and a peak ground acceleration of about 0.23g (using the recommended procedures). Data from Part I Seismological Investigation indicates the 0.23g peak acceleration to have a probability of exceedance less than 5% during an average two-year period (a reasonable construction period). The average recurrence of this ground motion level was indicated to be about 100 to 150 years. Based on consideration of the above, the 6H uniform seismic pressure was recommended for design of the temporary wall (see Figure 6-5).

E.3 LIQUEFACTION EVALUATION METHODS

E.3.1 Standard Penetration Resistance

The use of the Standard Penetration Test (SPT) in estimating the liquefaction potential of saturated cohesionless soil deposits has been the topic of many previous investigations. Results of these investigations have recently been summarized by Seed et al (1983). Basically, the method utilizes empirical relationships which have been developed from a comprehensive collection of SPT blow count data obtained from sites where liquefaction or no liquefaction was known to have taken place during past earthquakes. Empirical relationships that have been recently proposed by Seed et al. (1983) are shown in Figure E-1.

Corrected SPT "N" values (normalized to 2 ksf overburden pressure for 24 SPT tests in saturated granular alluvium ranged from 11 to 64 with an average of about 32. Determination of dynamic strength was based on an M6.0 for the ODE event and an M7.0 for the MDE event. The liquefaction analysis based on Seed et al (1983) indicated the granular soils could generally withstand the ODE without initial liquefaction. However, the analyses indicated there would be liquefaction of a few granular alluvium layers during the MDE event. Therefore, the granular alluvium is considered to have a low to moderate liquefaction potential during the MDE.

E.3.2 Shear Wave Velocity Measurements

Crosshole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project tunnel alignment were performed as part of the initial 1981 geotechnical investigation. Downhole and crosshole surveys were performed at Boring CEG-28. Average shear wave velocities measured in the Alluvium were about 1000 fps for the crosshole and the downhole measurements.

While shear wave velocity has not been as widely accepted in the past as SPT blow count data for estimating the liquefaction potential of a soil deposit, it has received some recent attention (Seed et al. 1983). Figure E-1 suggests that liquefaction potential at the site would be low based on the shear wave velocities measured.

