Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A415

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.

General Geotechnical Consultant
Converse Consultants, Inc.
126 West Del Mar Boulevard
Pasadena, California 91105
Telephone 213 795-0461

Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

May 18, 1984

Metro Rail Transit Consultants
548 South Spring Street
Los Angeles, California 90013

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

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A415 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 A415.

Our study team appreciate the assistance provided by the MRTC staff, especially Mr. B.I. Maduke. We also want to acknowledge the efforts of each member of the Converse team, in particular Howard Spellman and Jim Doolittle.

Respectfully submitted,

for Howard A. Spellman
Robert M. Pride, Senior Vice President
Converse Consultants, Inc.

RMP:h

General Geotechnical Consultant
Converse Consultants, Inc.
126 West Del Mar Boulevard
Pasadena, California 91105
Telephone 213 795-0461

PROFESSIONAL CERTIFICATION

REGISTERED PROFESSIONAL ENGINEER
ROBERT M. PRIDE
No. C 21036
CIVIL
STATE OF CALIFORNIA

Robert M Pride

Robert M. Pride
Senior Vice President

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.

REGISTERED GEOLOGIST
HOWARD A. SPELLMAN, JR.
No. 154
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Howard A. Spellman

Howard A. Spellman
Principal Engineering Geologist

Table of Contents

TABLE OF CONTENTS

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation and engineering analyses for the A415 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A415 Design Unit consists of the Hollywood Bowl Station. The structure will be constructed by cut-and-cover methods and will extend to depths of about 55 to 100 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 August 1983.

1.1 CONSTRUCTION CONSIDERATIONS

Excavation for construction will encounter a shallow depth of alluvium underlain by basalt bedrock. Considering the relatively open nature of the site, a sloped excavation may be used within the alluvium. Excavation within the basalt will likely require blasting, and support of vertical rock walls via rock bolts may be needed. Preconstruction dewatering should not be necessary; however, some ground water control may be required during construction.

1.2 DESIGN CONSIDERATIONS

The basalt rock at the subgrade elevation will provide a hard and uniform subgrade for support of the station structure with negligible settlement. Design lateral pressures for the permanent structure are provided in the report. Hydrostatic pressures are also provided for permanent design.

1.3 SEISMIC CONSIDERATIONS

Ground water levels measured at the site were within the bedrock which is not a liquefiable material. Alluvial soils are assumed to be non-saturated at this time because of the low current water levels. However, based on the conservative assumption of future higher water levels, a liquefaction evaluation was performed. The results of the liquefaction evaluation based on field correlations of SPT results and performance of granular soils indicate that the granular soils at the site have a low potential for liquefaction during a maximum design earthquake.

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 A415 are given in this report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for the A415 Design Unit which consists of the Hollywood Bowl Station. 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 A415.

- "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, Sheets 7 and 8, dated July 1983; and Preliminary Site Plans, Plans and Sections for Hollywood Bowl Station, dated August, 1983. The location and depth of the structure as indicated by the referenced plan is shown on Drawings 3 and 4. If the location or configuration of the proposed station is changed from that shown, this report will not be completely applicable to the changed conditions.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

The Hollywood Bowl Station will be located within the major entertainment center area off the Hollywood Bowl Road as shown on Drawings 2 and 3. The area to the north of the Hollywood Bowl Station is Los Angeles County parkland. The surrounding developed areas are of mixed low- to medium-density residential developments.

The station entrance will be located close to the entrance to the Bowl, adjacent to the ticket offices. The station has been designed with a single mezzanine centered on the length of the station. There will be two escalators and two stairs from entry to mezzanine and from mezzanine to platform. Ancillary space will be provided at each end of the station, and a traction power substation will be located below grade over the ancillary space on the outbound end of the station.

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 A415 investigation, a total of 12 borings were drilled at or near the proposed station site. The borings consisted of nine rotary wash holes numbered 29-3, 30-A and 31-1 through 31-7 drilled in 1983 and 1984, and rotary wash Borings CEG-30 and CEG-31 drilled in 1981. In addition, a man-size auger, Boring 30-B, was drilled in 1983. The locations of the borings are shown on Drawings 2 and 3, and the logs of the borings are provided in Appendix A. Ground water observation wells were installed in Borings 30, 30-A, 30-B, 31, 31-1, 31-2, 31-3 and 31-5. Section 5.3 presents a summary of ground water level measurements in these wells.

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). The foundation investigation borings included in the USGS report for this area are not shown on our drawings and were not used because they were too shallow for proper interpretation of subsurface conditions along the proposed grade of the station excavation.

4.3 GEOPHYSICAL MEASUREMENTS

Downhole compression and shear wave velocity surveys were performed in Boring CEG-31 which was drilled during the initial 1981 investigation. The CEG-31 boring was drilled on the south side of the Hollywood Bowl Station. 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 triaxial compression tests, unconfined compression tests, direct shear tests, and permeability tests. Appendix C summarizes the testing procedures and presents detailed results of the testing program.

4.5 WATER QUALITY ANALYSES

Chemical analyses were performed and selected parameters were evaluated on a water sample obtained in Boring CEG-31. The results of these tests are presented in Appendix D.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL DESCRIPTION

During the field program for this investigation and neighboring design units, the contact between Young and Old Alluvium has been difficult to identify since the soils in these two deposits are generally very similar. However, considering the close proximity of the site to the Santa Monica Mountains, it is concluded that the alluvial deposits at the Hollywood Bowl site are relatively young (geologically speaking). Therefore, for purposes of this report, all references to alluvial deposits should be assumed to mean Young Alluvium.

Drawings 2 and 4 show generalized subsurface cross-sections through the proposed Hollywood Bowl Station. The subsurface profile at the Station site consists of predominantly coarse-grained alluvium which extends to depths of up to about 40 feet and overlies basalt bedrock. The alluvium encountered was primarily granular, loose to very dense, consisting of silty sands with traces of clay and fine gravel grading in places to sandy silt with clay. Clayey and silty gravels were also encountered in a number of the borings near the surface of the bedrock. These lenses were up to about 6 feet in thickness, and the gravel was derived from the basalt. The alluvium may also contain zones of cobbles and boulders, although none was encountered.

The bedrock surface slopes downward to the south and east. Depths to bedrock at the Station vary between about 20 feet at the northwest end to about 40 feet at Boring 31-3 near mid-length along the proposed Station structure (see Drawing 4). From Boring 31-3 southward to near the southeast end of the Station, the bedrock surface is generally level at about Elevation 480, and then the rock surface slopes up, rising to the ground surface about 50 feet southeast of the Station.

5.2 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

For purposes of our engineering evaluation, the subsurface materials were grouped into two general subsurface units. These main subsurface units are the predominantly coarse-grained alluvium and the basalt bedrock of the Topanga Formation. Fill soils were not encountered in any of the borings drilled at this location.

The following paragraphs present engineering descriptions of each of the two main subsurface materials and engineering parameters assigned to these units for our analyses (see Table 5-1). The laboratory testing program and laboratory test results are presented in Appendix C.

° Alluvium: The alluvium encountered at this site consisted primarily of silty, fine to medium sand with a trace of clay and fine gravel. However, at Boring 31-3, the material graded to a sandy silt with clay. The gravel content of the alluvium tends to increase within about 10 feet of the bedrock surface. Standard Penetration Test (SPT) results ranged from 5 to 77 in this soil unit but averaged approximately 33. Laboratory density tests carried out on samples from this unit generally indicated dry densities ranging between 95 and 105 pcf. Triaxial test results indicated effective stress friction angles of 32° to 33° and total stress

TABLE 5-1

MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	GEOLOGIC UNIT	
	ALLUVIUM	TOPANGA BEDROCK
Moist Density Above Ground Water (pcf)	120	155
Saturated Density (pcf)	125	155
Effective Stress Strength		
ϕ' (degrees)	33	-
c' (psf)	200	-
Total Stress Strength(a)		
ϕ (degrees)	25	-
c (psf)	500	-
Average Unconfined Compressive Strength (ksf)	-	100 to 400(b) 400 to 1,500(c)
Permeability (cm/sec)	10^{-2} to 10^{-4}	-
Poisson's Ratio (non-saturated)	0.35	0.35
Initial Vertical Tangent Modulus (psf)	260 $\sigma_{v'}$(d)	-

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

(b) Intensely to moderately fractured.

(c) Moderately to little fractured.

(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.

friction angles of 24° to 25°. Direct shear test results were generally higher, with friction angles ranging from 29° to 45°. The strength values of Table 5-1 are somewhat lower than may be expected for "granular" alluvium due to the high silt content of the soils at this site. Undrained modulus values from triaxial tests exhibited a moderate increase with consolidation pressure. Permeability tests performed on silty sand specimens from this unit indicated permeabilities on the order of 10^{-3} to 10^{-4} cm/sec; however, the permeability of the more gravelly soils generally present at the base of the unit is considered to be 10^{-2} to 10^{-3} cm/sec.

- Topanga Formation Bedrock: All the borings drilled at the site penetrated to varying depths into basalt bedrock of the Topanga Formation. Soft and deeply to moderately weathered basalt was encountered to depths of a few feet to 10 feet immediately below the alluvium. Below this depth, the basalt is "fresher", moderately hard to hard, moderately strong to strong, moderately to little weathered, intensely to little fractured material. The fractures are weakly to strongly cemented. The "fresher" bedrock rings with a hammer blow; a dull thud occurs from a hammer blow on weathered bedrock.

There are significant variations in Rock Quality Designations (RQD) at each boring. RQD, as defined in this report, is the percentage of core 4 inches or longer obtained from a coring run. About 95% of the recorded core breaks are due to weakly cemented natural fractures; the balance are mechanical breaks. Table 5-2 shows RQD variations in basalt at various depths.

In general, RQD is poor to fair at the south end of the station (Boring 31-2), and good to excellent near the north end of the station (Boring 31-5). We believe the RQD is better at the north end of the station because this end is farther from the Hollywood Bowl fault zone located just south of the site. Mountain building (uplift) forces that created the Santa Monica Mountains also contributed to discontinuities (fractures) in the basalt as evidenced by numerous weakly cemented, slickensided fracture surfaces in Borings 31-2 through 31-5.

Basalt in Borings 31-2 through 31-4 is intensely fractured (spaced 0.05 foot to 0.1 foot apart) to moderately fractured (spaced 0.5 foot to 1.0 foot apart). Basalt in 3-5 is moderately fractured to little fractured (spaced 1.0 foot to 3.0 feet apart). Fractures nearer the Hollywood Bowl fault zone are weakly cemented with soft, secondary minerals; for example, chlorite, talc, calcareous clay. Thus, cores break more easily, and the RQD is generally poor to fair. Fractures in Boring 31-5, farthest from the Hollywood Bowl fault zone, are more strongly cemented, predominantly with quartz. Thus cores do not break readily, and the RQD is generally good.

If compared to rock tunnelling conditions, we believe the following Terzaghi Rock Condition Numbers, as shown on Drawing 5, would apply to the basalt:

Borings 31-2, 31-3 and 31-4: Terzaghi No. 4 (moderately blocky and seamy)
Terzaghi No. 5 (very blocky and seamy)
Boring 31-5: Terzaghi No. 3 (massive, moderately jointed)
Terzaghi No. 4 (moderately blocky and seamy)

TABLE 5-2

RQD VARIATIONS - BASALT

BORING No.	RQD (%)	APPROXIMATE DEPTH INTERVAL (ft)
31-2	96	35 - 37
	41	37 - 42
	79	42 - 47
	0	47 - 51
Station Grade →	38	51 - 56
	28	56 - 59
	32	59 - 63
	77	63 - 70
	43	70 - 76
	75	76 - 82
	34	82 - 85
	19	85 - 90

BORING No.	RQD (%)	APPROXIMATE DEPTH INTERVAL (ft)
31-3	17	46 - 51
	68	51 - 56
	63	56 - 59
	68	59 - 64
	100	64 - 69
Station Grade →	83	69 - 74
	60	74 - 79
	92	79 - 84
	45	84 - 89
	65	89 - 90
	84	90 - 95
	96	95 - 100

BORING No.	RQD (%)	APPROXIMATE DEPTH INTERVAL (ft)
31-4	76	21 - 26
	84	26 - 32
	67	32 - 36
	58	36 - 41
	27	41 - 46
	33	46 - 51
	61	51 - 56
	60	56 - 61
	60	61 - 66
	69	66 - 71
	84	71 - 76
	54	76 - 81
	57	81 - 86
	75	86 - 91
Station Grade →	90	91 - 100

BORING No.	RQD (%)	APPROXIMATE DEPTH INTERVAL (ft)
31-5	96	13 - 17
	100	17 - 27
	100	27 - 37
	98	37 - 45
	95	45 - 55
	79	55 - 65
	100	65 - 75
	95	75 - 80
	62	80 - 84
	82	84 - 88
	65	88 - 96
	93	96 - 105
	80	105 - 113
Tunnel →	98	113 - 121
	87	121 - 131
	97	131 - 150

RQD AS RELATED TO ROCK-MASS PROPERTIES:

RQD (%)	DESCRIPTION	APPROXIMATE EQUIVALENT FRACTURE SPACING
0- 25	Very poor	Intensely
25- 30	Poor	Closely
30- 75	Fair	Moderately
75- 90	Good	Little
90-100	Excellent	Massive

Laboratory testing of the bedrock for this study has generally been limited to unconfined compression tests performed during this investigation and the 1981 investigation. The material properties presented in Table 5-1 are, therefore, based primarily on published data and limited laboratory test results for basalt.

5.3 GROUND WATER

Ground water levels in the vicinity of the station were measured in piezometers installed at Borings 30, 30-B, 31, 31-1 and 31-5. Table 5-3 presents ground water levels measured in these piezometers. Based on the results of these measurements, it appears that current ground water levels are within the bedrock and slope in a southward direction across the site at gradients of about 5% to10%, which is approximately the same as the average ground surface gradients. Drawings 2 and 4 show that current water levels range from about Elevation 515 feet at the northwest end of the site to about Elevation 475 at the southeast end of the Station site. Table 5-3 shows ground water levels measured in the piezometers have remained relatively constant, but no piezometer readings to date have been obtained during heavy rainfall periods.

No gas odors or unusual ground water conditions were noted during the field exploration program in the site area.

TABLE 5-3

GROUND WATER OBSERVATION WELL DATA

	GROUND WATER ELEVATION*								
	Initial		1981	1982	1983			1984	
BORING	Elevation	Date	06/17	04/28	02/24	10/24	12/20	02/13	05/08
30	454	03/03/81	456	453					455
30-A	-	02/22/83							
30-B	442	02/23/83			445				
31	453	02/24/81		456				457	458
31-1	453	10/06/83				453	453	453	454
31-2	486	10/24/83							
31-3	470	10/09/83							
31-5	530	10/19/83				531	531	533	531

*Rounded to the nearest foot

Section 6.0

Geotechnical Evaluation and Design Criteria for Stations

6.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA FOR STATIONS

6.1 GENERAL EVALUATION

Construction of the Hollywood Bowl Station will involve excavation of alluvial soils and hard basalt rock to depths of 55 to 100 feet below the existing ground surface. Excavation within the alluvium may be shored or, considering the relatively open nature of the site, a sloped excavation may be used. Excavation of the basalt rock will likely require blasting. Shoring by means of rock bolts and wire mesh within the basalt may be required. Current ground water levels measured at the site are within the basalt and, therefore, preconstruction dewatering should not be required; however, some ground water control may be required during construction.

The basalt rock at the subgrade elevation will provide hard and uniform subgrade support for the proposed Station with negligible differential settlement. Lateral loads on the permanent structure will vary between that of the alluvial soils and that of the basalt. Seismic performance of the basalt will be governed by the local structure of the formation. The dynamic performance of this formation is discussed in the Southern California Rapid Transit District (SCRTD) 1984 report entitled "Guidelines for Design of Underground Structures".

The following subsections present more detailed evaluations and recommendations for design and construction of the Hollywood Bowl Station.

6.2 EXCAVATION DEWATERING

6.2.1 General

Considering that current ground water levels at the site are within the basalt bedrock below the alluvial soils, preconstruction dewatering in the alluvium is not anticipated. Current ground water levels within the basalt range from 25 to 65 feet above the proposed subgrade (see Drawings 2 and 4). The basalt encountered at the boring locations was intensely to little fractured (fracture spacing of 1/2 inch to 12 inches), but most of the fractures were well healed with hard mineral deposits. Some gravity seepage from the bedrock is expected as the excavation proceeds, but flow quantities should generally be moderate to low. Based on our current data, dewatering at this site could he handled by a drain and sump system within the excavation. Localized zones of high flow may be encountered, but these are expected to be of limited extent and would likely drain within a short period of time. Blasting may cause minor changes in ground water inflow.

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. It is anticipated that a dewatering system will consist of drains and sumps at the base of the excavation. The system should satisfy at least the following criteria:

- ° The sump system should have the capability of quickly increasing pumping capacity to handle zones of high flow.

- The sump system should operate continuously.
- The sump system should include emergency power and backup pumps in case of power or equipment failure.
- Disposal of pumped water must be in accordance with all local ordinances.

6.2.3 Induced Subsidence

Due to the fact that current ground water levels are entirely within the relatively incompressible basalt rock, no significant subsidence due to dewatering is anticipated.

6.3 UNDERPINNING CONSIDERATIONS

Figure 6-1 presents general underpinning guidelines for building structures based on the depth of alluvium and the expected zone of influence from a shored excavation. Since the nearest significant structure (The Bowl) is more than 170 feet from the proposed Station, it is expected that the "Underpinning Report" for this Station will indicate that underpinning is not required. Therefore, no further discussion or recommendations for underpinning are presented herein.

6.4 .TEMPORARY EXCAVATIONS

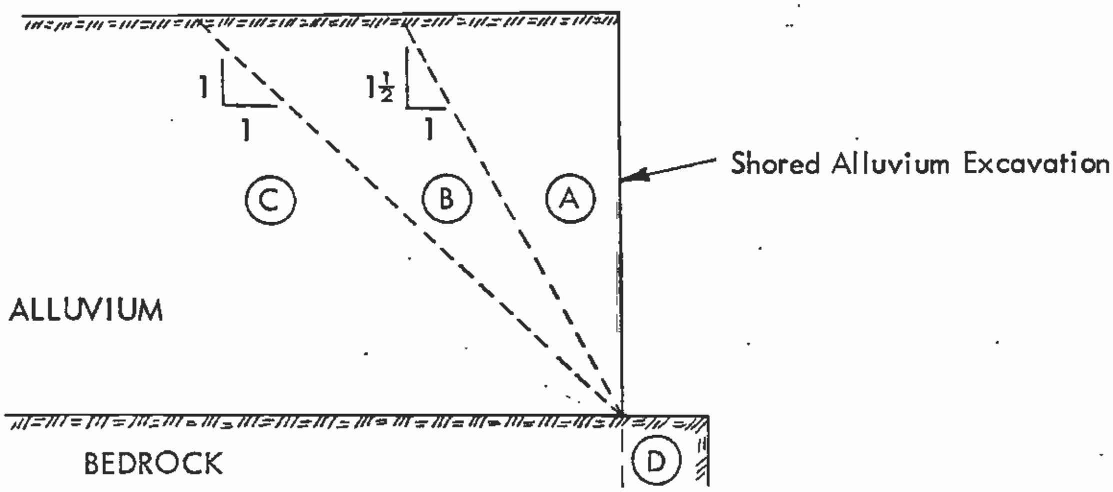
6.4.1 General

The required A415 Station excavation will extend some 55 to 100 feet below the existing ground surface through alluvium and basalt rock. Alluvial soils are expected to range between 15 and 40 feet in thickness, and the top of basalt 30 to 80 feet above the proposed subgrade. Ground water levels are below the top of the basalt and, therefore, will not affect the excavation of the alluvium. 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.

6.4.2 Excavation within the Alluvium

6.4.2.1 General: Considering the current site conditions, excavation within the alluvium at the site could be either sloped or shored. A sloped excavation is considered feasible due to the absence of structures around the site and the relatively thin alluvium. A shoring method considered feasible would be soldier piles with lagging. Bracing systems are probably limited to tiebacks in soil, rock bolts in rock, and internal bracing.

The following subsections present further evaluations and our recommendations for both sloped and shored excavations within the alluvium.



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.

Ⓓ Area of required reinforcement to maintain lateral and vertical stability of shoring system supporting the alluvium and applicability of these guidelines (See Figure 6-3).

Approved for publication 5/84 by FAD

UNDERPINNING GUIDELINES - ADJACENT TO SHORING

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
6-1

Converse Consultants Geotechnical Engineering and Applied Sciences

6.4.2.2 Sloped Excavation in Alluvium: Safe, stable construction slopes are the contractor's responsibility (refer to CALOSHA Article 6 1540d) and must be determined in the field based on actual construction conditions. However, for construction feasibility purposes, we have evaluated possible excavation slopes. Based on the field and laboratory investigation, it is our opinion temporary excavation slopes in natural alluvium may be made at slope ratios given in the table below:

SLOPE HEIGHT (ft)	MAXIMUM SLOPE RATIO* (horizontal:vertical)
<5	vertical
5 to 20	1:1
20 to 40	1 1/4: 1

* Slope ratio is assumed to be uniform from the top to toe of slope.

It should be noted that the recommended slope ratios are for uniform, dry (dewatered) slopes without surcharge; composite slopes or other conditions would require special evaluation. A setback (bench) should be provided between the toe of slope and the bedrock cut. The minimum setback should be 5 feet or 1/2 of the slope height, whichever is greater (see Figure 6-3b).

Field observations indicate that alluvial thickness on the existing slopes north of the site is very small. Therefore, it appears feasible to excavate the relatively minor amount of soil between the Station and toe of existing slope to expose the basalt bedrock and eliminate any alluvial slopes in those areas. If alluvial soils are removed along the east and north sides of the excavation, some provisions may be required to intercept surface drainage from the existing slopes along those sides.

6.4.2.3 Shored Excavations in Alluvium: Shoring for support of the alluvium should be "toed" into the bedrock. This would require excavating a nominal distance into rock which may be very difficult.

Driven sheet pile shoring does not appear feasible at this site due to the expected difficulty in penetrating the basalt to provide a "toe-in" at the top of the basalt.

A soldier pile system is considered feasible for alluvium support, but the piles will encounter problems penetrating the basalt. The need for a stiff shoring system (such as a slurry wall) does not appear to exist at this site since no structures are within the zone of influence normally considered with such excavations.

Internal bracing would appear to be preferable over tiebacks from the installation standpoint due to the difficulty of penetrating the basalt. Consideration may be given to a combination of tieback support in the upper portion of the shoring and internal bracing in the lower portion.

Considering the above-discussed items and local construction practice, we feel that a conventional soldier pile and lagging shoring system with tiebacks in the alluvium 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 may be considered by the contractor, and further recommendations can be provided for such designs as required.

6.4.2.4 Alluvium Shoring Design Criteria: This section provides design criteria for a conventional soldier pile shoring system 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 loads, the piles were assumed to have circular concrete sections.

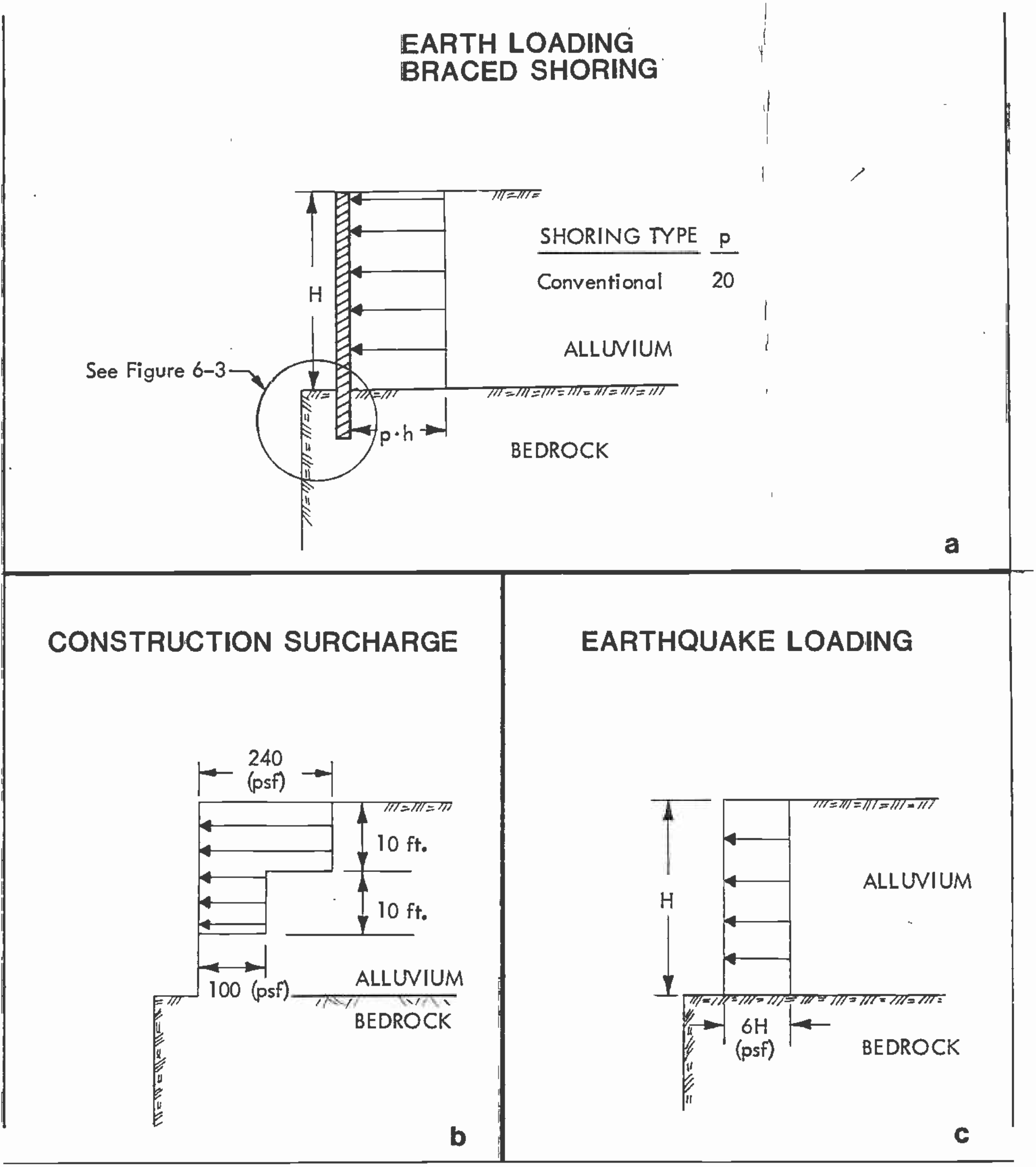
All shored soils at the site will require support between soldier piles to reduce loss of ground. Typically, wooden lagging is used although precast concrete or steel panels could also be used.

Specific shoring design criteria include:

° Design Wall Pressure: Figure 6-2a presents the recommended lateral earth pressure for temporary braced shoring walls within the alluvium. Construction surcharge pressures for a conventional shoring system are presented in Figured 6-2b. Appendix E.2 provides technical support for the recommended seismic pressures of Figure 6-2c. The full loading diagram above the bottom of the alluvium should be used to determine the design loads on tieback anchors. 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.

° Pile Placement and Embedment: Soldier piles for support of alluvial soils should be placed back 5 feet or more from the edge of the planned bedrock cut line. All shoring piles must penetrate through any deeply weathered basalt and at least 5 feet into the fresher basalt (see Figure 6-3a). The allowable vertical bearing capacity for properly placed and embedded soldier piles is 30 ksf. No passive resistance should be assumed for the bedrock. All lateral resistance for the shoring should be provided entirely by the bracing system. A brace, internal struts or long rock bolts extending well into fresh basalt rock should be provided at the bedrock/alluvium interface.

° Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. At the A415 site the alluvial soils encountered were generally silty sand and sandy silt and would be subject to ravelling and sloughing. Thus, it is recommended that the pile spacing be limited to about 8 feet and that continuous lagging



Approved for publication 5/84 by

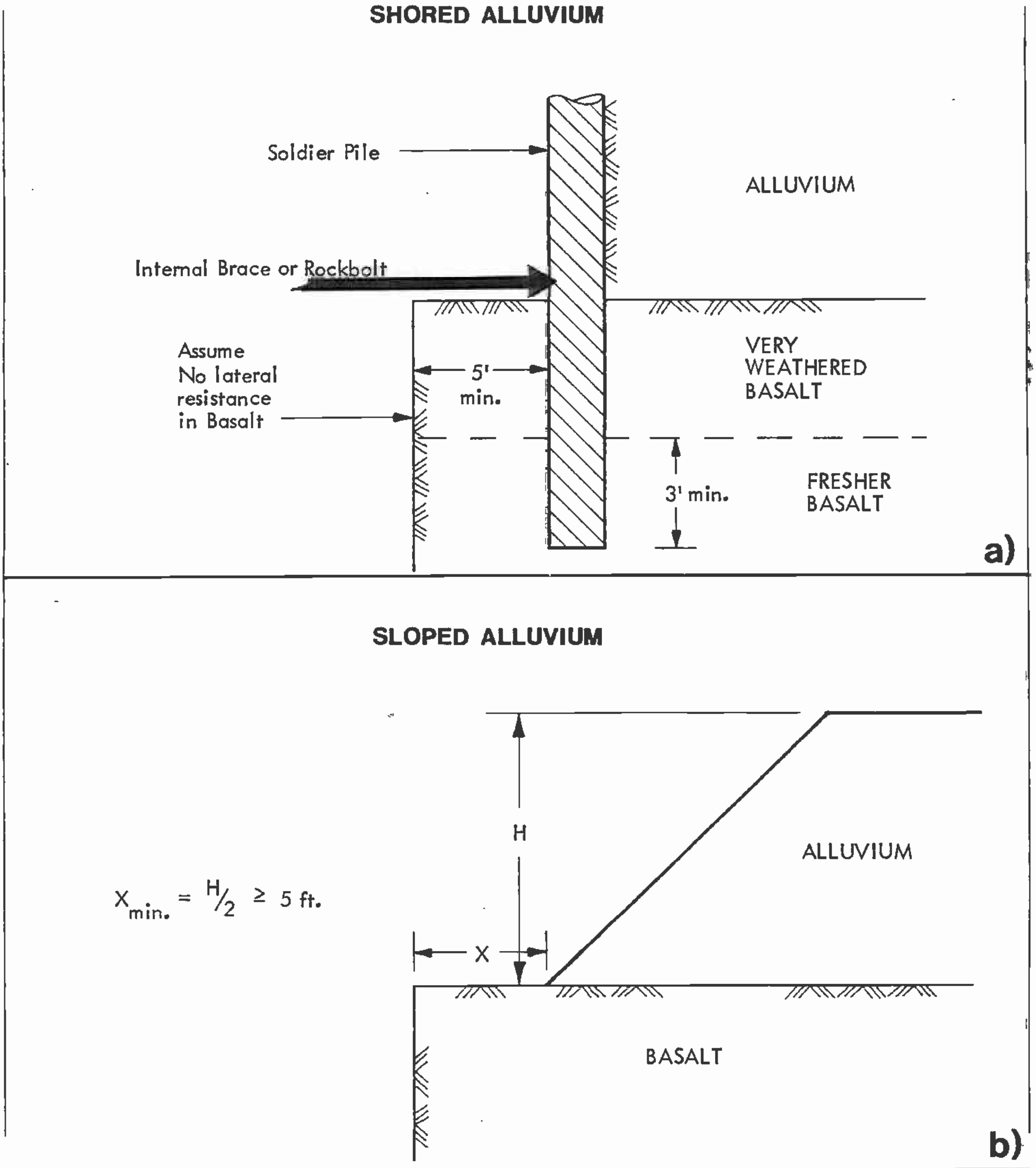
LATERAL LOADS ON TEMPORARY SHORING IN ALLUVIUM

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
6-2

Converse Consultants Geotechnical Engineering and Applied Sciences



Approved for publication 5/1/84 by

EXCAVATION DESIGN AT ALLUVIUM/BEDROCK CONTACT

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
6-3

be placed to minimize ravelling of soils and loss of ground between soldier piles. The contractor should limit the temporarily exposed granular soil height to less than 3 feet to control ravelling problems.

- Excavation Stability: 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 through the alluvium or weathered bedrock horizons.

6.4.2.5 Internal Bracing: The contractor should not be allowed to extend the excavation through the alluvium 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 12 feet as the maximum allowable vertical distance between struts.

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.

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.

6.4.2.6 Tieback Anchors: Numerous types of tieback anchors may be constructed, such as 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 alluvial soils which are stable and dewatered.

Actual tieback anchor capacity can only be determined in the field based on anchor load tests. For estimating purposes, we recommend that the capacity of drilled straight shaft friction anchors in alluvial soils at this site 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:

$$q = 20d < 750 \text{ psf (in dewatered alluvium)}$$

Where:

d = average depth of the anchor in feet beyond the no-load zone; measured vertically from the ground surface.

Figure 6-4 illustrates the anchor design parameters

Allowable anchor capacity/length relationships for tieback types other than straight shaft friction anchors cannot be generalized. Design parameters for 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.

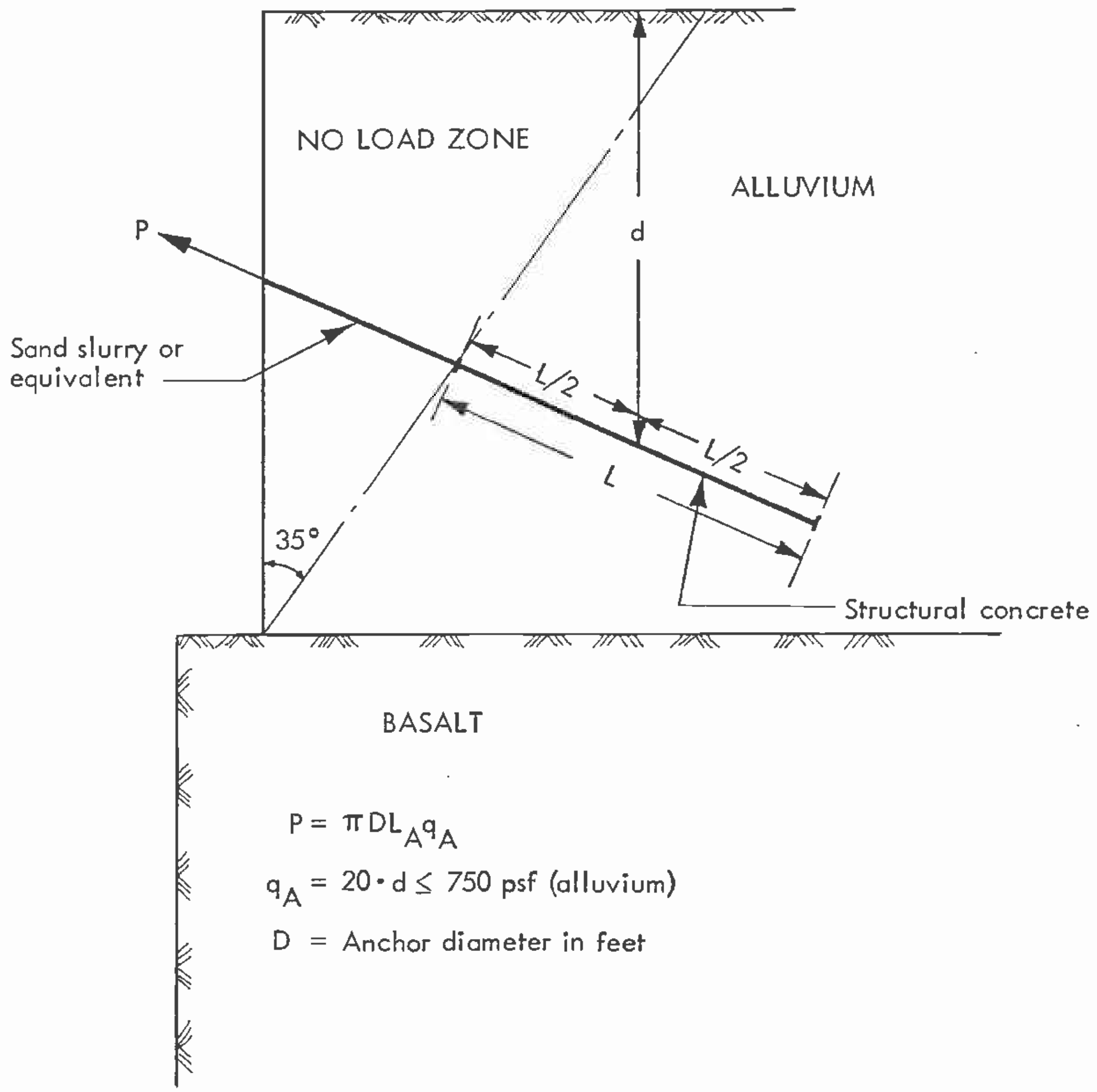
For design purposes, it should be assumed that the potential wedge of failure behind the shored excavation is determined by a plane drawn at 35° with the vertical from the bottom of the alluvium. Only the frictional resistance developed beyond this no-load zone should be assumed effective in resisting lateral loads.

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.

6.4.3 Excavation within the Bedrock

Compressional wave velocities measured in Boring 31 (basalt) were about 9000 feet per second (Appendix B, Table B-1). This boring is near the Hollywood Bowl fault, where the basalt would be expected to be more fractured than the basalt to the north. The RQD in basalt cores is quite variable, both laterally and with depth (See Table 5-2). Based on limited geophysical data and RQD values, it is our opinion that excavation of basalt bedrock, in the main, will require blasting. However, because of numerous intervals of poor to fair RQD values caused by weakly cemented fractures, there should be several horizons that are rippable with very heavy duty grading equipment. The rock hardness horizons will vary laterally as well as with depth.

The excavation within the bedrock may be made with vertical sidewalls; however, a bench should be provided around the excavation perimeter to provide a setback from alluvium slopes and shoring (see Figure 6-3). Continuous observation by an engineering geologist should be made during bedrock excavation operations to observe the evaluate the exposed fracture patterns,



$$P = \pi D L_A q_A$$

$$q_A = 20 \cdot d \leq 750 \text{ psf (alluvium)}$$

D = Anchor diameter in feet

TIEBACK ANCHOR SCHEMATIC

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
6-4

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 5/8/84 by

evaluate the required rock bolt patterns and ground water seepage with respect to sidewall stability. Some spalling and pop-out failures should be expected due to stress relief within the bedrock. Rockbolt anchors and wire mesh should be used as required to support the sidewalls. A possible rockbolt system might consist of rockbolt anchors 10 feet in length spaced 10 feet on center. Final recommendations for rockbolt spacing and penetration should be provided by the field engineering geologist based on actual observed rock conditions during construction. Disturbance of the bedrock perimeter to a depth of 10 feet below the soldier piles should be prevented by rock bolting or other appropriate method.

6.5 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed A415 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. Instrumentation is also important due to the likelihood of blasting at this site.

We recommend the following instrumentation program be considered:

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

- Surface Survey Control: It is recommended that several locations around the excavations be surveyed prior to any construction activity and then periodically to monitor potential vertical and horizontal movement to the nearest 0.01 feet. If shoring is used, survey markers should be placed at the top of soldier piles spaced no more than every fourth pile or 25 feet, whichever is less. Survey markers should also be established at the perimeter bedrock surface at 25-foot intervals.

- Blast Monitoring: We recommend that the effects of blasting be monitored both within the excavation and at the location of adjacent structures. Monitoring within the excavation should include measurements of acceleration and displacements at the excavation walls as well as careful inspection of the walls after blasting to detect any weakened areas which might become unstable. Monitoring outside the excavation should include measurements of acceleration, velocity and displacement at the locations of significant nearby structures, especially the bowl structure.