E.3.3 Gradation/Plasticity Characteristics

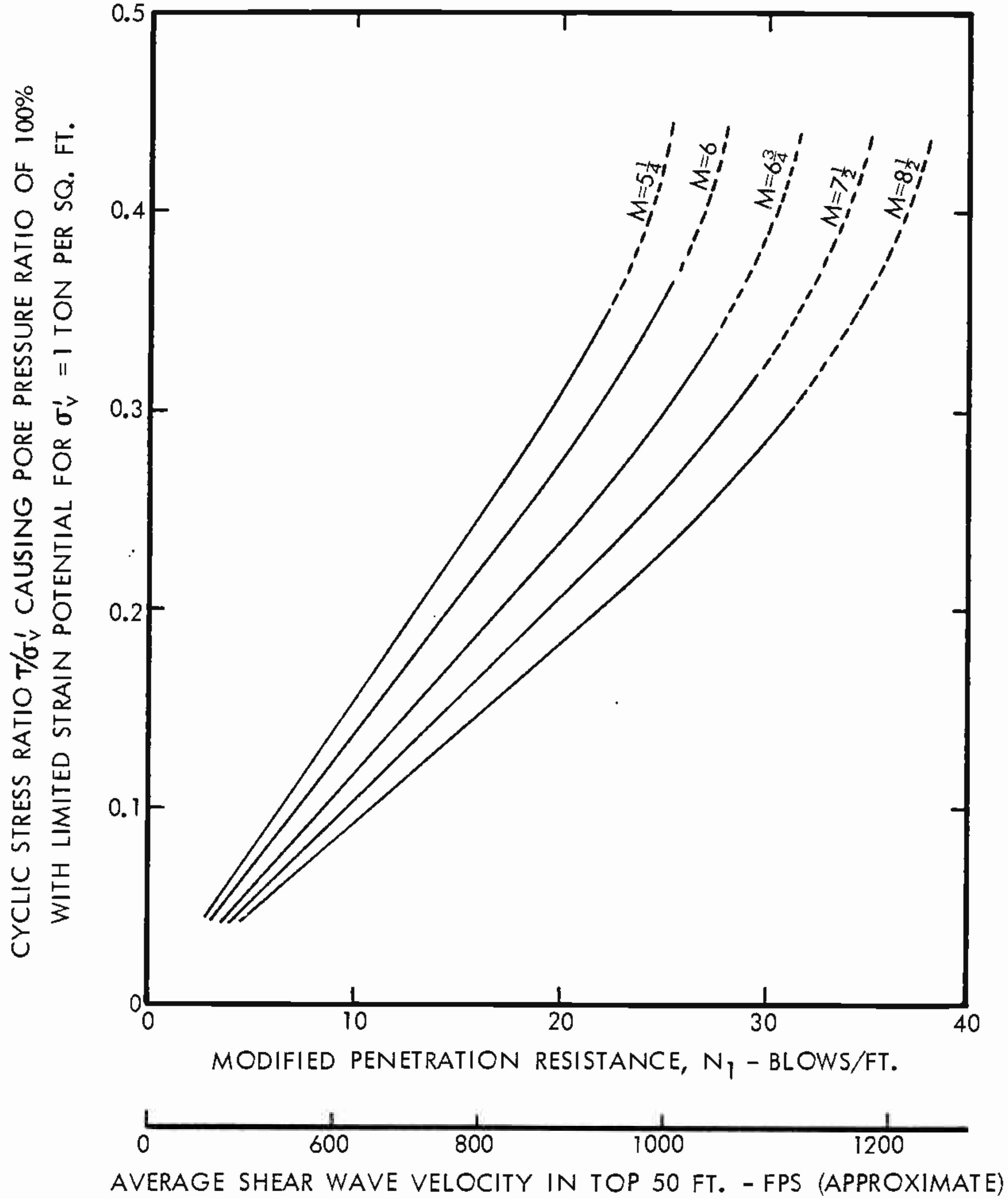
Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figures E-2 and E-3. The ranges shown in these figures have been complied by Lee and Fitton (1968), Seed and Idriss (1967), Kishida (1969), and Youd (1982) and appear to indicate that the soil types most susceptible to liquefaction consist of primarily poorly graded silty sands and sandy silts.

It is important to note that all the gradational ranges shown in Figure E-2 have less than 20% by weight clay size particles (i.e., particles less than 0.005 mm), suggesting that clayey (cohesive) soils have a low liquefaction potential. Seed and Idriss (1983) stated that clayey soils are not vulnerable to significant strength loss during earthquakes if the percentage of particles finer than 0.005 mm is greater than 20 or if the water content is less than 90% of the Liquid Limit. As can be verified by Tables C-1 and C-2 of Appendix C, moisture contents of the clayey soils test are all well below 90% of the Liquid Limit moisture content, thereby indicating the clayey soils to be nonliquefiable.

The gradation characteristics of the various soils which comprise the onsite Alluvium were compiled from laboratory tests performed during this and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" soils shown in Figure E-2 are presented in Figures E-3 and E-4. Several samples tested fall within the range of gradations of soils considered more "susceptible" to liquefaction are shown on Figures E-3 and E-4.

E.3.4 Conclusions

Based on the above considerations and comparisons, it is our judgement that the fine-grained (clayey) alluvial soil deposits would have low liquefaction potential during ground shaking from both the operating design earthquake (ODE) and the maximum design earthquake (MDE). The layers of granular alluvium within the clay soil matrix have a low potential for liquefaction during the ODE; however, localized zones would likely liquefy during the MDE event. In our opinion, liquefaction of the granular alluvium would not result in catastrophic changes in the overall dynamic soil loads on the structure because most of the alluvium is dense and the fine-grained soils are expected to maintain their integrity during the MDE.