- 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 survey information.

- Measurement of Strut Loads: Where 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 changes 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 before and after any blasting and 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 blasting or 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 an experienced instrumentation 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.

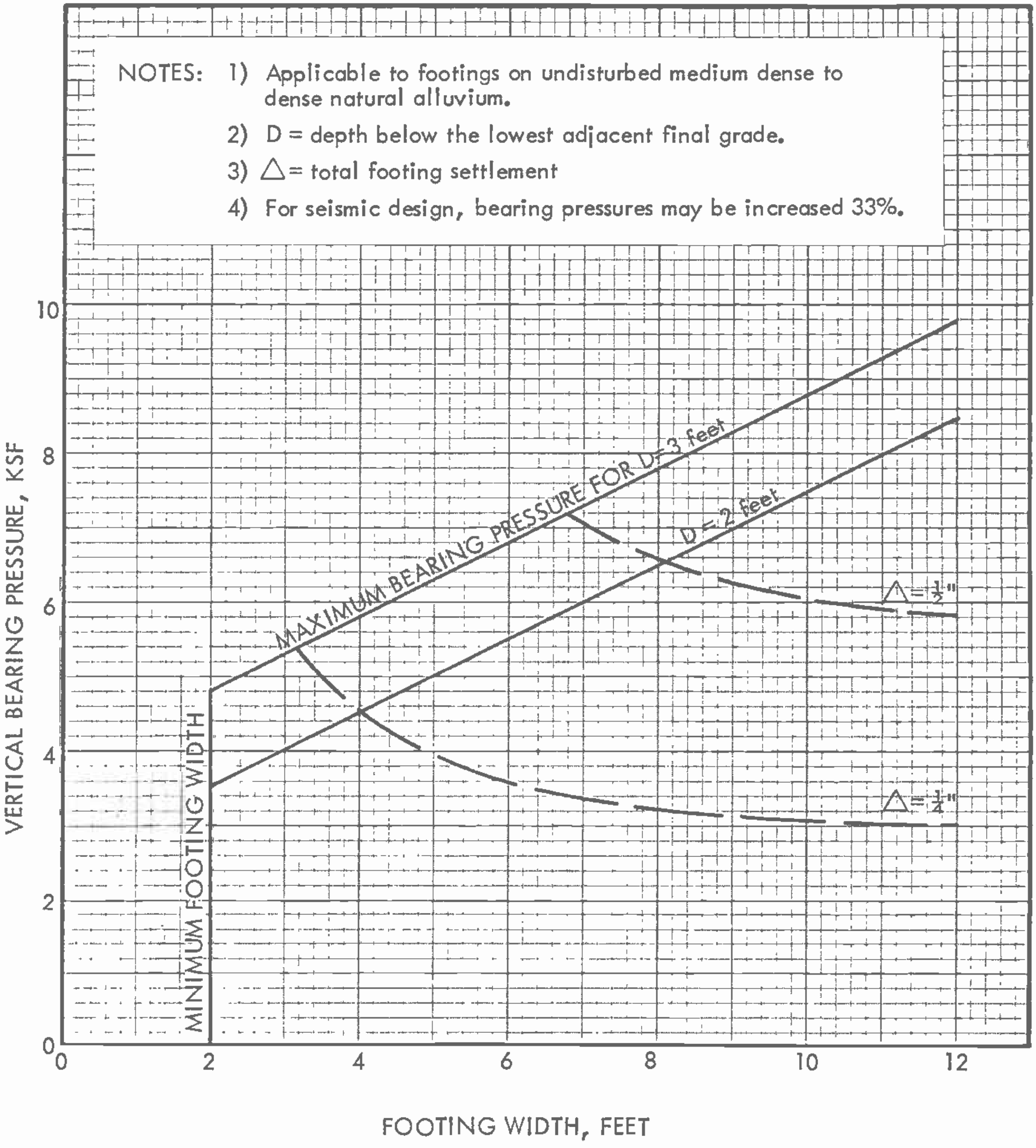
6.6 FOUNDATION SYSTEMS

6.6.1 Main Stations

It is understood that the proposed Hollywood Bowl Station will be supported on a thick base slab which will function as a rigid mat foundation. We estimate that the net mat foundation bearing pressures may range from about 1900 to 3300 psf, depending upon ground water conditions. In our opinion the Station can be adequately supported by the mat foundation on the bedrock. Total and differential settlements across the structure will be negligible due to the hard nature of the basalt subgrade.

6.6.2 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 6-5 and 6-6. These figures are based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed



NOTES:
1) Applicable to footings on undisturbed medium dense to dense natural alluvium.
2) D = depth below the lowest adjacent final grade.
3) △ = total footing settlement
4) For seismic design, bearing pressures may be increased 33%.

SPREAD FOOTING BEARING/SETTLEMENT ON ALLUVIUM

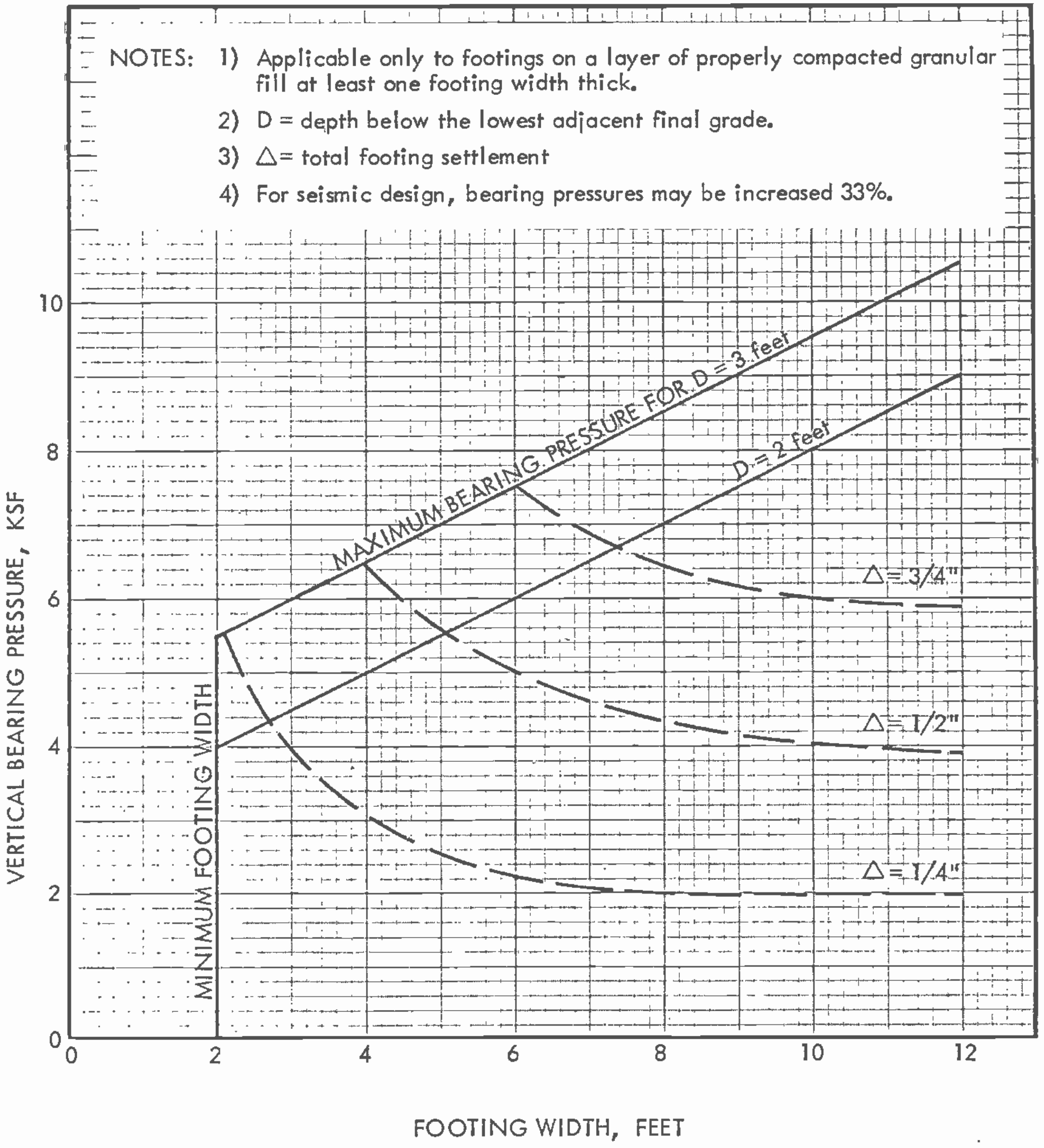
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
6-5

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 5/84 by JKD



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

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
6-6

Approved for publication 5/84 by FAD

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 6-5 and 6-6 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 6-5 and 6-6, 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 0.4 with dead load forces.

6.7 PERMANENT GROUND WATER PROVISIONS

We understand that the Station will be designed to be water-tight and to resist the full permanent hydrostatic pressures. We recommend that full waterproofing be carried at least 5 feet above the recommended maximum ground water levels given in Section 6.8.

6.8 LOADS ON SLAB AND WALLS

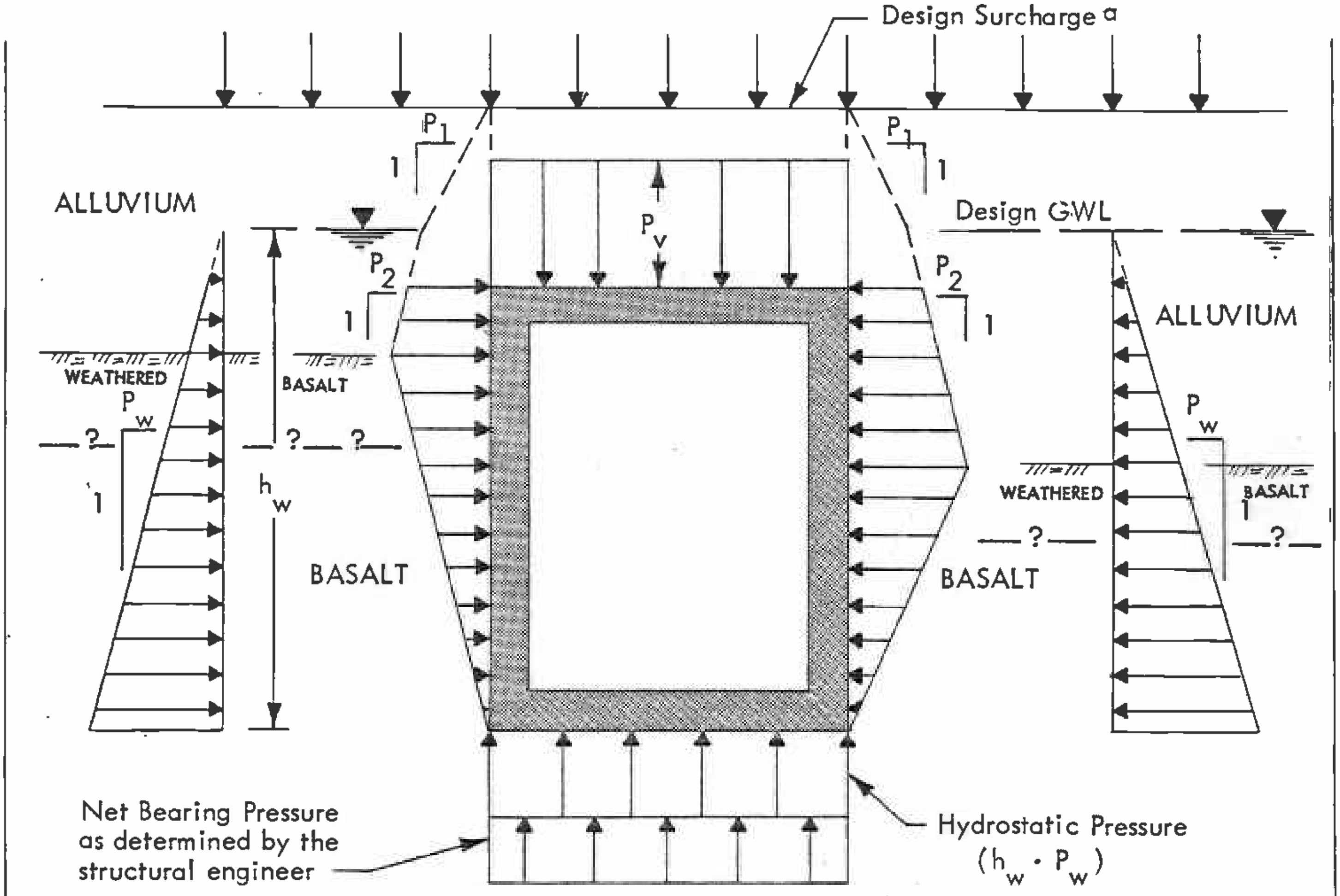
6.8.1 Hydrostatic Pressures

As discussed in Section 5.3, the existing ground water levels are expected to range from about Elevation 475 at the southeast end of the Station to about Elevation 515 at the northwest end of the Station. Considering the relatively narrow width of the alluvial valley in which the site is located, significant fluctuations of ground water levels are assumed possible during periods of heavy rainfall. For purposes of this Design Summary, we will assume conservative water levels could occur at least temporarily. It is recommended that the following ground water levels be used for determining hydrostatic pressures:

LOCATION	ELEVATION
Northwest end of Station	530
Southeast end of Station	490

6.8.2 Permanent Static Lateral Pressures

Figure 6-7 presents average lateral pressure diagrams recommended for design of permanent subsurface walls. Lateral pressures within the bedrock were



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	35	20	62.4	a	b
Long Term	55	30	62.4	a	c
Side sway d	35/55	20/30	62.4	a	b

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

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

c Varies linearly from elev. 490 at the southeast end to elev. 535 at the northwest end.

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

e Pressures in the basalt assume that the basalt is properly supported by rockbolting during excavation.

LOADS ON PERMANENT WALLS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
6-7

Approved for publication 5/84 by

conservatively assumed to equal the lateral soil pressure at the bedrock surface, decreasing to zero at the base of the structure. Due to the possibility of adverse joint/fracture patterns in the bedrock, points of stress concentration on subsurface walls formed directly against the rock surface could develop over the long term or due to dynamic loadings. Placement of a soil backfill between the station walls and the bedrock surface would reduce the effects of stress concentrations.

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

6.8.3 Surcharge Loads

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.

6.9 SEISMIC CONSIDERATIONS

6.9.1 General

Evaluation of the seismological conditions which may impact the project and the probable and maximum 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. Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD 1984 report entitled "Guidelines fcr Seismic Design of Underground Structures". The 1984 report complements and supplements the 1983 report and includes discussions of the seismic performance of the bedrock.

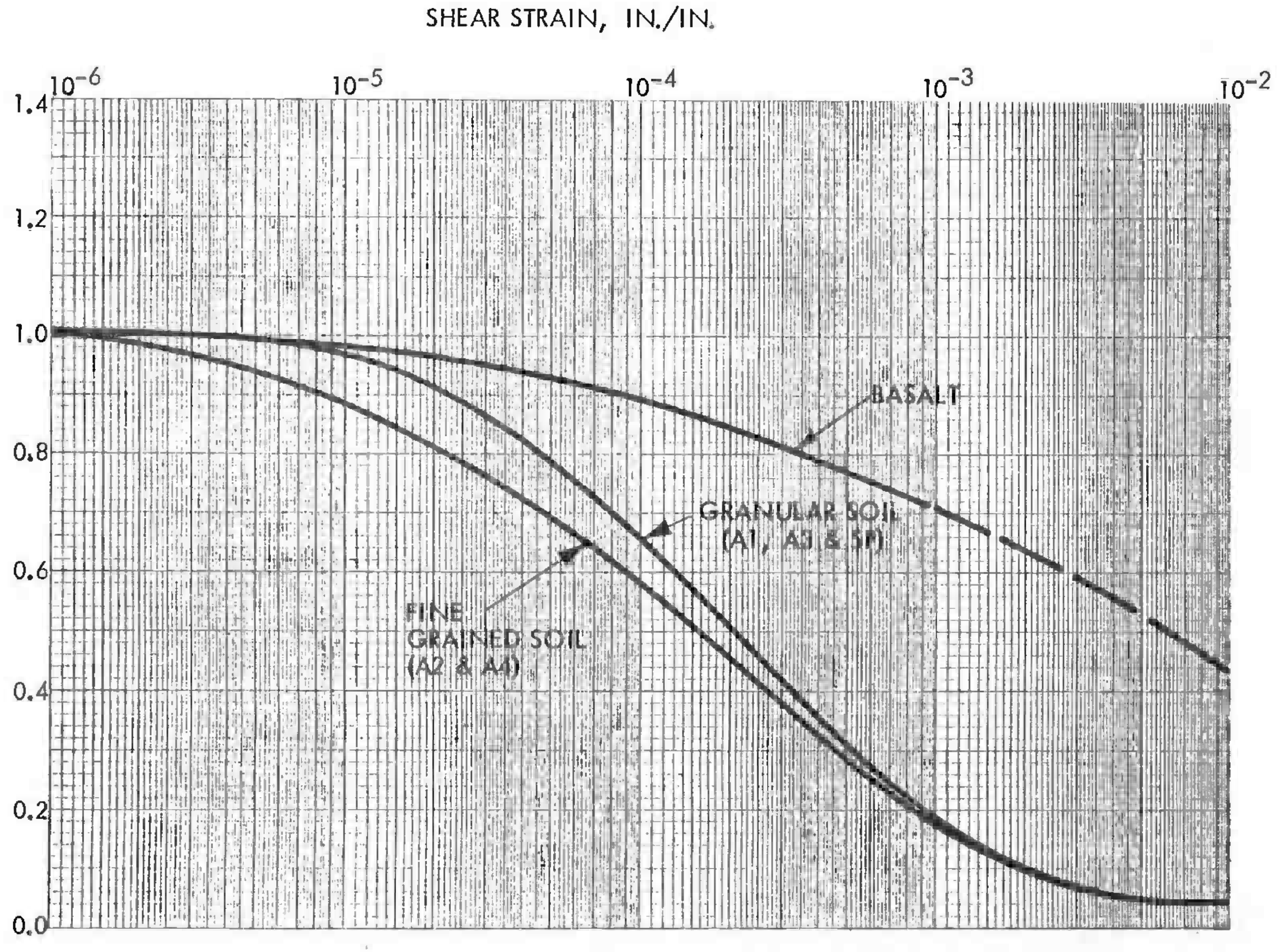
6.9.2 Dynamic Material Properties

Dynamic soil parameters will be required for input into the various types of design dynamic analyses. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels. In addition, certain types of equivalent linear analyses require the variation of dynamic shear modulus and soil hysteretic damping with shear strain.

Average values of compression and shear wave velocities based on interpretation of limited downhole geophysical surveys performed in Boring CEG-28 and CEG-34 during the 1981 investigation are presented at the top of Table 6-1. These velocities have been used together with corresponding values of density and Poisson's ratio to establish appropriate modulus values at low strain levels. Computed moduli values for the alluvium and bedrock are tabulated in Table 6-1.

The variation of dynamic shear modulus, with shear strain is presented in Figure 6-8 for alluvial soils and basalt. 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 6-9. The modulus and damping curves for soil

Approved for publication 5/84 by



RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. 6-8

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Approved for publication 5/84 by JAB

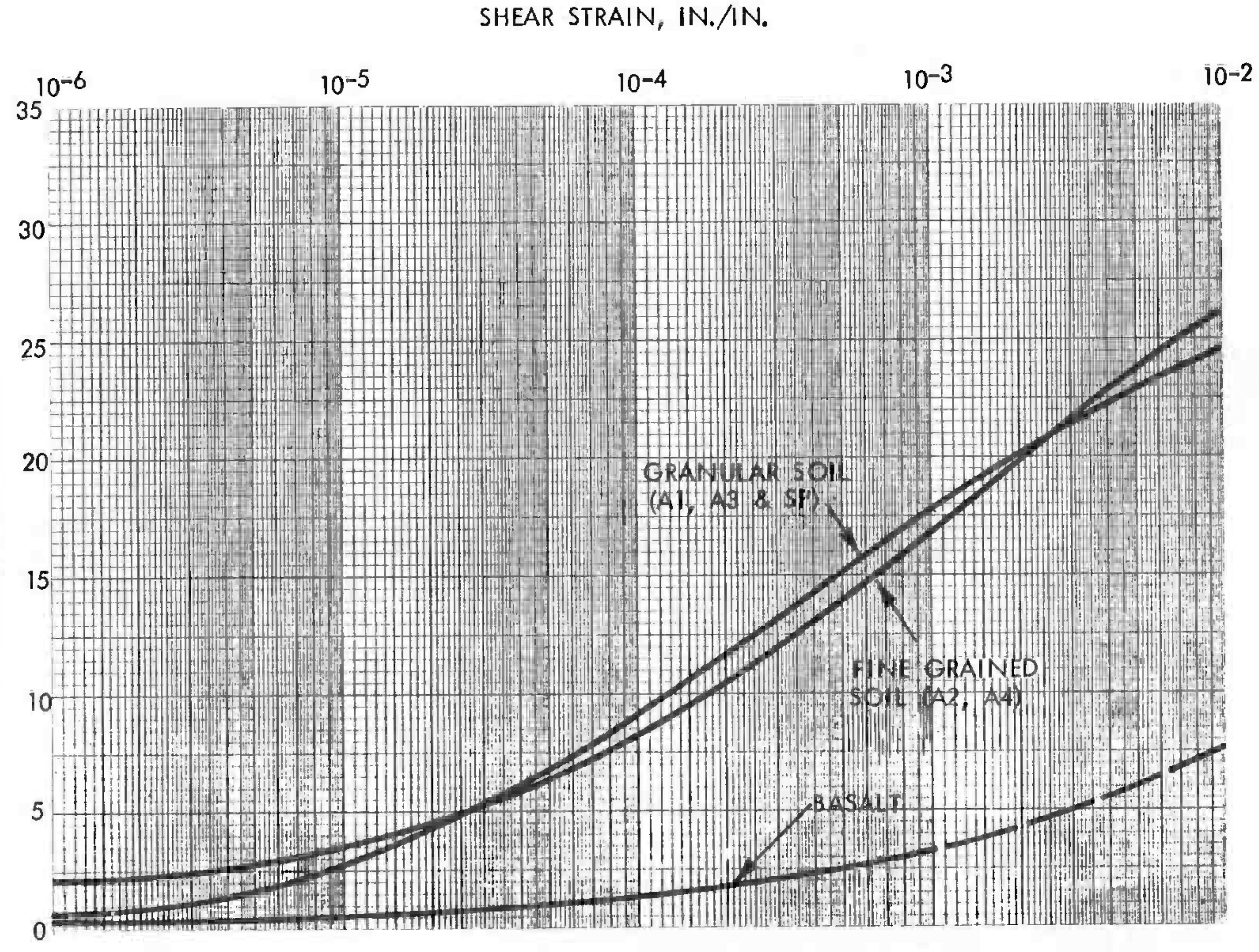
Converse Consultants
Geotechnical Engineering and Applied Sciences

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. 6-9



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. Modulus and damping curves for the basalt were based on published data by Schnabel, Seed and Lysmer (1972)

TABLE 6-1

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		YOUNG ALLUVIUM (A_1)	BEDROCK (Tb)
Average Compression Wave Velocity, V_c (ft/sec)	- moist	2000	9000
	- saturated	5000	9000
Average Shear Wave Velocity, Vs (ft/sec)		760	5000
Poisson's Ratio		*0.35	0.35
**Young's Modulus, E, (psi)	- moist	65,000	1,690,000
	- saturated		1,690,000
**Constrained Modulus, E_c, (psi)	- moist	104,000	2,700,000
	- saturated		2,700,000
**Shear Modulus, G_{max}, (psi)		15,000	835,000

* For saturated condition, use value of 0.45.

** Saturated values of modulus should be used for undrained loading conditions.

6.9.3 Liquefaction Potential

Generalized subsurface cross-sections are shown on Drawings 2 and 4. Measured ground water levels in the site area (as shown on Drawings 2 and 4) were within the bedrock and, therefore, the alluvial soils are assumed to be non-saturated. This is assumed to represent the norm at this site and, therefore, there would be no potential for liquefaction at this site under normal conditions.

The higher ground water levels recommended for permanent static design would indicate the lower portions of the alluvium to be saturated; however, this condition is conservative and has not been verified by existing piezometer data. In addition, it is anticipated that higher water levels would occur for only a short period during heavy rainfall. Notwithstanding these points, a simplified liquefaction evaluation was performed assuming a ground water level about 15 feet below the existing ground surface.

Liquefaction evaluation procedures were based 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 geotechnical investigation were used for our evaluation of the liquefaction potential of the alluvial soils.

Our analysis of the SPT data was performed in accordance with the simplified procedures of Seed et al (1983). Corrected "N" values (normalized to 2 ksf overburden pressure) for five SPT tests in silty sand soils from Borings 31-2 and 31-4 ranged from a minimum of 31 to a maximum exceeding 75+, with an average of about 50+. Corrected "N" values recorded in Borings 31-3 and 31-6 were only 11 and 21 feet, respectively. Determination of dynamic strength was based on an M7.0 earthquake event. Results of the analyses indicated that, where corrected "N" values equaled 30 or greater, the soils would not liquefy during the maximum design earthquake (MDE).

Considering the results of the SPT analyses combined with the conservative assumption of high ground water levels and the expected short duration of same, it is our conclusion that the site would not be subject to significant liquefaction during the maximum design earthquake unless the unfavorable assumptions occurred at the same time.

6.10 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, and backfill for subterranean walls and footings and utility trenches. Recommendations for dewatering and major temporary excavations 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 are presented in Appendix F. Recommended specifications for compaction of fill are also presented in Appendix F. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand) is 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 are not considered suitable because these fine-grained materials will make compaction difficult and could lead to fill settlement problems after construction. If granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the geotechnical engineer. Excavated bedrock materials are expected to consist of cobble- and boulder-sized angular fragments and would require special processing and compaction procedures to place properly.

6.11 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 any 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. A qualified engineering geologist should also be onsite full-time during excavation of the basalt. The engineering geologist should provide recommendations for supporting the rock walls as the excavation proceeds.

References

REFERENCES

CHAPMAN, K., Cording, E.J., and Schnabel, H., Jr., 1972, Performance of a Braced Excavation in Granular and Cohesive Soils: ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue University, Vol. III, pp. 271-293.

CLOUGH, G.W., 1980, Slurry walls for underground tram facilities: U.S. Department of Transportation Report FHWA-TS-80-221.

CLOUGH, G.W., Buchignani, A.L., 1981, Slurry walls in San Francisco Bay area: ASCE National Conference, New York, p. 81-142.

CORDING, E.J., and O'Rourke, T.D., 1977, Excavation, ground movements, and their influence on buildings: American Soc. of Civil Engineers, Preprint, San Francisco.

CORPS OF ENGINEERS, 1953, Technical Memorandum No. 3-357, March, 1953.

CRANDALL, L.R., and Maljian, P.A., 1977, Use of earth anchors to restrict ground movements: Am. Soc. Civil Engineers, Preprint 2974, p. 1-27.

DEPARTMENT OF THE INTERIOR, Bureau of Reclamation, 1963, Earth Manual.

EVANS, L.T., 1968, Swell and settlement study Equitable Life Building, Los Angeles, California: Report by L.T. Evans, Inc.

GOLDBERG, D.T., Jaworski, W.E., and Gordon, M.D., 1976, Lateral Support Systems and Underpinning: Federal Highway Administration, Offices of Research & Development, Vols. I, II, III.

HARDIN, B.O., 1970, Suggested Methods of test for shear modulus and damping of soils by resonant column: ASTM Special Technical Publication 479.

KISHIDA, H.J. 1969, Characteristics of liquified sand during Mino-Owari Tohnakai and Fukui earthquakes: Soils and Foundations, Japan, Vol. 9, No. 1, March, p. 79-92.

LEE, K.L., and Fitton, J.A., 1968, Factors affecting the cyclic loading strength of soil, vibration effects of earthquakes in soils and foundations: American Society for Testing and Materials, Special Technical Publication 450.

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT, 1976, Hydrologic Report 1974-1975.

MALJIAN, P.A., and Van Beveren, J.F., 1974, Tied-back deep excavations in Los Angeles area: Journal of Constr. Div., ASCE, Vol. 100 CO3, p. 337-356.

MANA, A.I., Clough, G.W., 1981, Prediction of movements for braced cuts in clay: ASCE Geotechnical Journal, June.

MATSUO, H., and O'Hara, S., 1960, Lateral earth pressures and stability of quay walls during earthquakes: Proceedings of Second World Conference on Earthquake Engineering, Tokyo, Japan.

MONOBE, N., and Matsuo, H., 1929, On the determination of earth pressures during earthquakes: Proceedings, World Engineering Conference, Vol. 9, p. 176.

NAVFAC, 1971, Design Manual 7-Soil mechanics, foundations, and earth structures: Department of the Navy, Naval Facilities Engineering Command.

NAVFAC, 1982, Design Manual 7.1-Soil mechanics: Department of the Navy, Naval Facilities Engineering Command, May.

NELSON, J.C., 1973, Earth tiebacks support excavation 112 feet deep in Civil Engineering: Am. Soc. Civil Engineers, Nov. 1973, p. 41-44.

OKABE, S., 1926, General theory of earth pressure: Journal of Japanese Society of Civil Engineers, Vol. 12, No. 1.

PRAKASH, S., 1981, Soil dynamics: McGraw-Hill, New York.

SCHNABEL, P., Seed, H.B., and Lysmer, J., 1972, Modification of seismograph records for effects of local soil conditions: Conference on Earthquake-Resistant Design of Engineering Structures, U.C. Berkeley, June 1972.

SCHULTZ, M.S., 1981, An empirical investigation into the behavior of diaphragm walls, Masters Thesis, Massachusetts Institute of Technology.

SEED, H.B., Idriss, I., Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data: ASCE Journal of Geotechnical Division, Vol. 109, No.3, March 1983, p. 458-482.

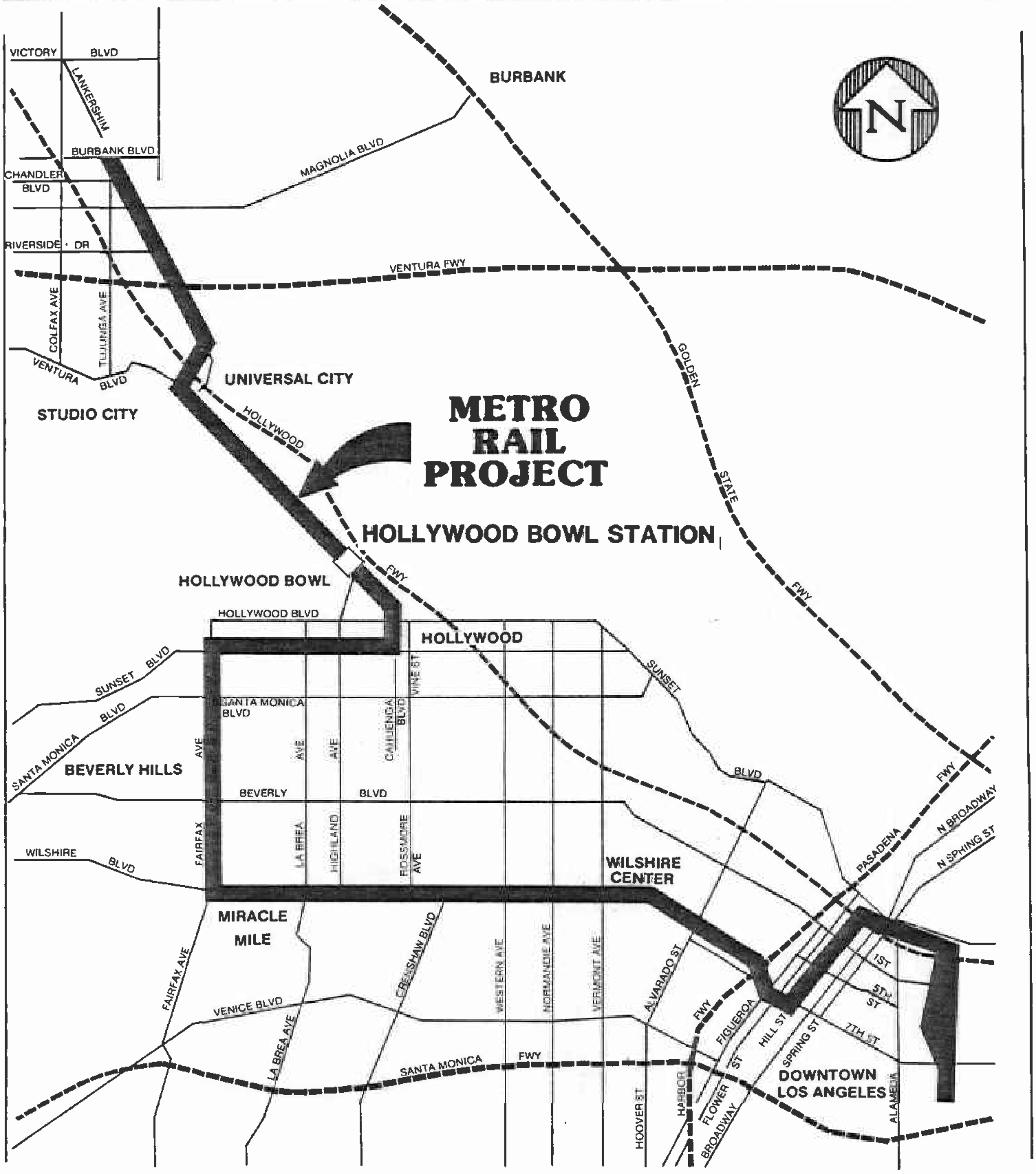
SEED, H.B, and Whitman, R.V., 1970, Design of earth retaining structures for dynamic loads in Ground and Design of Earth Retaining Structures: ASCE Specialty Conference on Lateral Stresses New York, p. 103-148.

SEED, H.B. and Idriss, I.M., 1967, Analysis of soil liquefaction: Niigata Earthquake Journal of the Soil Mechanics and Foundations Division, ASCE Vol. 93, No. SM3, Proceedings Paper 5233, May, p. 83-108.

WESTERGAARD, H.N., 1933, Water pressures on dams during earthquakes: Transactions, ASCE, p. 418-433.

YOUD, L.T., 1982, U.S. Geological Survey, Menlo Park; personal communication.

Drawings



VICINITY MAP

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

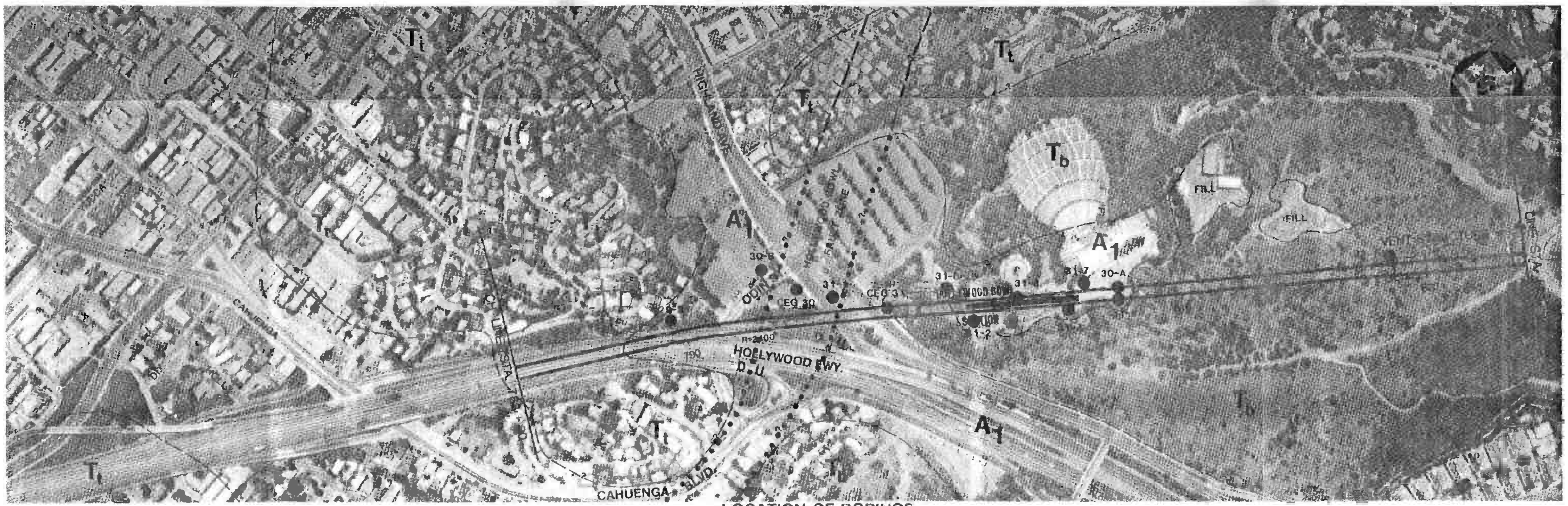
Project No.
83-1140

Drawing No.
1

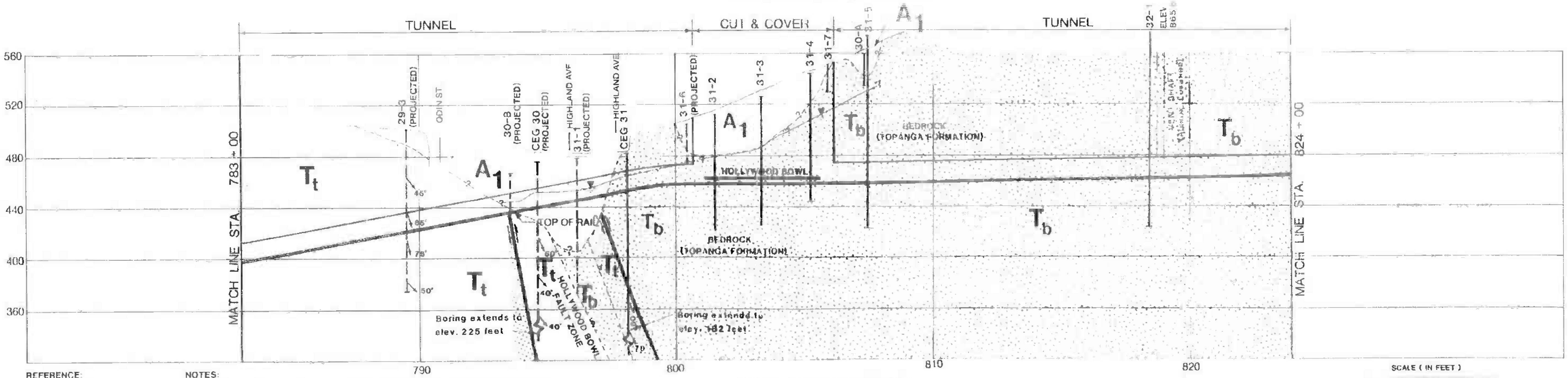
Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 5/84 by

0-10



LOCATION OF BORINGS



GEOLOGIC SECTION

REFERENCE:
MILESTONE 10 SHEET 16 OF 2[illegible] ALIGNMENT PLAN & PROFILE STATION 783+00 TO STATION 824+00 DATED MARCH 1983

NOTES:

1.) LOCATION AND GRADE OF TUNNEL AND STATION SUBJECT TO CHANGE.