(after Seed, 1983)

CORRELATION BETWEEN PENETRATION RESISTANCE AND FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS

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Figure No.
E-1

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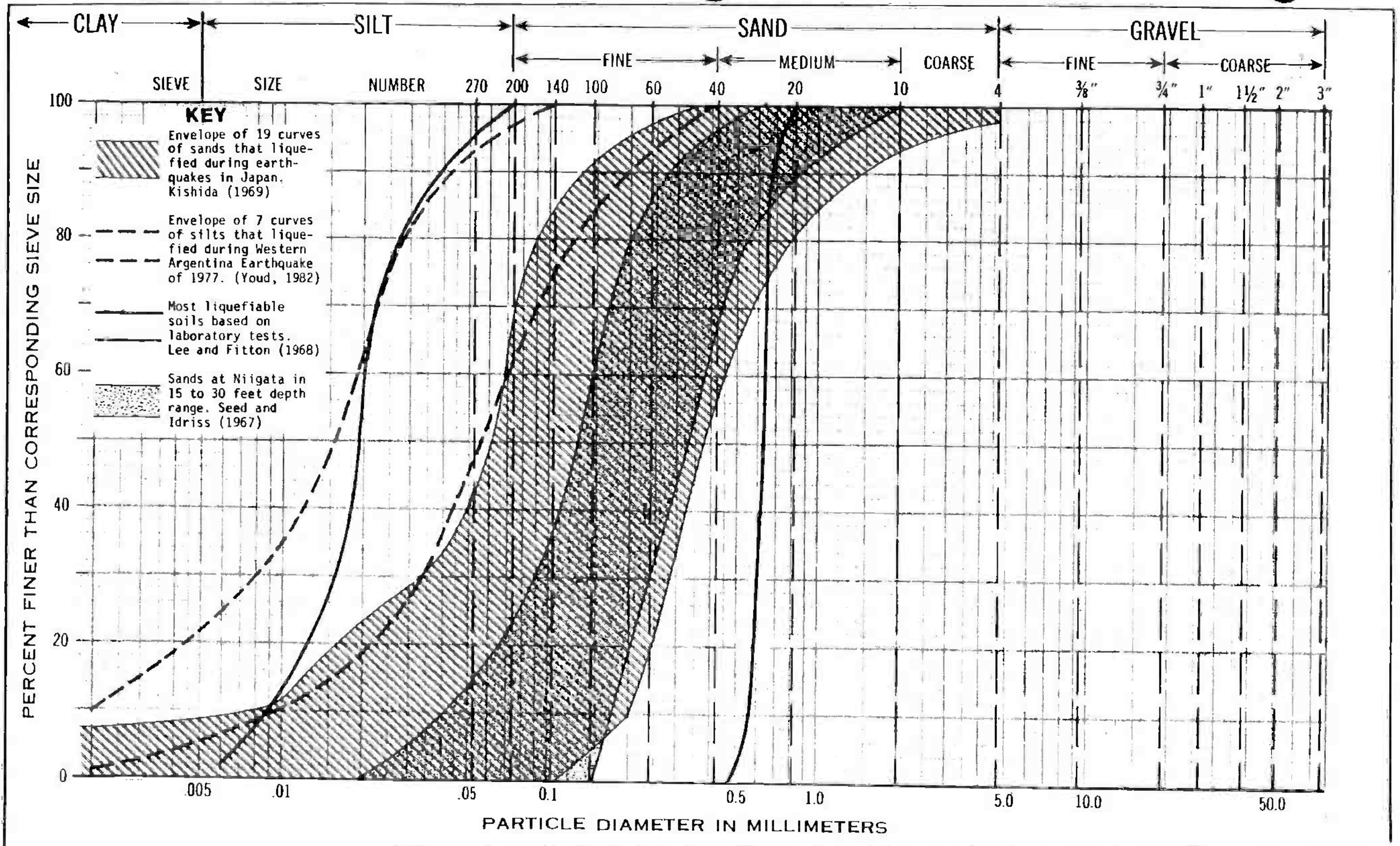