2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 1

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

SCALE (IN FEET)
VERT. 0 20 40 60 80
HORIZ. 0 200 400

PRELIMINARY

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

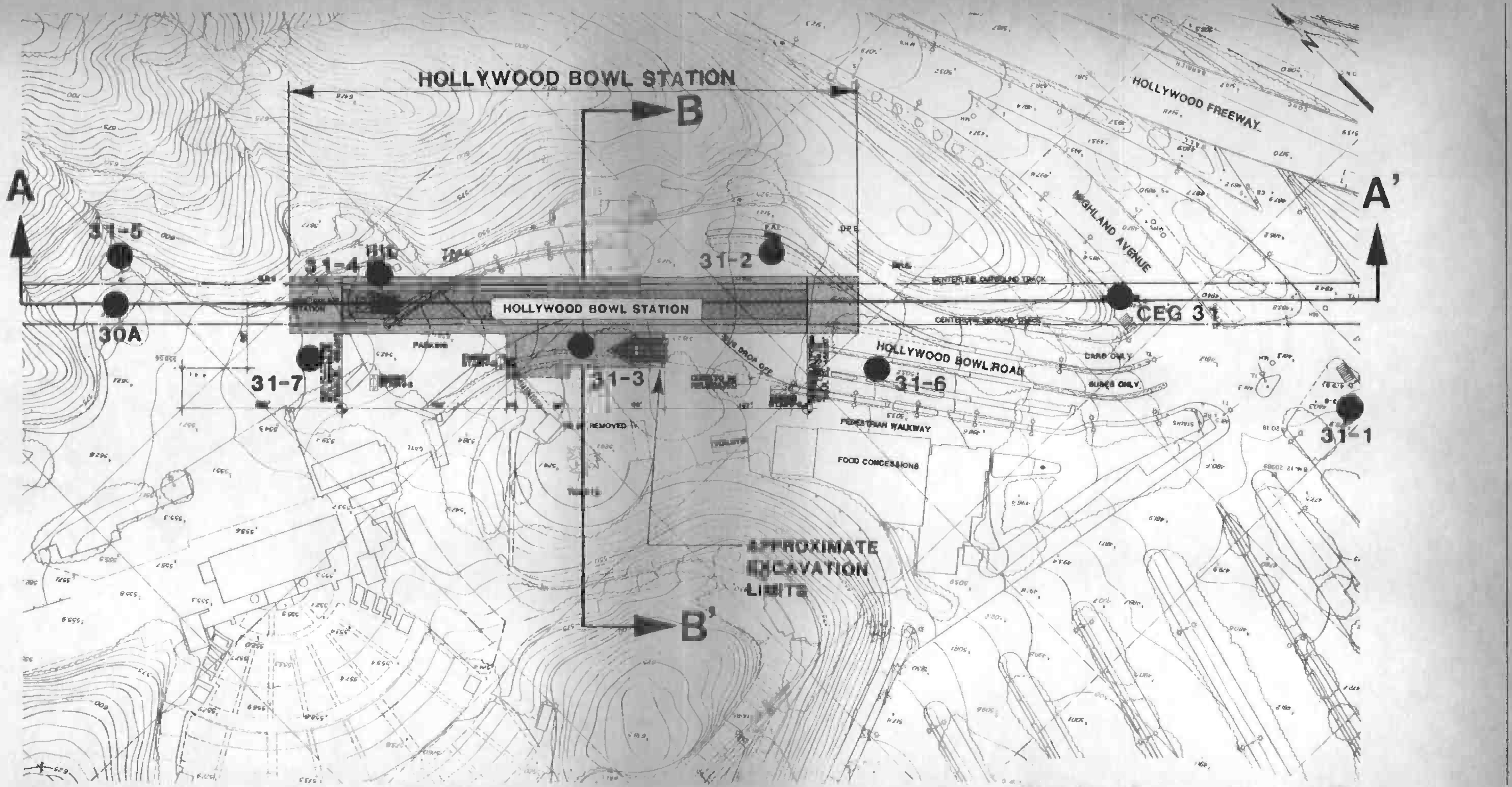
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted Ha Spellman Date May 1984

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

DESIGN UNIT A415
LOCATION OF BORINGS AND GEOLOGIC SECTION

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



REF: "PRELIMINARY HOLLYWOOD BOWL STATION SITE", DRAWING #A-60, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"= 100', DATED 8-24-83.

NOTES: 1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 4
2.) FOR SUBSURFACE SECTION B-B' SEE DRAWING NO. 5
3.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 6

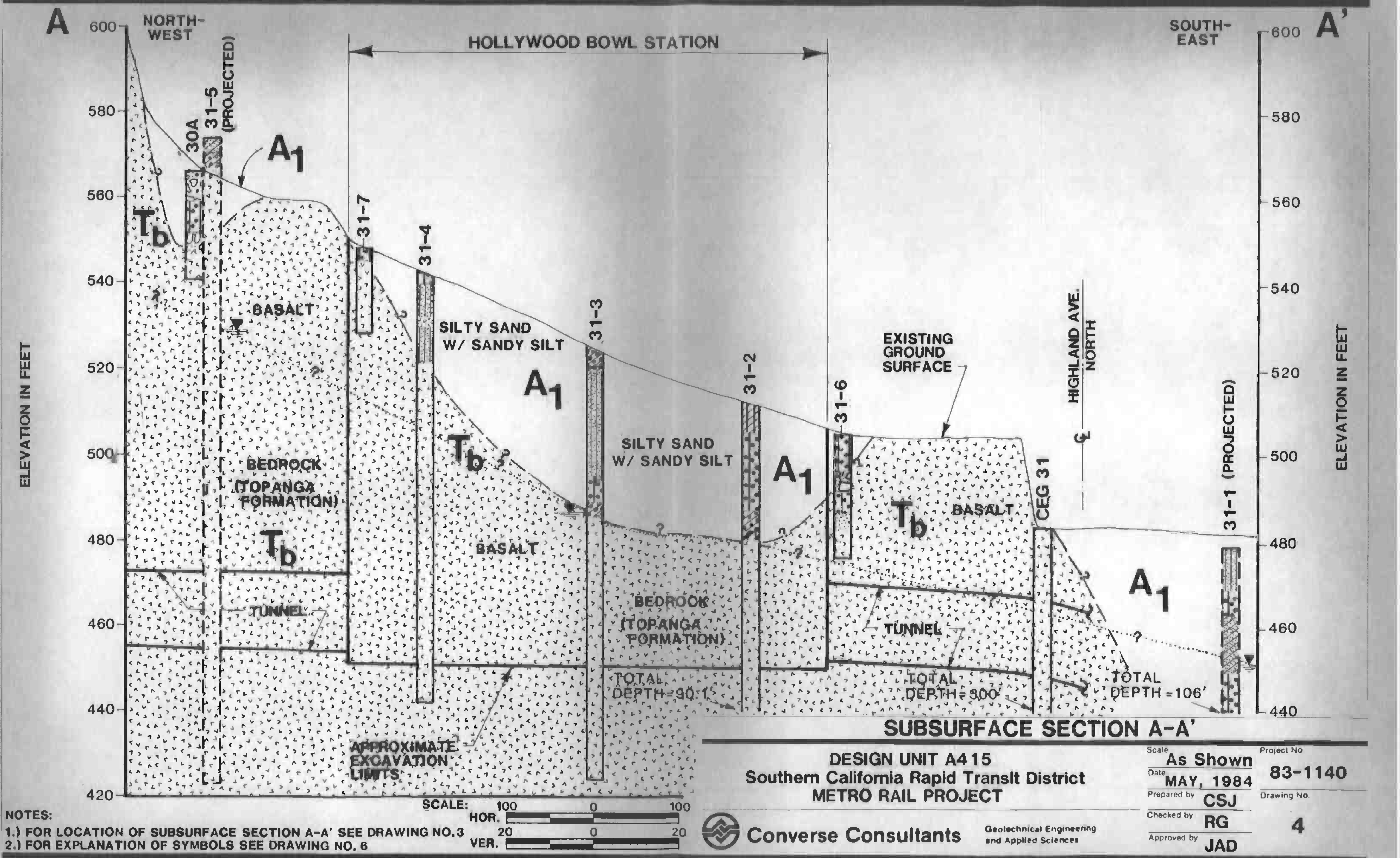
scale 100 50 0 100 200 feet

LOCATION OF BORINGS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Scale As Shown
Date MAY, 1984
Prepared by 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. 6

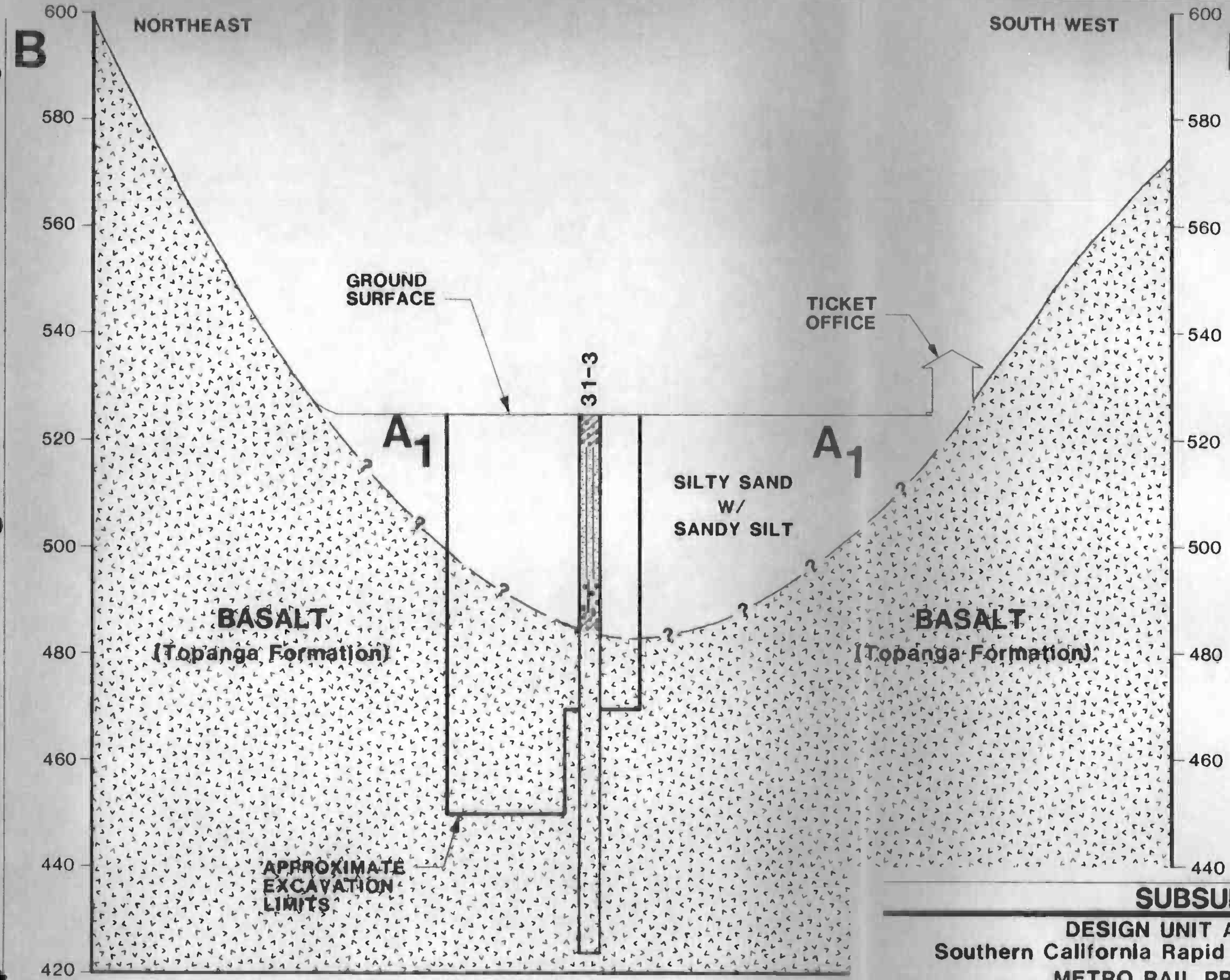
SUBSURFACE SECTION A-A'

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

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

Form D60/78



NOTES:

1.) FOR LOCATION OF SECTION B-B' SEE DRAWING NO. 3

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 6

SCALE:

HOR. 50 0 50 100

VER. 20 0 20 40

scale feet

SUBSURFACE SECTION B-B'

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale As Shown	Project No 83-1140
Date MAY, 1984	
Prepared by CSJ	Drawing No
Checked by RG	5
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)*

TERTIARY	MIOCENE	3	← Terzaghi Rock Condition Number ← Approximate boundary between Terzaghi numbers
		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
▭	CEMENTED ZONE
▩	META—SANDSTONE
▩	BASALT
▩	BRECCIA
▤	SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A415
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 6

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A415 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, and the 1983 and 1984 borings drilled for this A415 investigation. The specific boring logs included are summarized below:

- 1981

  CEG-30 and CEG-31

- 1983

  30-A, 30-B, 31-1 through 31-5

- 1984

  29-3, 31-6 and 31-7

Locations of the borings are shown on Drawings 2 and 3. Ground water observation wells (piezometers) were installed in the borings listed in Section 5.4 (Table 5-1). Geophysical downhole surveys were made for the 1981 investigation at Boring CEG-31 within the A415 investigation site.

The borings were drilled to depths generally ranging from 20 to 300 feet, and penetrated through the limited amount of alluvium into the underlying bedrock of the Topanga Formation. 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 deeply weathered basalt bedrock and the alluvium. NX-diamond coring was used to drill through the and basalt rock formation.

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 logs and for sample/core identification, labeling and storage of all samples, and installation of piezometer pipe, gravel pack and bentonite seals.

A.2.2 Drilling Contractor and Equipment

Most of the drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 750 and 1500 rotary wash rigs, each operated by a two-man crew. A&W Drilling Company of Brea, California, also provided a Mayhew 1000 rotary wash rig and the man-sized bucket auger rig for the site drilling.

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 a down-hole 320-pound to 340-pound slip-jar hammer with an 18-inch drop. The Converse sampler was followed with a standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the alluvium and weathered Topanga Formation were encountered, the borings were generally continuously sampled using a Pitcher Barrel sampler and Converse ring sampler. NX coring was utilized in the hard Topanga bedrock.

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. (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.) 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 the borings logs is presented in Table A-1 below.

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. In addition, the rock quality designation (RQD) based on core recovery is shown on the boring logs in the "Remarks" column. The RQD percentage represents the approximate percentage of intact pieces of core that are more than 10 cm (4 inches) long from a particular core run.

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings 30, 30B, CEG-31, 31-1 and 31-5 located either at or in the vicinity of the Hollywood Bowl Station site. 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 5.3 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.

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 29-3

Proj: DESIGN UNIT A415
Date Drilled 3-12-14-84
Ground Elev. 503'
Drill Rig Failing 1500
Logged By M. Schulter
Total Depth 126.0
Hole Diameter 4 7/8"
Hammer Weight & Fall 325# @ 18"/140# @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.5 ASPHALT			C	
	ML	ALLUVIUM 0.5-6.5 CLAYEY SILT: moderate yellowish brown, with sand; stiff to very stiff; slightly moist to moist			A	
2						
4						
		gravelly zone, moderate brown	C-1		DR	
6						
	SC	6.5-8.0 CLAYEY SAND: dusky yellowish brown; medium dense to dense; slightly moist to moist; trace of gravel			A	
8	SM	8.0-11.5 SILTY SAND: moderate yellowish brown; very dense; slightly moist				
10			J-1		SS	
					RD	
12	55°	BEDROCK 11.5-126.0 SILTSTONE/SANDSTONE: light brown and medium gray; slightly moist; dense; thinly laminated; cemented layers				
					C	
14		Physical Condition: closely fractured; friable; deep to moderate weathering	Box 1	1		3.2/3.5 recovery RQD = 0%
16						
				2		1.9/5.0 recovery
18		18.0-30.0 INTERBEDDED CLAYSTONE & SILTSTONE: intensely fractured; soft to friable deep weathering; plastic to friable				
20						

Sheet 1 of 6

Project DESIGN UNIT A415 Date Drilled 3-12-13-84 Hole No. 29-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20		11.5-126.0 SILTSTONE: continued			C	RQD = 20%
22		intensely fractured; soft; siltstone with interbedded claystone	Box 1	3		1.5/3.0 recovery RQD = 50%
24				4		1.6/2.0 recovery RQD = 80%
26						
28		intense to closely fractured		5		1.5/5.0 recovery RQD = 30%
			Box 2			
30		30.0-34.0 SANDSTONE/SILTSTONE: moderate yellowish brown; dense; moist Physical Condition: moderately fractured; friable; deeply weathered				
32				6		1.7/2.5 recovery RQD = 68%
34		34.0-38.0 SILTSTONE: with interbedded sandstone, moderate yellowish brown; stiff; moist Condition: moderately fractured; friable; deep to moderate weathering		7		1.2/1.5 recovery RQD = 48%
36				8		1.7/2.5 recovery RQD = 48%
38		38.0-41.5 CLAYSTONE/SILTSTONE: dark gray; moist, very stiff Physical Condition: moderately fractured, friable, moderately weathered				
40				9		2.5/4.0 recovery RQD = 63%
42		41.5-56.0 SANDSTONE/SILTSTONE: medium gray with moderate yellowish brown weathering and dark gray (siltstone) sandstone; thinly bedded; moderately fractured, low hardness; weak, mod. weathered	Box 3	10		3/12/84
44						

Sheet 2 of 6

Project DESIGN UNIT A415 Date Drilled 3-13-84 Hole No. 29-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		SILTSTONE: thinly bedded, moderate to closely fractured, fragile; moderate weathering		10	C	3.0/3.5 recovery RQD = 26%
46		<u>SANDSTONE/SILTSTONE</u> (interbedded) grayish black to medium light gray, thinly bedded Sandstone: moderately to little fractured, low to moderately hard, moderaterly strong, little weathering Siltstone: moderately to little fractured, low hardness, weak to moderately strong, little weathering	Box 3			
48	62°			11		4.1/4.5 recovery RQD = 75%
50						
						slightly petroliferous
52						
		53.6 fractured zone in sandstone, little weathering		12		5.0/5.0 recovery RQD = 84%
54			Box 4			
56		56.0-63.5 SILTSTONE WITH SANDSTONE <u>Physical Condition</u>: little fractured, low (siltstone) to moderately hard (sandstone), moderately strong, little to fresh weathering, very thinly bedded				
58				13		5.0/5.0 recovery RQD = 72%
60						
62			Box 5	14		5.0/5.0 recovery RQD = 86%
		63.5-74.5 SANDSTONE: medium light gray, moderately hard to hard, medium to thickly bedded, little fractured to massive, moderately strong, fresh				
64						
66				15		
68						

Sheet 3 of 6

Project DESIGN UNIT A415 Date Drilled 3-13-14-84 Hole No. 29-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		63.5-74.5 SANDSTONE: continued	Box 5	15		4.6/5.0 recovery RQD = 66%
70						
		little fractured, low-moderately hard, weak to moderately strong, little weathering	Box 6			
72				16		4.0/5.0 recovery RQD = 60%
74		74.5-78.7 SILTSTONE/SANDSTONE: dark gray, thinly laminated, little to closely fractured, moderately hard, moderately strong, little weathering to fresh, moist				
76				17		5.0/5.0 recovery RQD = 84%
78		78.7-84.5 SANDSTONE: medium gray, medium bedding, little fractured, moderately hard, moderately strong to strong, fresh, trace gravel, moist				
80	50°					
82			Box 7	18		4.75/5.0 recovery RQD = 94%
84		84.5-86.5 SILTSTONE/SANDSTONE: dark gray, thinly to very thinly laminated, little fractured, moderately hard, strong, fresh, moist				
86		86.5-88.5 SANDSTONE: medium gray, medium bedding, massive, fresh		19		3.5/3.5 recovery RQD = 97%
88		88.5-96.5 SILTSTONE/SANDSTONE: medium gray to dark gray, thinly laminated, closely to little fractured, low to moderately hard, moderately strong				3-13-84 3-14-84
90			Box	20		
92						

Sheet 4 of 6

Project DESIGN UNIT A415 Date Drilled 3-14-84 Hole No. 29-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		88.5-96.5 SILTSTONE/SANDSTONE: continued thinly bedded, little fractured	Box 8	20	C	4.5/4.9 recovery RQD: 67%
94						
96		96.6-98.6 SANDSTONE: medium light gray, little fractured, moderately hard, moderately strong, fresh		21		5.0/5.0 recovery RQD = 70%
98		98.6-106.5 SILTSTONE/SANDSTONE: medium gray to dary gray, thinly laminated, moderately fractured, moderately hard, strong, fresh, moist				
100			Box 9	22		5.0/5.0 recovery RQD = 52%
102						
104						
106		106.5-108.5 soft to low hardness		23		4.7/5.0 recovery RQD = 80%
108						
110	39°		Box 10			
112		111.5-115.5 closely fractured, moderately hard to soft		24		4.9/5.0 recovery RQD = 78%
114						
116				25		

Sheet 5 of 6

Project DESIGN UNIT A415 Date Drilled 3-14-84 Hole No. 29-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116		117.0-118.5 SILTSTONE/SANDSTONE: continued intense to closely fractured, friable to soft, deep to moderate weathering		25		4.0/5.0 recovery RQD = 72%
118			Box 11			
120				26		4.4/5.0 recovery RQD = 60%
122		122.0-124.5 intensely fractured, friable to soft				
124				27		1.4/1.4 recovery RQD = 100%
126		End of Boring 126.0' Flushed hole 3-15-84 Flushed hole. Performed single packer pressure test @ 50' and 86'. Installed 1" PVC piezometer 0-46' non perforated 46-66' perforated (saw cut) 66-86' non perforated 86-126' perforated (saw cut) Backfilled with pea gravel.				
128						
130						
132						
134						
136						
138						
140						

Sheet 6 of 6

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

BORING LOG CEG 30

Proj: DESIGN UNIT A415
Date Drilled 2/26/81-3/3/81
Ground Elev. 476'
Drill Rig B-40
Logged By Stephen M. Testa
Total Depth 251.0'
Hole Diameter NX
Hammer Weight & Fall 140 lb., 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.2 CONCRETE:				
	SC	0.2-16.0 CLAYEY SAND: Alluvium grayish brown; moist			RD	clear day
2						
4						
6						
8						
10		continued; moist; loose			SS	
			J-1			1.0/1.5 recovery
					RD	
12						
14						
16						
18						
20						

Sheet 1 of 11

Project DESIGN UNIT A415 Date Drilled 2/26/81-2/27/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	SC	16.0-29.0 CLAYEY SAND: dark yellowish brown with gravel; moist; medium dense	J-2		SS	1.2/1.5 recovery
					RD	
22						
24						
26						
		27.0-29.0 gravelly layer				minor rod chatter 27.0-29.0'
28		WEATHERED BEDROCK 29.0-45.0' SAND, SILTY SAND AND SANDY CLAY: stratified; very thin to thin lamina of dark yellowish orange and dusky yellowish brown fine sand and silty sand; and pale yellowish brown sandy clay; mottled; moist; dense				
	15-20° SP (SM) (CL)				SS	
30			J-3			1.5/1.5 recovery
					RD	2/27/81
32						
34						
		35.0-40.0 gravelly zone				minor rod chatter from 35.0-40.0
36						
38						
40		moist; very dense				
			J-4		SS	1.5/1.5 recovery
					RD	
42						
44						

Sheet 2 of 11

Project DESIGN UNIT A415 Date Drilled 2/27/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		29.0-45.9 SAND, SILTY SAND AND SANDY CLAY (continued)			RD	
46						
48						
		45.0-155.8 INTERBEDDED SANDSTONE AND SILTSTONE: wavy, parallel, alternating very thin to medium lamina of light olive-gray siltstone and dark yellowish orange fine sandstone; moist;			C	
50				1		3.0/3.0 recovery
52		Physical Condition: massive; friable to low hardness; friable to weak strength; moderately weathered	Box #1			
54				2		3.5/4.9 recovery
56						
58		alternating very thin to medium lamina of brownish black siltstone and yellowish gray fine moderately weathered sandstone grading to fresh medium light gray sandstone				
	30°			3		3.9/3.9 recovery
60						
62						
		primarily sandstone to 62.8 then primarily siltstone to 70.3'		4		1.8/2.4 recovery
64						
			Box #2			
66				5		3.5/3.5 recovery
68				6		

Project DESIGN UNIT A415 Date Drilled 2/27/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		45.0-155.8 INTERBEDDED SANDSTONE AND SILTSTONE (continued): wavy parallel alternating very thin to medium lamina of light olive gray siltstone and fine to coarse medium light gray sandstone	Box #2	6	C	3.4/3.4 recovery
70		Physical Condition: little fractured, moderately hard to hard; low to moderate strength; fresh		7		oil film in drilling water 1.5/1.9 recovery
72		70.3-71.1 well cemented fine to medium sandstone; hard				
74			Box #3	8		4.5/4.5 recovery
76						
78				9		1.8/1.8 recovery
80				10		3.7/3.7 recovery
82						
84			Box #4	11		pocket penetrometer > 4.5 tsf
86		87.7-89.7 fine to medium sandstone also at 90.3 to 90.9, from 90.9 alternating sandstone and siltstone				4.7/4.7 recovery
88	40° ↘					
90				12		4.7/6.7 recovery
92						

Sheet 4 of 11

Project DESIGN UNIT A415 Date Drilled 2/27/81-2/28/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		45.0-155.8 INTERBEDDED SANDSTONE AND SILTSTONE (continued):	Box #4	12	C	
94		92.4-93.8 fine to medium sandstone; alternating sandstone and siltstone from 93.8		13		4.8/4.8 recovery
96			Box #5			
		Physical Condition: little fractured; moderately hard to hard; low to moderate strength; fresh; tends to fracture along bedding planes				
98				14		3.8/3.8 recovery
100						2-28-81
102				15		heavy continuous rain 2.0/2.0 recovery
104			Box #6			
106				16		5.0/5.0 recovery
108						
110		110.3-118.0 primarily greenish gray fine to medium grained well cemented sandstone with siltstone		17		5.0/5.0 recovery
112			Box #7			
				18		1.0/1.0 recovery
114		114.5-115.3 coarse sandstone; well cemented; moderately hard		19		1.0/1.0 recovery
116				20		

Project DESIGN UNIT A415 Date Drilled 2/28/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116	40°	45.0-155.8 INTERBEDDED SANDSTONE AND SILTSTONE (continued):	Box #7	20	C	3.0/3.0 recovery
118		from 118.9 wavy parallel alternating very thin to medium lamina of greenish gray sandstone and siltstone				
120				21		4.0/5.0 recovery
122		Physical Condition: little fractured; moderately hard to hard; low to moderate strength; fresh		22		0.5/2.5 recovery
124			Box #8			
126				23		2.5/2.5 recovery
128				24		pocket penetrometer >4.5 tsf
130		131.5 to 132 coarse well cemented greenish gray sandstone		25		2.5/2.5 recovery 2.0/2.5 recovery
132		closely fractured		26		1.5/2.0 recovery
134			Box #9	27		3.0/3.0 recovery
136		137.0-137.5 primarily sandstone; little fractured				
138				28		
140						

Project DESIGN UNIT A415 Date Drilled 2/28/81-3/2/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
140		45.0-159.8 INTERBEDDED SANDSTONE AND SILTSTONE (continued): wavy, parallel, alternating very thin to medium lamina of greenish gray fine to coarse sandstone and brownish black siltstone	Box #9	28	C	5.0/5.0 recovery
142						
144		Physical Condition: moderately to closely fractured; moderate to low hardness; moderately strong; fresh				
146			Box #10	29		5.0/5.0 recovery
148		148.5-152.8 fine to medium sandstone				3-2-81 heavy rain until 11:00 a.m.
150	40°			30		4.8/4.8 recovery
152		low hardness from 151.0'; closely fractured				
154		155.8 to 156.4 clay shear zone	Box #11	31		4.5/4.5 recovery
156		157.2 to 157.9 fine to medium well cemented sandstone; moderately hard				
158		159.7-162.6 CLAY GOUGE: dark greenish gray		32		2.5/2.5 recovery
160				33		3.2/3.2 recovery
162		162.6-171.3 VOLCANIC BRECCIA: dark greenish gray; fine grained basalt fragments in a clay matrix	Box #12	34		
164						

Sheet 7 of 11

Project DESIGN UNIT A415 Date Drilled 3/2/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
164		162.6-171.3 VOLCANIC BRECCIA (continued):		34	C	3.0/3.0 recovery
166				35		2.0/2.0 recovery
168		168.5 to 170.0 closely to intensely fractured; slickensided surfaces	Box #12	36		3.3/3.3 recovery
170						
172		171.3-173.0 SANDSTONE: greenish black, medium to coarse; hard		37		1.7/1.7 recovery
174		173.0-174.9 VOLCANIC BRECCIA: dark greenish gray; fine grained basalt in a clay matrix; intensely to closely fractured		38		1.7/1.9 recovery
176		174.9-183.5 SANDSTONE: light gray fine to to coarse grained; hard; closely to intensely fractured	Box #13	39		3.1/3.1 recovery
178						
180				40		4.6/4.6 recovery
182						
184		183.5-251.0 BASALT: greenish black; many slickensided surfaces		41		3.5/3.5 recovery
186		Physical Condition: crushed to closely fractured; moderately hard; weak to moderately strong; moderately weathered		42		1.9/1.9 recovery
188						

Project DESIGN UNIT A415 Date Drilled 3/3/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
188		183.5-251.0 BASALT: greenish black; numerous slickensided fractures	Box #14	43	C	3/3/81 3.3/3.3 recovery
190		190.0-191.0 Physical Condition: little fractured; moderately hard to hard; moderately strong; fresh 191.6 6" shear zone				
				44		1.7/1.7 recovery
192						
			Box #15	45		
194						4.7/4.7 recovery
196						
				46		
198						
						4.1/4.1 recovery
200						
202						
				47		1.2/1.2 recovery
		203.0-215.0 intensely to closely fractured, numerous slickensides	Box #16	48		
204						2.4/2.4 recovery
				49		1.2/2.6 recovery
206						
208		208.5-212.0 crushed to intensely fractured		50		1.5/1.9 recovery
210				51		2.2/2.7 recovery
212						

Sheet 9 of 11

Project DESIGN UNIT A415 Date Drilled 3/3/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
212.0		183.5-251.0 <u>BASALT (continued):</u>	Box #16	51		
				52		
214.0			Box #17			2.4/3.0 recovery
		215.0-226.0 closely to moderately fractured with fault gouge and intensely fractured zones				
216.0				53		2.2/2.4 recovery
		217.8 thin fault gouge zone				
218.0				54		2.2/2.5 recovery
		219.0 and 214.5 fault gouge				
220.0				55		1.9/2.5 recovery
222.0		222.0-223.0 fault gouge				
				56		
224.0		224.0-225.0 gouge; intensely fractured				2.0/2.0 recovery
			Box #18	57		
226.0		226.0 very thin gouge zone 226.0-232.0 <u>Physical Condition:</u> moderately fractured; hard; strong; fresh				2.9/3.0 recovery
228.0				58		3.0/3.0 recovery
230.0				59		
232.0						2.0/2.0 recovery
				60		3.8/3.8 recovery
234.0			Box #19			
236.0						

Sheet 10 of 11

Project DESIGN UNIT A415 Date Drilled 3/3/81 Hole No. 30

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
236.0		Basalt (continued):	Box #19	60		
		Physical Condition: moderately fractured; hard; sttong; fresh		61		1.1/1.2 recovery
238.0				62		2.6/2.6 recovery
		239.8-240.6 intensely fractured				
240.0						
				63		
242.0						2.4/2.4 recovery
				64		
244.0						
			Box #20			4.7/4.7 recovery
246.0						
248.0		most fracture sets rehealed with silica		65		
						2.5/3.2 recovery
250.0						
		End of Boring 251.0'				
		3/3/81; 3/4/81 E-log 3/4/81 water pressure test				hole cleaned out 2 times before completing E logs
252.0						
254.0						
256.0						
258.0						
260.0						

Sheet 11 of 11

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 30-A

Proj: DESIGN UNIT A415

Date Drilled 2/22/83

Ground Elev. 560'

Drill Rig Mayhew 1000

Logged By G. Halbert

Total Depth 25'

Hole Diameter 4 7/8"

Hammer Weight & Fall SPT 140 lb, 30" C-340 lb, 24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.3 AC PAVEMENT 0.3-7.5 GRAVELLY SAND: coarse sand with fine gravel (slopewash)			RD	
2						
4		grading finer				
6		with silty sand				
8	SM	ALLUVIUM 7.5-15.0 SILTY SAND: dark yellowish brown and black, medium dense, moist				
10			J-1		SS	1.2/1.5 recovery
12						
14						
	SW	15.0-20.0 GRAVELLY SAND: dense, moist, with fines				light chatter
16						
18						
20						

Sheet 1 **of** 2

Project DESIGN UNIT A415 Date Drilled 2-22-83 Hole No. 30A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20		BEDROCK 20.0-25.0 <u>BASALT</u>: olive black, intensely fractured, moderately weathered, friable to weak strength	C-1 J-2			0.7/0.7 recovery refusal at 7"
22						
24						
		End of Boring 25.0'				No water entered hole while open. Backfilled with pea gravel & plugged with 6" concrete.
26						
28						
30						
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

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

BORING LOG 30-B

Proj: DESIGN UNIT A415 **Date Drilled** 2-23-83 **Ground Elev.** 467'

Drill Rig B. Auger **Logged By** D. Gilette **Total Depth** 32.0'

Hole Diameter 36" **Hammer Weight & Fall** N/A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT				2" asphalt
		FILL 0.2-9.0 SANDY CLAY: various shades of brown, contains bottles, pipe, wood, very stiff, moist	OBSERVATION HOLE - NO SAMPLES REQUIRED			0.0-11.0 slight ravelling
2						11-32 hole stands well
4						
6						GROUND WATER DATA - location and eatimated amount of seepae: ±1.0 gpm from north ±0.5 gpm from south
		7.0 - roots and wood				
8						
	CL GP	9.0-15.0 SANDY CLAY AND BOULDERS: dark yellowish orange and light brown, 8-12" boulders (sandstone & basalt), very stiff, moist				
10						
12						
14						
	CL	15.0-22.0 CLAY AND BOULDERS: medium light gray and light brown with fine sand and 10" boulders (basalt), stiff to very stiff, moist				
16						
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A415 Date Drilled 2-23-83 Hole No. 30B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	CL	15.0-22.0 CLAY: (continued)				W.L. 21.7 after 20 hours
22 24 26	GP	22.0-27.5 SANDY GRAVEL: dark reddish brown, contains cobbles&boulders, dense, wet				ground water seeps in at bedrock contact
28 30	72° ↘	BEDROCK 27.5-32.0 SANDSTONE: dark yellowish orange, slightly weathered, moderately hard				hard drilling bedding dips 72° northerly not able to drill deeper, too hard
32 34 36 38 40 42 44		END OF BORING 30'				2-24-83 hole backfilled with native material

Sheet 2 of 2

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

BORING LOG CEG 31

Proj: DESIGN UNIT A415 **Date Drilled** 2-12-24-81 **Ground Elev.** 482.0'

Drill Rig Mobile B-40 **Logged By** Schoeberlein/Testa **Total Depth** 300.0'

Hole Diameter 3" **Hammer Weight & Fall** 140 lb, 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.2 CONCRETE			RD	
		0.2-0.6 SAND BASE				
		BEDROCK 0.6-85.4 BASALT: moderate yellowish brown to light olive brown				
2						
		Physical Condition: crushed, friable hardness, friable strength, deeply weathered				
4						
			J-1		SS	0.3/0.3 recovery
6					RD	
8						
						rig chatter
10			Box 1	1	C	0.0/1.0 recovery
		moderate olive brown, fine grained vesicular, vesicules commonly filled with chlorite and zeolites Physical Condition: intensely to closely fractured, moderately hard, moderately strong, deep to moderate weathering, fracture planes commonly coated with iron oxides, clay (up to 3 mm) and occasional calcite		2		1.6/2.0 recovery
12						
				3		0.0/1.0 recovery
14					RD	
16						
						rig drilling smoother
18						
20						

Sheet 1 of 13

Project DESIGN A415 Date Drilled 2-12-13-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20		0.6-85.4 BASALT: continued moderate olive brown, fine grained, vesicular Physical Condition: intensely to closely fractured, moderately hard, moderately strong, deeply weathered	Box 1		C	
22						
					RD	0.1/0.4 recovery
24						
			J-2		SS	
					RD	
26		26.5-26.8 hard lens				
28		29.0-29.5 hard				
30			J-3		SS	0.1/0.1 recovery
					RD	2-12-81 2-13-81
32						5-8 min/ft drilling rate
34		olive gray and light olive brown to brownish black				
			Box 1	6	C	2.5/3.0 recovery
36						
38				7		
40		40.4-45.0 dark yellowish brown, fresh volcanic glass fragments in altered olive gray ground mass, 41.5-42.0 intensely fractured, clay binder				4.3/4.3 recovery
42			Box 2	8		
44						

Sheet 2 of 13

Project DESIGN UNIT A415 Date Drilled 2-12-13-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		0.6-84.5 BASALT: continued	Box 2	8	C	4.7/4.7 recovery
46		45.0-46.0 fracture set, green basalt inclusions increasing in frequency, greyish blue green, rock becoming more competent, . most freactures healed				
48				9		1.9/2.2. recovery
		Physical Condition: closely to moderately fractured, moderately hard to hard, strong, little weathered		10		3.0/3.0 recovery
50						
52			Box	11		1.7/1.7 recovery
54				12		2.9/3.3 recovery
56		56.0-84.5 greyish blue green Physical Condition: closely to moderately fractured, moderately hard to hard, strong, moderately weathered				
				?		
58				13		5.0/5.0 recovery
60						
62						
			Box 4	14		2.7/2.7 recovery
64		64.0-65.0 extremely weathered zone, crushed to intensely fractured 65.0-85.4 breccia, well recemented				
66				15		2.3/2.3 recovery
68				16		

Sheet 3 of 13

Project DESIGN UNIT A415 Date Drilled 2-12-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		56.0-85.4 BASALT: continued fined grained vesicular basalt with numerous hairline fractures	Box 4	16	C	4.6/4.6 recovery
70						
72						
			Box 5	17		4.8/4.8 recovery
74		74.0-75.0 intensely to closely fractured zone				
76		calcite filled fractures				
				18		4.1/4.8 recovery
78						
80						
		81.0-82.5 intensely to closely fractured zone				
82				19		0.3/0.5 recovery
				20		
			Box 6			
84		conformable contact				drilling slower 4.9/4.9 recovery
		85.4-91.0 SILTSTONE: greyish black, thin sandstone inclusions, well cemented, fractures all rehealed with calcite filling, alternating wavy parallel very thin to medium fine grained siltstone and sandstone Physical Condition: little fractured to massive, hard, strong, fresh				oil in return water
86						petroleum sample 87-89
				21		drill rate increasing 4.7/4.7 recovery
88	50°					
90		conformable contact				
		91.0-102.7 SANDSTONE BRECCIA:				
92						

Sheet 4 of 13

Project DESIGN UNIT A415 Date Drilled 2-12-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		91.0-102.7 <u>SANDSTONE BRECCIA</u>: continued olive grey to medium dark grey in a greyish black fine grained matrix, thin inclusions of sandstone and fine grained porphyry slightly metamorphased	Box 6	22	C	4.5/4.5 recovery
94		<u>Physical Condition</u>: moderately to little fractured, hard, strong, fresh	Box 7			
96				23		1.0/1.0 recovery
98				24		4.9/4.9 recovery
100		slickensides present, infilling of fracture surface				
102						
		102.7-109.0 <u>CONGLOMERATE</u>: medium gray, matrix of quartz sand, feldspar sand, siltstone, sandstone and igneous gravels well cemented, volcanic and granitic grains up to 1.5" in diameter	Box	25		4.9/4.9 recovery
104						
106		<u>Physical Condition</u>: closely to little fractured, hard, strong, fresh				
				26		4.7/4.7 recovery
108		conformable contact				minor rig chatter
		109.0-114.5 <u>SANDSTONE BRECCIA</u>: olive grey, medium gray in greenish black fine grained matrix, thin inclusions of sandstone and fine grained porphyry slightly metamorphased				
110						
112			Box 9	27		2.4/2.4 recovery
		113.7-114.7 deeply weathered shear zone				
114						
		114.5-121.4 <u>INTERBEDDED SANDSTONE AND SILTSTONE</u>: dark gray to greyish				
116						