GRADATIONS OF SOILS CONSIDERED SUSCEPTIBLE TO LIQUEFACTION

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Figure No.
E-2

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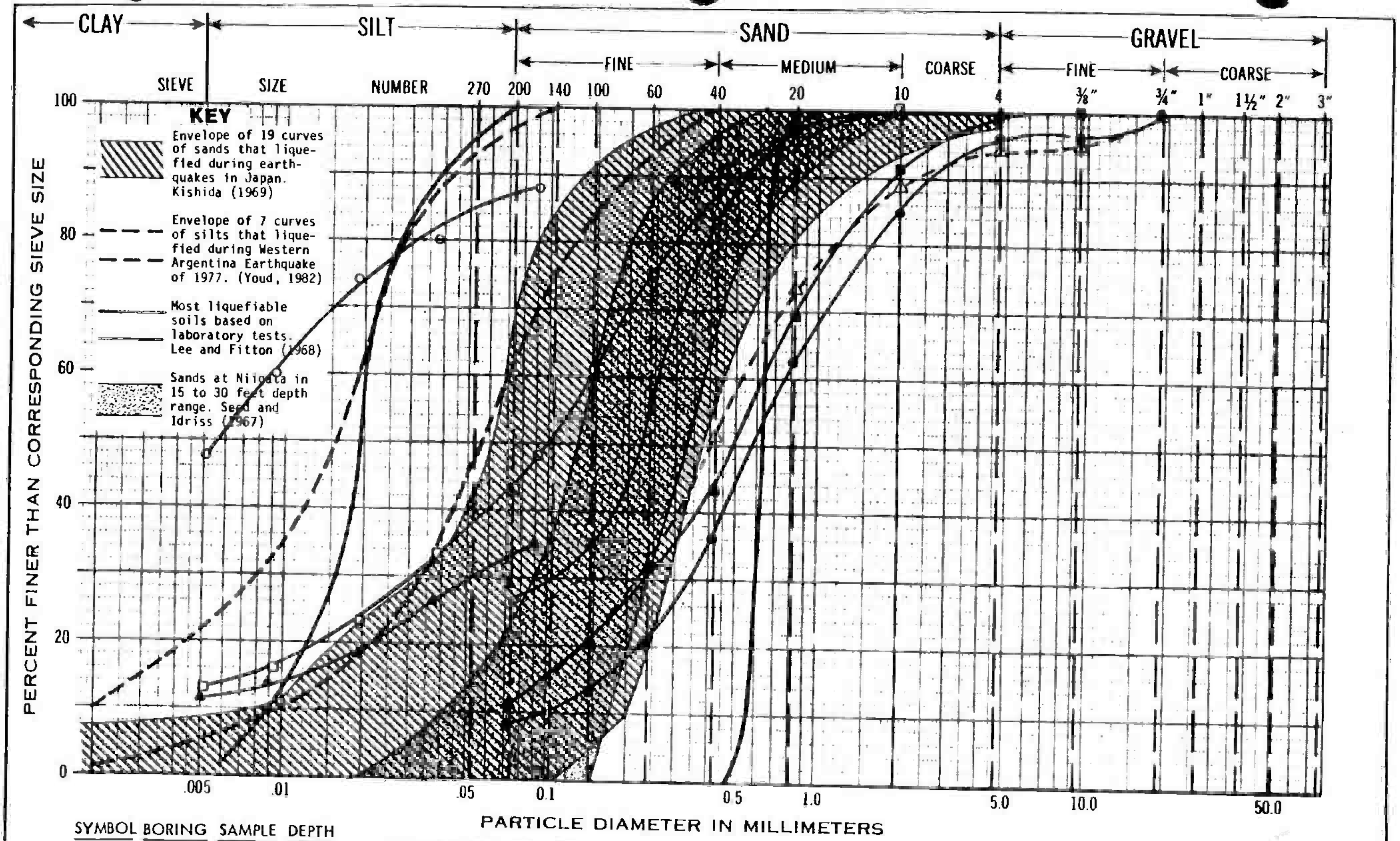