Sheet 5 of 13

Project DESIGN UNIT A415 Date Drilled 2-12-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116	60°	114.5-121.4 INTERBEDDED SANDSTONE AND SILTSTONE : continued black, well cemented, bedding dips 60°, fracturing has 3 sets 70°, 20° and 90°, 20° clay filled, 70° fracture open and clay coated, 90° fractures calcite filled Physical Condition: moderately fractured, very hard, strong, fresh	Box 9	28	C	2.9/2.9 recovery
118				29		1.0/1.0 recovery
				30		2.8/2.8 recovery
120		120.8-121.4 shear zone crushed				
		121.4-141.6 METASANDSTONE: medium gray, coarse sand~50% quartz, quartz cement in fractures Physical Condition: closely to moderately fractured, very hard, strong, fresh	Box 10	31		1.2/1.2 recovery
122				32		4.9/4.9 recovery
124						
126						
				33		1.8/1.8 recovery
128	70°	128.5 4" coarse grained zone				
				34		2.9/2.9 recovery
130						
		131.5 6" metaconglomerate, wavy lineated weakly schistose due to larger grain size up to 4" in diameter	Box 11			
132				35		4.6/4.6 recovery
134						
		135.0 grain size decreases				
136	70°					
				36		
138						
		139.0-141.0 brecciated, closely fractured				
140						

Sheet 6 of 13

Project DESIGN UNIT A415 Date Drilled 2-15-16-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
140		121.4-141.6 METASANDSTONE: continued	Box11	36	C	4.4/4.4 recovery
142		141.6-151.2 INTERBEDDED SANDSTONE & SILTSTONE: medium gray to grayish black fine sand, fractures open and calcite filled Physical Condition: intensely to moderately fractured, hard to very hard. strong, fresh, from 144 intensely fractured, many open fractures		37		1.0/1.0 recovery
144			Box 12	38		4.8/4.8 recovery
146						
	↘ 70°					
148						
		149.0-150.7 little fractured, finer grained		39		3.8/3.8 recovery
150		slickensides				
		151.2-190.8 BASALT: olive black, fine to medium grained, closely to intensely fractured, primarily chlorite along fracture planes, commonly showing slickenside surfaces and minor calcite		40		0.2/1.4 recovery
152			Box 13	41		2-16-81 1.8/2.2 recovery
154						
156				42		2.4/2.8 recovery
158		Physical Condition: closely to intensely fractured, moderately hard, moderately strong, fresh		43		1.8/1.8 recovery
				44		0.4/0.5 recovery
160				45		1.2/2.0 recovery
162		60° fracture planes most prominant	Box 14	46		0.7/0.7 recovery
				47		1.5/1.9 recovery
164						

Sheet 7 of 13

Project DESIGN UNIT A415 Date Drilled 2-16-17-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
164		151.7-190.8 BASALT: continued		47	C	
				48		0.7/1.0 recovery
		fracture planes straight to irregular, numerous hairline fractures		49		0.5/1.0 recovery
166						
				50		0.8/1.2 recovery
168		Physical Condition: closely to intensely fractured, moderately hard, moderately strong, fresh	Box 14	51		2.0/2.5 recovery
170						
				52		0.8/1.0 recovery
172				53		0.0/1.4 recovery
				54		1.7/3.0 recovery
174			Box 15			
176				55		2-17-81 0.7/1.0 recovery
				56		1.0/1.0 recovery
178				57		1.8/2.4 recovery
180				58		0.7/1.0 recovery
182				59		1.4/1.6 recovery
				60		1.8/2.2 recovery
184						
			Box 16	61		2.5/2.8 recovery
186						
188				62		

Project DESIGN UNIT A415 Date Drilled 2-11-23-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
188		151.2-190.8 BASALT: continued			C	
				62		4.8/4.8 recovery
190						
		190.8-260.7 SILTSTONE AND SANDSTONE INTERBEDS: primarily olive black, very thin to medium parallel lamina siltstone with subordinate fine well cemented bluish gray sandstone, hairline fractures apparent				
192				63		oil film in drilling water 1.5/2.0 recovery
194				64		pocket penetrometer >4.5 tsf 2.0/2.o recovery
196				65		0.7/1.2 recovery
198		primarily brownish black siltstone and fine light bluish gray sandstone		66		4.5/4.8 recovery
200						
202		very thin to medium wavy lamina				2-18-81
204		Physical Condition: moderately fractured, moderate hard, weak to moderately strong, fresh, tends to fracture along bedding planes and healed fractures	Box 18	67		4.2/4.2 recovery
206						
				68		0.9/1.0 recovery
208				69		4.7/4.7 recovery
210						
212						

Sheet 9 of 13

Project DESIGN UNIT A415 Date Drilled 2-12-23-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
212		190.8-260.7 SILTSTONE AND SANDSTONE INTERBEDS: continued	Box 18	69 70		0.5/0.5 recovery
214		wavy alternating very thin to medium lamina of primarily brownish black siltstone and subordinate light bluish gray fine sandstone, numerous silica filled and hairline fractures	Box 19	71		oil film in drilling water 4.1/4.8 recovery
216						
218		Physical Condition: moderately fractured, moderately hard, weak to moderately strong, fresh, tends to fracture along bedding planes and healed fractures		72		pocket penetrometer >4.5 tsf 4.7/4.7 recovery
220						
222						
224			Box 20	73		4.3/4.8 recovery
226		continued, fossiliferous siltstone		74		2-23-81 0.5/0.5 recovery
228				75		4.9/4.9 recovery
230						
232						oil film in drilling water
234			Box 21	76		5.0/5.0 recovery
236						

Sheet 10 of 13

Project DESIGN UNIT A415 Date Drilled 2-23-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
236		190.8-260.7 SILTSTONE AND SANDSTONE INTERBEDS: continued		76	C	5.0/5.0 recovery
238		Physical Condition: moderately fractured, moderately hard, weak to moderately strong, fresh, tends to fracture along bedding planes and healed fractures continued, fossiliferous, moderate yellowish brown siltstone inclusion at 239.5	Box 21	77		3.5/3.5 recovery oil film in drilling water
240						
242				78		1.5/1.6 recovery
			Box 22			
244				79		3.2/4.8 recovery
246						pocket penetrometer >4.5 tsf
				80		2.8/2.8 recovery
248						
250						
				81		2.4/2.4 recovery
252			Box 23			
				82		4.7/4.7 recovery
254						
256						
		grades sandier from 256.9				
				83		2-24-84 4.8/4.8 recovery
258						
260			Box 24			

Sheet 11 of 13

Project DESIGN UNIT A415 Date Drilled 2-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
260		190.8-260.7 SILTSTONE AND SANDSTONE INTERBEDS: continued			C	
		260.7-262.1 SANDSTONE: medium gray, fine to coarse, well cemented, quartz rich, lower contact 50°		83		4.8/4.8 recovery
262		262.1-265.1 CONGLOMERATE: greenish gray, coarse (up to 10 mm max. dia.), intensely fractured (clay filled), clasts include quartz, numerous volcanics, grades to coarse sandstone with depth		84		0.5/1.0 recovery
264			Box 24	85		4.5/4.5 recovery
		265.1-300.0 BASALT: dark greenish gray, fine grained, vesicular, moderately to closely fractured, fracture planes commonly filled with chlorite; slickenside surfaces at 20° to core axis, numerous hairline fractures apparent				
266						loss of circulation water
268				86		4.8/4.8 recovery
270						
272						
		at 272.4 olive black, fine to medium grained, vesicules	Box 25			pocket penetrometer 4.5 tsf
274				87		4.8/4.8 recovery
276						gas detector 0.0% LEL, no gas encountered
278						
280			Box 26	88		4.8/4.8 recovery
282				89		4.7/4.7 recovery
284						

Sheet 12 of 13

Project DESIGN UNIT A415 Date Drilled 2-24-81 Hole No. 31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
284		265.1-300.0 BASALT: continued olive black, fine to medium grained, moderately to closely fractured, numerous hairline fractured apparent	Box 26	89	C	4.7/4.7 recovery
286				90		0.9/0.9 recovery
288				91		2.1/2.1 recovery
290				92		1.0/1.0 recovery
			Box 27	93		1.9/1.9 recovery
292				94		1.5/3.4 recovery
294						no recovery, mislatch of sample tube
296				95		0.0/4.0 recovery
298						
						2-24-81
300						

End of Boring 300.0'

2-25-81: Water pressure test conducted at intervals 51.0' to 138.0', and 138.0' to 300.0' at 20, 40 and 60 psi
2-27-81: Performed downhole geophysics
2-28-81: Piezometer installation, installed 180.0' to 2" PVC perforated PVC at the following intervals: 80.0' to 100.0' and 155.0' to 175.0', backfilled hole with pea gravel
3-2-81: water sampled

302
304
306
308

Sheet 13 of 13

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

BORING LOG 31-1

Proj: DESIGN UNIT A415
Date Drilled 10/3/83-10/6/83
Ground Elev. 479
Drill Rig Failing 750
Logged By DG/MD/SS
Total Depth 106.0
Hole Diameter 3"
Hammer Weight & Fall 140 lbs., 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT			GB	
	AF	0.2-0.5 BASE ROCK			AD	
	ML/ SM	ALLUVIUM 0.5-10.0 SANDY SILT/SILTY SAND: dusky yellowish brown with gravel; moist; stiff; medium dense				
2						
4						
6	SM/ GP	6.0-8.0 gravelly layer				6.0-8.0 rig chatter
8	ML/ SM				RD	
10	SM	10.0-16.0 SILTY SAND: light brown with gravel; moist; medium dense			SS	
					RD	
12						
14						
16	CL	16.0-29.0 SANDY CLAY: dusky yellowish brown with gravel; moist; firm				
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A415 Date Drilled 10/3/83-10/6/83 Hole No. 31-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	CL	16.0-29.0 SANDY CLAY (continues as above):			SS	
					RD	
22						
24						
26						
28						
	SM	29.0-48.0 SILTY SAND: dusky brown; moist; loose; with gravel				
30					SS	
					RD	
32						
34						
36						
38						
40					SS	
		becoming medium dense				
					RD	
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A415 Date Drilled 10/3/83-10/6/83 Hole No. 31-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44	SM	29.0-48.0 SILTY SAND (continued): 44.0-46.0 gravelly layer			RD	44.0-46.0 rig chatter
46						
48	SM	48.0-56.0 SILTY SAND: moderate brown and dark gray with gravel; moist; dense; trace organics and slight organic odor				
50					SS	
					RD	
52						
54						
56	SC	56.0-72.5 CLAYEY SAND: greenish black; wet; very dense	PB-1	1	PB	2.0/2.5 recovery
58					RD	
60						
62						
64		64.0-65.0 gravelly layer	PB-2	2	PB	1.5/2.5 recovery
66			PB-3	3		2.0/2.5 recovery
68						

Project DESIGN UNIT A415 Date Drilled 10/3/83-10/6/83 Hole No. 31-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68	SC	56.0-72.5 CLAYEY SAND (continued):	PB-3	3	PB	
					RD	
70						
72		BEDROCK				
		72.5-106.0 BASALT: medium light gray; fine grained		4	PB	
74		Physical Condition: intensely fractured; low hardness; weak strength; deeply to moderately weathered	PB-4			1.5/2.5 recovery
76		dark gray; slightly clay, wet, firm to very stiff	PB-5	5		1.2/2.5 recovery
78			PB-6	6		0.9/2.5 recovery
80					RD	
82						
84						
86						
		87.0-90.1 shear zone: gravelly clay; moist to wet; stiff; color change to olive gray		1	C	1.8/2.0 recovery
88		Physical Condition: crushed, soft to low hardness; plastic to weak strength; deep weathering; some fractures filled with dark gray clay-moist; firm to very stiff	Box #1	2		2.0/2.0 recovery
90				3		
92						

Sheet 4 of 5

Project DESIGN UNIT A415 Date Drilled 10/3/83-10/6/83 Hole No. 31-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		72.5-106.0 BASALT (continued): color becoming mottled-dark greenish gray, greenish black, and grayish black	Box #1	3	C	3.5/3.7 recovery
94		Physical Condition: intensely fractured; moderate to low hardness; weak to moderate strength; moderate weathering		4		
96						4.4/4.4 recovery drill rate = 20 min/ft
98						
				5		0.0/1.2 recovery drill rate = 8 min/ft
100			Box #2	6		1.0/2.3 recovery drill rate = 22 min/ft
102		thin clay-filled fractures at 104', 105'				5 October 1983 6 October 1983
		103.0-103.1 clay-filled shear zone		7		3.4/3.4 recovery
104		Physical Condition: intensely to closely fractured; moderately hard; weak to moderately strong; deeply to moderately weathered				
106		end of boring = 106.0'				flushed hole; set 2" ABS piezometer from 0.0-106.0, perforated from 86.0-106.0. Pulled casing and backfilled with pea gravel. Sealed top (0.5-4.0) with concrete. Cleaned site; covered hole with steel cap
108						
110						
112						
114						
116						

Sheet 5 of 5

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 31-2

Proj: DESIGN UNIT A415
Date Drilled 10/22/83-10/24/83
Ground Elev.
Drill Rig Failing 750
Logged By Steve Slaff
Total Depth 90.1
Hole Diameter NX
Hammer Weight & Fall 140 lb @ 30" SS; 320 lb @ 18" DR

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.3 ASPHALT			GB	
		ALLUVIUM 0.3-6.0 SANDY SILT: moderate yellowish brown; trace gravel; dry; with organics (rootlets); becoming moist, firm			AD	
2	ML		C-1		DR	1.0/1.0 recovery
					AD	
4			J-1		SS	1.5/1.5 recovery
6	SM/ ML	6.0-18.6 SILTY SAND/SANDY SILT: dark yellowish brown with gravel; moist, medium dense/very stiff			AD RD	
					DR	
8	GC	7.8-8.8 clayey gravel lens with color change to moderate yellowish brown	C-2			1.0/1.0 recovery
					RD	
	SM/ ML		J-2		SS	0.9/1.5 recovery
10					RD	
12			C-3		DR	1.0/1.0 recovery
					RD	
14			J-3		SS	0.9/1.5 recovery
					RD	
16			C-4		DR	1.0/1.0 recovery
					RD	
18						
	SM/ ML	18.6-24.6 SILTY SAND/SANDY SILT: dark yellowish brown with gravel;	J-4		SS	0.75/0.75 recovery refusal at 9"
20					RD	

Sheet 1 of 4

Project DESIGN UNIT A415 Date Drilled 10/22/83-10/24/83 Hole No. 31-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	SM/ML	18.6-24.6 SILTY SAND/SANDY SILT (continued): moist; very dense/hard 19.1-19.2 lense of fine quartz sand			RD	
22			C-5		DR	0.9-0.9 recovery refusal @ 11"
					RD	
24			J-5		SS	0.6/0.6 recovery refusal @ 7" rig chatter
	GC	24.6-30.0 CLAYEY GRAVEL: moderate yellowish brown; moist; very dense			RD	
26						
28			PB-1	1	PB	1.5/2.5 recovery
30	CL	30.0-32.3 SILTY CLAY: moderate yellowish brown; moist; hard with sand and gravel	PB-2	2		0.5/2.5 recovery Pitcher tube end damaged
32		BEDROCK 32.3-90.1 BASALT: mottled-dusky green and medium dark gray; porphyritic; much of basic glass devitrified; phenocrysts fine grained pyroxene and plagioclase feldspar. Clasts of fresher basalt are set in matrix of more highly altered basalt characterized by secondary minerals. Slickensides fairly common. Secondary minerals include quartz, chlorite and epidote.	PB-3	3		1.9/2.5 recovery
34			PB-4	4		1.2/1.2 recovery
36						
		Physical Condition: intensely to closely fractured; moderately hard; moderately strong; moderately weathered; thin calcite coatings on some fracture surfaces, most fractures closed and healed.	Box #1	1	C	RQD = 96% 2.3/2.3 recovery
38						
				2		0.3/0.3 recovery
40				3		RQD = 41% 2.8/2.8 recovery
42						10/22/83
				4		10/23/83
44						

Sheet 2 of 4

Project DESIGN UNIT A415 Date Drilled 10/22/83-10/24/83 Hole No. 31-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		32.3-90.1 BASALT (continued): quartz, chlorite are most abundant vessicle, cavity, and fracture filling minerals. Some zones exhibit slickensides at various orientations including horizontal. Predominantly greenish black; minor brown orange and yellow iron oxide on fracture surfaces.	Box #1	4	C	RQD = 79% 4.8/4.8 recovery
46						
48		46.7-47.2 core is fractured	Box #2	5		RQD = 0% 3.6/3.6 recovery
50		47.2-49.9 core completely disaggregated into fragments				
52		49.9-50.5 disaggregated basalt and alteration product: clay-dusky green to greenish black; moist; stiff (shear zone) fine grained calcite on fracture surfaces		6		RQD = 38% 5.6/5.6 recovery
54		51.1-53.1 core disaggregated into fragments				
56		core gradually changes color from green to dark gray after removal from the hole decreasing slickensides				
58		basalt nearly completely altered	Box #3	7		RQD = 28% 2.9/2.9 recovery
60				8		RQD = 32% 4.1/4.1 recovery
62		61.6-62.4 disaggregated basalt-fragments				
64			Box #4	9		RQD = 77% 6.9/6.9 recovery
66		calcite common on fracture surfaces				
68						

Sheet 3 of 4

Project DESIGN UNIT A415 Date Drilled 10/22/83-10/24/83 Hole No. 31-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		32.3-90.1 BASALT (continued):			C	
		some surfaces have a glossy sheen due to microcrystaline micaceous chlorite	Box #4	9		
70						23 October 1983
						24 October 1983
72		72.3-75.1 disaggregated zone with fragments		10		RQD = 43% 6.0/6.0 recovery
74		core does not break cleanly, but makes hackly, uneven surface or disaggregates when struck with hammer				
76		thin layers (0.05") of calcite ubiquitous on fracture surfaces	Box #5			
		disaggregated core may indicate fracture, shear, or fault zone				
78				11		RQD = 75% 6.0/6.0 recovery
80						
82		slickensides common; within 20° of horizontal				
				12		RQD = 34% 2.9/2.9 recovery
84			Box #6			
86				13		RQD = 19% 4.2/4.2 recovery
88						
			Box #7	14		1.0/1.0 recovery RQD = 100%
90		end of boring 90.1'				
92			tremmied in 2 sack cement grout; cleaned site; topped hole with concrete 10/24/83.			