SYMBOL	BORING	SAMPLE	DEPTH
—□—	28/2	C-5	23'
—△—	28/2	C-12	58'
—○—	28/3	C-11	52.5'
—■—	28/3	C-16	76'
—▲—	28/4	C-8	37'
—●—	28/4	C-10	47'
- -△- -	28/4	C-14	67'

COMPARISON OF GRADATIONS

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Figure No
E-3

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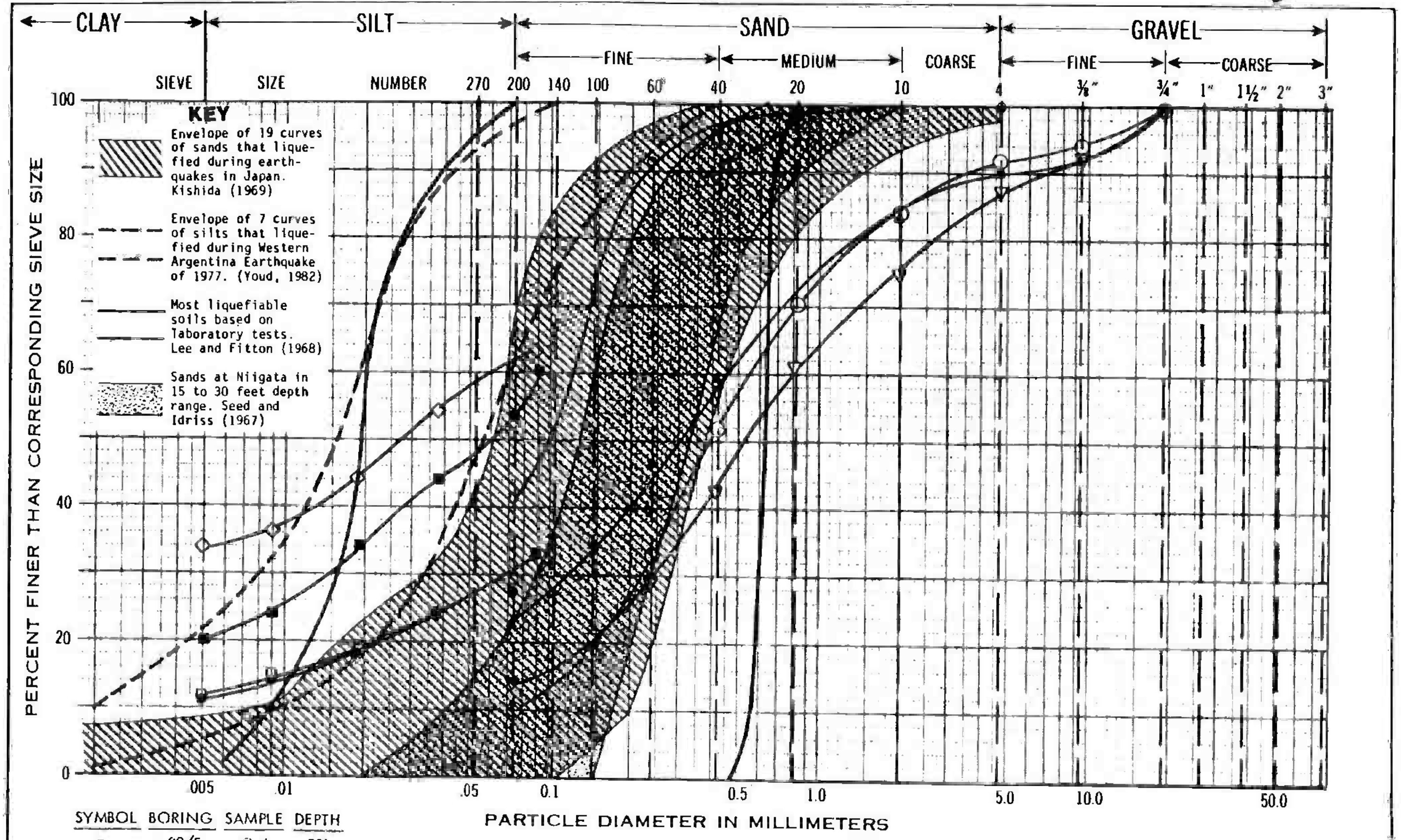

SYMBOL	BORING	SAMPLE	DEPTH
—□—	28/5	C-4	20'
—○—	28/5	C-10	50'
—▲—	28/5	C-15	74'
—■—	28/6	C-5	24'
—●—	28/6	C-7	33'
—▽—	28/6	C-11	53'
—◇—	28/7	C-5	29'
—○—	28/7	C-11	73.5'

COMPARISON OF GRADATIONS

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Figure No.
E-4

Appendix F

Earthwork Recommendations

APPENDIX F EARTHWORK RECOMMENDATIONS

The following guidelines are recommended for earthwork associated with site development. Recommendations for dewatering and major temporary excavations are presented in the text sections 6.2 and 6.4, respectively.

- <u>Site Preparation (surface structures)</u>: Existing vegetation, debris, and soft or loose soils should be stripped from the areas that are to be graded. Soils containing more than 1% by weight of organics may be re-used in planter areas, but should not be used for fill beneath building and paved areas. Organic debris, trash, and rubble should be removed from the site. Subsoil conditions on the site may vary from those encountered in the borings. Therefore, the soils engineer should observe the prepared graded area prior to the placement of fill.

- <u>Minor Construction Excavations</u>: Temporary dry excavations for foundations or utilities may be made vertically to depths up to 5 feet. For deeper dry excavations in existing fill or natural materials up to 15 feet, excavations should be sloped no steeper than 1:1 (horizontal to vertical). Recommendations for major shored excavations are presented in Section 6.4.

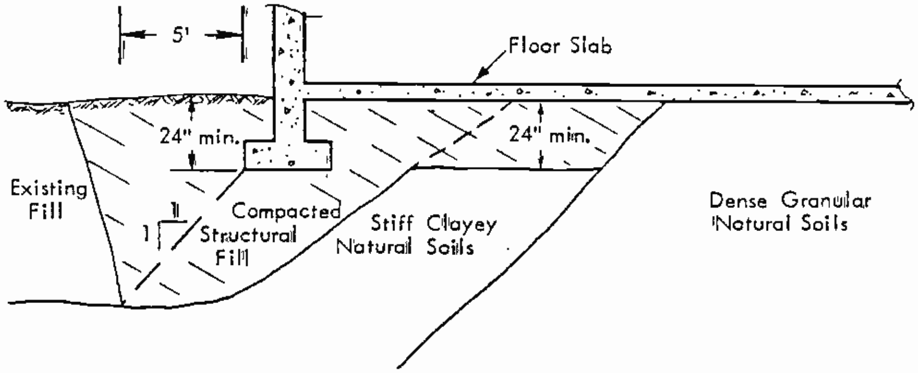
- <u>Structural Fill and Backfill</u>: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed prior to backfilling the walls. Onsite soils or imported granular soils should be placed and compacted in accordance with "Recommended Specifications for Fill Compaction". In deep fill areas or fill areas for support of settlement-sensitive structures, compaction requirements should be increased from the normal 90% to 95% or 100% of the maximum dry density to reduce fill settlement.

  Where space limitations do not allow for conventional backfill compaction operations, special backfill materials and procedures may be required. Sand-cement slurry, pea gravel or other selected backfill can be used in limited space areas. Sand-cement slurry should contain at least 1-1/2 sacks cement per cubic yard. Pea gravel should be placed in a moist condition or should be wetted at the time of placement. Densification should be accomplished by vibratory equipment; e.g., hand-operated mechanical compactor, backhoe mounted hydraulic compactor, or concrete vibrator. Lift thickness should be consistent with the type of compactor used. However, lifts should never exceed 5 feet. A soils engineer experienced in the placement of pea gravel should observe the placement and densification procedures to render an opinion as to the adequate densification of the pea gravel.

  If granular backfill or pea gravel is placed in an area of surface drainage, the backfill should be capped with at least 18 inches of relatively impervious type soil; i.e., silt-clay soils.

- Foundation Preparation: Where foundations for near surface appurtenant structures are underlain by existing fill soils, the existing fill should be excavated and replaced with a zone of properly compacted structural fill. The zone of structural fill should extend to undisturbed dense or stiff natural soils. Horizontal limits of the structural fill zone should extend out from the footing edge a distance equal to 5 feet or 1/2 the depth of the zone beneath the footing (a 1:1 ratio), whichever is larger. The structural fill should be placed and compacted as recommended under "Structural Fill and Backfill".

FOUNDATION/SUBGRADE PREPARATION



- Subgrade Preparation: Concrete slabs-on-grade at the subterranean levels may be supported directly on undisturbed dense materials. The subgrade should be proof rolled to detect soft or disturbed areas, and such areas should be excavated and replaced with structural fill. If existing fill soils are encountered in near surface subgrade areas, these materials should be excavated and replaced with properly compacted granular fill. Where clayey natural soils (near existing grade) are exposed in the subgrade, these soils should be excavated to a depth of 24 inches below the subgrade level and replaced with properly compacted granular fill. Where dense natural granular soils are exposed at slab subgrade, the slab may be supported directly on these soils. All structural fill for support of slabs or mats should be placed and compacted as recommended under "Structural Fill and Backfill".

- Site Drainage: Adequate positive drainage should be provided away from the surface structures to prevent water from ponding and to reduce percolation of water into the subsoils. A desirable slope for surface drainage is 2% in landscaped areas and 1% in paved areas. Planters and landscaped areas adjacent to the surface structures should be designed to minimize water infiltration into the subsoils.

- Utility Trenches: Buried utility conduits should be bedded and backfilled around the conduit in accordance with the project specifications. Where conduit underlies concrete slabs-on-grade and pavement, the remaining trench backfill above the pipe should be placed and compacted in accordance with "Structural Fill and Backfill".

° <u>Recommended Specifications for Fill Compaction</u>: The following specifications are recommended to provide a basis for quality control during the placement of compacted fill.

1. All areas that are to receive compacted fill shall be observed by the soils engineer prior to the placement of fill.

2. Soil surfaces that will receive compacted fill shall be scarified to a depth of at least 6inches. The scarified soil shall be moisture-conditioned to obtain soil moisture near optimum moisture content. The scarified soil shall be compacted to a minimum relative compaction of 90%. Relative compaction is defined as the ratio of the inplace soil density to the maximum dry density as determined by the ASTM D1557-70 compaction test method.

3. Fill shall be placed in controlled layers the thickness of which is compatible with the type of compaction equipment used. The thickness of the compacted fill layer shall not exceed the maximum allowable thickness of 8 inches. Each layer shall be compacted to a minimum relative compaction of 90%. The field density of the compacted soil shall be determined by the ASTM D1556-64 test method or equivalent.

4. Fill soils shall consist of excavated onsite soils essentially cleaned of organic and deleterious material or imported soils approved by the soils engineer. All imported soil shall be granular and non-expansive or of low expansion potential (plasticity index less than 15%). The soils engineer shall evaluate and/or test the import material for its conformance with the specifications prior to its delivery to the site. The contractor shall notify the soils engineer 72 hours prior to importing the fill to the site. Rocks larger than 6 inches in diameter shall not be used unless they are broken down.

5. The soils engineer shall observe the placement of compacted fill and conduct inplace field density tests on the compacted fill to check for adequate moisture content and the required relative compaction. Where less than 90% relative compaction is indicated, additional compactive effort shall be applied and the soil moisture-conditioned as necessary until 90% relative compaction is attained. The contractor shall provide level testing pads for the soils engineer to conduct the field density tests on.