Sheet 4 of 4

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

BORING LOG 31-3

Proj: DESIGN UNIT A415 **Date Drilled** 10-6-9-83 **Ground Elev.** 526.0
Drill Rig Failing 750 **Logged By** S. Slaff **Total Depth** 100.1
Hole Diameter NX **Hammer Weight & Fall** 140 lbs, 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT			GB	
		ALLUVIUM 0.4-4.7 GRAVELLY CLAY: moderate yellowish brown to moderate brown; with sand; moist; firm			AD	
2						
4		becoming less gravelly				
	ML	4.7-24.2 SANDY SILT: dark yellowish brown with gravel; moist; stiff				
6					RD	
			J-1		SS	1.0/1.5 recovery
8					RD	
10						
12						
			PB-1		PB	1.5/2.5 recovery
14						
		becoming very stiff				
16	(SM ML)	becoming sandier	J-2		SS	1.0/1.5 recovery
					RD	
18						
	GC	19.5-19.8 gravel lens				rig chatter at 19.5'
20	ML					

Project DESIGN UNIT A415 Date Drilled 10-6-9-83 Hole No. 31-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	ML	4.7-24.2 SANDY SILT: continued			RD	
22			lost		PB	0.0/2.5 recovery
24	ML	24.2-27.0 SANDY SILT: dark yellowish brown; moist; stiff	PB-2			2.5/2.5 recovery
26			J-3		SS	0.75/1.5 recovery
28	CL	27.0-29.4 SILTY CLAY: dark yellowish brown; moist; stiff			RD	10-6-83 10-7-83
30	CL	29.4-35.6 SANDY CLAY: moderate brown; moist; with gravel; very stiff	PB-3		PB	2.5/2.5 recovery
32			J-4		SS	1.0/1.5 recovery
	(SM)	becoming sandier				
34					RD	
36	GC	35.6-30.0 CLAYEY GRAVEL: moderate brown; moist; with sand; very dense				slight rig chatter
38			PB-4		PB	sporadic rig chatter 2.5/2.5 recovery
40		BEDROCK 39.0-100.1 BASALT: dusky yellowish green; aphanitic to fine grained; some quartz-filled fractures Physical Condition: intensely fractured, fractures closed by secondary minerals (calcite, zeolite, quartz); moderately hard, moderately strong, moderately weathered	J-5		SS RD	0.3/0.3 recovery refusal at 4"
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A415 Date Drilled 10-6-9-83 Hole No. 31-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		39.0-100.1 BASALT: continued basalt is medium gray on fresh surfaces; porphyritic with fine grained plagioclase feldspar, pyroxene in aphanitic groundmass. Physical Condition: fractures to 1.0" wide commonly filled with secondary minerals including chlorite. Some zones crushed, with clay filling fractures; some zones hard, strong, little weathered. Predominantly medium dark gray with highly altered zones dusky yellowish green. Gray basalt clasts are hard, moderately strong in matrix of dark green alteration products that are low hardness, weak, feel soapy, display slickensides	PB-5		PB	2.0/2.0 recovery disturbed
46					C	
48			Box 1	1		RQD of core was higher but it disaggregates upon removal from barrel RQD = 17% 4.8/4.8 recovery
50						
52				2		RQD = 68% 5.0/5.0 recovery
54						
56			Box 2	3		RQD = 63% 3.2/3.2 recovery
58						
		Hairline fractures common. Basalt clasts are 0.05"-3.0" long; clasts compose 60-90% of the rock; very little calcite in this zone.				10-7-83 10-8-83
60				4		RQD = 68% 55' 7:00 10-8-83
62		This zone has ~40% matrix of secondary minerals, ~60% primary basalt clasts. Slickensides common on fracture surfaces. More calcite filled fractures, matrix becoming strong. Clasts more completely altered.				
64						
			Box 3	5		RQD = 100% 5.2/5.2 recovery
66						
68						

Sheet 3 of 5

Project DESIGN UNIT A415 Date Drilled 10-6-9-83 Hole No. 31-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		39.0-100.1 <u>BASALT</u>: continued	Box 3	5	C	
70				6		RQD = 83% 4.8/5.0 recovery
72		72.1-72.9 open fractures to 1/2" wide, lines with quartz crystals up to 0.2" across; rock sheared, clay in shear zone				
74			Box 4			
76				7		RQD = 60% 4.9/4.9 recovery
78						
80		white to light green calcite coating fractures; clasts have reaction rims and are more highly altered; slickensides; core breaks into jagged, hackly fragments when struck with hammer		8		RQD = 92% 5.0/5.0 recovery
82						
84		grayish black, clay on fractured surfaces, breccia decreasing rock becoming basalt with secondary mineral filled fractures	Box 5			
86				9		RQD = 45% 4.6/4.6 recovery
88						10-8-83
				10		10-9-83 56' 7:10 10/9/83 RQD = 65% 1.7/1.7 recovery
90		basalt is serpentinized; slickensides are common	Box 6	11		
92						

Sheet 4 of 5

Project DESIGN UNIT A415 Date Drilled 10-6-9-83 Hole No. 31-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		39.0-100.1 BASALT: continued brecciated zone 92.0-93.0	Box 6	11	C	RQD = 84% 5.0/5.0 recovery
94		white quartz and calcite filling some fractures. Green secondary minerals can be scratched with finnger nail				
96		considerable mottling of secondary minerals, especially quartz, calcite		12		RQD = 96% 4.9/4.9 recovery
98		slickensides rare	Box 7			10-9-83
100		End of Boring 100.1'				Tremmied in two sacks cement to grout hole. Removed casing, backfilled with concrete to ground surface. Cleanded site.
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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

BORING LOG 31-4

Proj: DESIGN UNIT A415
Date Drilled 10-25/11-1-83
Ground Elev.
Drill Rig Failing 750
Logged By S. Slaff
Total Depth 100.3'
Hole Diameter NX
Hammer Weight & Fall 320 lbs,18" DR/140 lbs, 30" SS

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT			GB	
	ML	ALLUVIUM 0.4-3.2 CLAYEY SILT: moderate yellowish brown; trace gravel and sand; dry; stiff			AD	
2	(SM)	2.0 - becoming moist; increasing sand content	C-1		DR	1.0/1.0 recovery
	ML	3.2-10.8 SANDY SILT: moderate yellowish brown; moist; stiff; with gravel			AD	
4			J-1		SS	1.5/1.5 recovery
6					RD	
			C-2		DR	1.0/1.0 recovery
8					RD	
		becoming very stiff and gravelly	J-2		SS	1.2/1.5 recovery
10					RD	rig chatter
	GM	10.8-12.6 SILTY GRAVEL: mottled - moderate yellowish brown and grayish with sand, orange; moist; very dense;				
12	(ML)	becoming more silty	C-3		DR	1.0/1.0 recovery
	ML	12.6-21.3 SANDY SILT: moderate yellowish brown; with gravel; moist; hard			RD	
14			J-3		SS	1.0/1.5 recovery
16		becoming wet; color change to shade between moderate brown and moderate yellowish brown; gravel content increasing			RD	
			C-4		DR	1.0/1.0 recovery
18					RD	rig chatter
		becoming hard	J-4		SS	
20						

Sheet 1 of 5

Project DESIGN UNIT A415 Date Drilled 10-25/11-1-83 Hole No. 31-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	ML	12.6-21.3 SANDY SILT: continued	J-4		SS RD	1.1/1.4 recovery refusal at 17"
22		BEDROCK 21.3-100.3 BASALT: mottled - greenish black and dark gray; porphyritic with aphanitic to hemihyaline ground mass, fine grained phenorysts of plagioclase feldspar and pyroxeme. Secondary minerals include quartz filling vessicles and veinlets, chlorite, epidote with calcite coating fracture surfaces and less commly in vessicles, iron oxide coating fracture surfaces. Physical Condition: intensely to closely fractured; hard to moderately weathered. Rock consists of relatively fresh dark gray basalt clasts in a greenish black matrix of altered basalt. Clasts are 0.1-4.0" long, average 1.5" long. 22.3-22.8 core is disaggregated. Relatively fresh clasts comprise 30-40% of the formation. 36.5-41.4 relatively fresh clasts comprise 50-60% of formation.	C-5		DR C	0.25/0.25 recovery refusal at 3"
24			Box 1	1		RQD = 76% 4.2/4.2 recovery
26						
28				2		RQD = 84% 5.0/5.0 recovery
30						
32				3		RQD = 67% 5.0/5.0 recovery
34			Box 2			
36						10-25-83 10-26-83 ▼ 7.7'
38		Physical Condition: chlorite-filled fractures are 0.05-0.25" wide, average 0.1" wide		4		RQD = 58% 4.9/4.9 recovery
40		41.3-42.8 zone with softer matrix				4.6/4.6 recovery
42			Box 3	5		RQD = 27% recovered 0.3' of core from run 4
44		43.1-46.0 zone composed of 90% fresher basalt, 10% matrix				

Sheet 2 of 5

Project DESIGN UNIT A415 Date Drilled 10-25/11-1-83 Hole No. 31-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		21.3-100.3 BASALT: continued	Box 3	5	C	
		45.1-46.0 disaggregated zone				
46						
		medium dark gray pyrolucite (manganese dioxide) on some fracture surfaces				
48				6		RQD = 33% 5.0/5.0 recovery
		rare slickensides				drilling fluid is greenish gray
		~80% relatively fresh basalt clasts, ~20% altered matrix				
50		49.1-49.8, 51.4-55.8 zones of less resistant matrix, core darkens and loses much of its green cast after several hours of exposure to air.				
52			Box 4	7		RQD = 61% 4.8/4.8 recovery
54		rock breaks along hackly, uneven surfaces when struck with hammer				
56						10-26-83 10-31-83 ▼ 27.5
58		~90% relatively fresh basalt clasts, ~10% altered matrix		8		RQD = 72% 5.0/5.0 recovery
60			Box 5			
		brown iron oxide on some fracture surfaces				
62		~95% relatively fresh basalt, ~5% secondary minerals (mostly filling fractures). small fault, about of offset undeterminable, surfaces have slickensides in secondary minerals		9		RQD = 60% 5.0/5.0 recovery
64		75% relatiively fresh basalt, ~25% altered matrix				
66				10		RQD = 69% 4.9/4.9 recovery
68			Box 6			

Sheet 3 of 5

Project DESIGN UNIT A415 Date Drilled 10-25/11-1-83 Hole No. 31-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		21.3-100.3 BASALT: continued			C	
			Box 6	10		
70		slickensides on some fracture surfaces - oriented close to horizontal. relatively fresh basalt clasts are angular to subrounded, mostly subangular; size range is 0.1"-6.0", average 0.6"				
72						
				11		RQD = 84% 5.1/5.1 recovery
74						
76		greenish gray, microcrystalline calcite coating many fracture surfaces in layers 0.01-0.1" thick	Box 7			
78				12		RQD = 54% 4.8/4.8 recovery
80		anahedral, white, quartz filling cavities and veinlets 0.1-0.5" thick, average 0.1" thick				10-31-83 11-1-83 ▼ 28'
82		slickensides continue calcite and chlorite present on most fracture surfaces		13		RQD = 57% 4.9/4.9 recovery
84						
86		60% relatively fresh clasts, 40% altered matrix	Box 8			
88		minor very fine grained, euhedral pyrite grains on some fracture surfaces		14		RQD = 75% 4.8/4.8 recovery
90		alteration products coating fracture surfaces feel slippery	Box 9			
				15		
92						

Sheet 4 of 5

Project DESIGN UNIT A415 Date Drilled 10-25/11-1-83 Hole No. 31-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		21.3-100.3 BASALT: continued			C	
		70% relatively fresh clasts, 30% altered matrix		15		RQD = 90% 4.9/4.9 recovery
94			Box 9			
96		core tends to fracture around relatively fresh clasts when broken with hammer. pyrolucite coating some fracture surfaces		16		RQD = 88% 5.1/5.1 recovery
98			Box 10			
100						11-1-83
		END OF BORING 100.3'				Tremmied in 2 sack cement grout. Removed casing. Cleaned site. Covered hole with steel street cover. 11-6-83 removed street cover, capped with concrete.
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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 31-5

Proj: DESIGN UNIT A415
Date Drilled 10/9/83-10/19/83
Ground Elev. 574
Drill Rig Failing 750
Logged By Steve Slaff
Total Depth 150.0
Hole Diameter NX
Hammer Weight & Fall 140 lbs @ 30" SS; 320 lbs @ 18" DR

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.25 ASPHALT			GB	
		0.25-0.5 BASE ROCK			AD	
		ALLUVIUM 0.5-3.5 SANDY CLAY: grayish brown with gravel; dry, firm; minor organics; color change to moderate yellowish brown at 1.0				
2			C-1		DR	1.0/1.0 recovery
4	CL	3.5-7.0 GRAVELLY CLAY: moderate yellowish brown; moist, hard, with sand, minor organics			AD	
6			J-1		SS	1.5/1.5 recovery
		BEDROCK 7.0-150.0 BASALT: brownish black, fine grained			RD	
8		Physical Condition: intensely fractured (some fractures closed by secondary minerals), hardness-friable, strength-friable, deeply weathered, contains some clay.				
10			C-2		DR	refusal at 5"
		becoming harder, stronger			RD	0.3/0.3 recovery 9 October 1983 10 October 1983
12		Basalt: medium dark gray where little altered, greenish black where altered			C	
14		fine grained, includes plagioclase feldspar, pyroxene, chlorite, epidote, quartz. Minor unfilled vessicles. Altered zones with a waxy to dull luster.		1		RQD = 96% 4.2/4.5 recovery
16			Box #1			
18		Physical Condition: Basalt: intensely fractured, moderately hard to hard, moderately stront , moderately weathered. Altered Basalt: intensely fractured, low to friable hardness, weak strength, little to moderately weathered.		2		RQD = 100% 9.7/9.7 recovery
20						

Sheet 1 of 7

Project DESIGN UNIT A415 Date Drilled 10/9/83-10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20		7.0-150.0 BASALT (continued): fractures straight to uneven, rarely curved. Some fractures filled with quartz, chlorite, epidote, from hairline width to 0.2". Most fractures closed. Rock tends to break along hackly, uneven surfaces and along fractures.	Box #1		C	
22				2		9.7/9.7 recovery
24						
26		predominant fractures dip 50°-60° from horizontal	Box #2			
		grayish yellow green, very fine grained calcite coats some fracture surfaces				RQD = 100%
28						
30		rock is hard and strong where less altered				
				3		10.2/10.2 recovery
32		harder zone - higher proportion of fresh basalt to altered basalt				
		quartz, chlorite, epidote				
34			Box #3			
36						10 October 1983 11 October 1983
38		rock is ~90% altered basalt, 10% fresh basalt				
40		clasts of fresh basalt within altered basalt are 0.1"-8.0", have angular to rounded shape		4		RQD = 98% 8.3/8.3 recovery
42			Box #4			
44						

Sheet 2 of 7

Project DESIGN UNIT A415 Date Drilled 10/9/83-10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		7.0-150.0 BASALT (continued): most fractures closed or very narrow	Box #4	4	C	
46						RQD = 95%
48						
50				5		10.0/10.0 recovery
52		pyroxene and plagioclase feldspar in a very fine grained groundmass				
54		thin calcite coating fracture surfaces	Box #5			
56		quartz filled vessicles and fractures up to 1.0" wide				RQD = 79%
58				6		10.1/10.1 recovery
60						
62		predominant color: dark gray	Box #6			
64		Physical Condition: intensely to closely fractured, moderately hard to hard, moderately strong to strong, moderately weathered. Nearly all fractures closed, filled with secondary minerals. 2" wide quartz-filled cavity 67.5				11 October 1983 12 October 1983
66			Box #7	7		
68						

Sheet 3 of 7

Project DESIGN UNIT A415 Date Drilled 10/9/83-10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		7.0-150.0 BASALT (continued):			C	RQD = 100%
70						
			Box #7	7		10.0/10.0 recovery
72						
		slickensides in chlorite show relative movement from horizontal plane to various orientations up to 60° from horizontal				
74						
		chlorite and other alteration products coating fracture surfaces can be scratched with fingernail; have waxy luster and feel				
76						
				8		RQD = 95% 4.2/4.5 recovery
78						
80						
		abundant chlorite-filled veinlets up to 0.15" wide	Box #8	9		RQD = 62% 3.9/3.9 recovery
82						
		some vessicles only partially filled with secondary minerals; color change to dark greenish gray; very fine grained pyrite in matrix; pyrite more concentrated on fracture surfaces				
84						
86				10		RQD = 82% 4.6/4.6 recovery
		86.3-87.3 clay filled shear or fracture zone; abundant very fine to fine grained pyrite, dark greenish gray clay, horizontal slickensides				
88						
		calcite coating fracture surfaces in layers up to 0.1" thick	Box #9			
90				11		RQD = 65% 7.3/7.3 recovery
92						

Sheet 4 of 7

Project DESIGN UNIT A415 Date Drilled 10/9/83 to 10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		7.0-150.0 BASALT (continues as above): 92.0-93.6 shear zone with brecciated rock, sand, silt, clay in fractures	Box #9	11	C	
94						
96		pyrite decreasing				
						RQD = 93%
98			Box #10			
				12		9.5/9.5 recovery
100						
102						
104		tale on some fracture surfaces can be scratched with fingernail; quartz-filled cavities up to 0.4" wide				
106						RQD = 80%
			Box #11			
108						
				13		8.3/8.3 recovery
110						
112						
114		Physical Condition: intensely to closely fractured, moderately hard to hard, strong to moderately strong, moderately weathered	Box #12	14		7.6/7.6 recovery
116						

Project DESIGN UNIT A415 Date Drilled 10/9/83 to 10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116		7.0 to 150.0 BASALT (continued):			C	RQD = 98%
		Physical Condition (cont'd): nearly all fractures are filled with secondary minerals and closed				
118			Box #12	14		
120						
122						RQD = 87%
124			Box #13	15		10.2/10.2 recovery
126						
128						
130						
132		calcareous clay on some fracture surfaces, also calcite				RQD = 97%
134			Box #14			
		quartz-filled fractures		16		9.6/9.6 recovery
136						
138						
			Box #15			
140						

Sheet 6 of 7

Project DESIGN UNIT A415 Date Drilled 10/9/83 - 10/19/83 Hole No. 31-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
140		7.0-150.0 BASALT (continued):		16	C	
142		142.0 - 1" thick quartz-filled fracture				RQD = 96%
144		quartz-filled fractures up to 0.3" thick	Box #15	17		
146		open fractures coated with secondary minerals				8.7/8.7 recovery
148			Box #16			
150		END OF BORING 150.0'		18		0.5/0.5 recovery terminated hole at 150.0; set 2" diameter ABS piezometer from surface to 150.0; backfilled annulus with pea gravel; piezometer perforated from 110-150; set 5" PVC sleeve from ground surface to 2.0; covered with steel street cap; cleaned site
152						
154						
156						
158						
160						
162						
164						

Sheet 7 of 7

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 31-6

Proj: DESIGN UNIT A415
Date Drilled 2/24/84
Ground Elev.
Drill Rig Failing 1500
Logged By Mark Schluter
Total Depth 29.0'
Hole Diameter 4 7/8"
Hammer Weight & Fall 325 lb @ 18", 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.3 A.C. PAVEMENT			C	
	SM	FILL 0.3-6.0 SILTY SAND: moderate brown; loose; slightly moist to moist; trace gravel; trace of brick; asphalt and rootlets			A	
2						
			C-1		DR 325	
4					RD	
6	SM/ML	ALLUVIUM 6.0-8.5 SILTY SAND/SANDY SILT: moderate brown to dark yellowish brown; medium dense/stiff; moist to slightly moist; trace gravel	J-1		SS 140	0.5/1.5 recovery
8						
	GM	8.5-13.5 SANDY GRAVEL/SILTY GRAVEL: dusky yellowish brown; medium dense; moist	C-2		DR 325	
10					RD	variable rig chatter
12						
	SM	13.5-18.0 SILTY SAND: moderate brown; medium brown; medium dense; moist with gravel	C-3		DR 325	
14					RD	
16			J-2		SS 140	0.6/1.5 recovery
18	SW	18.0-22.5 SAND: moderate brown to dusky yellowish brown; trace fines and gravel; medium dense; increasing gravel content			RD	
			C-4		DR 325	
20						

Sheet 1 of 2

Project DESIGN UNIT A415 Date Drilled 2/24/84 Hole No. 31-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	SW	18.0-22.5 SAND (continued):			RD	
22		BEDROCK 22.5-29.0 BASALT: dusky brown and grayish black; aphanitic to fine grain	C-5		DR 325	drill rig chatter refusal @ 11"
24		Physical Condition: intensely fractured; moderately to deeply weathered; soft	C-6		RD DR 325	refusal @ 6"
26		25.0-28.0 moderately weathered; intensely fractured; moderately hard, narrow to very narrow fracture walls with clay filling			RD	significant drilling fluid loss in basalt formation
28		moderately to deeply weathered; intensely fractured	C-7		DR 325	
		END OF BORING 29.0'				
30		Passive Percolation Test: hole depth 29.0', 5" I.D. casing 13" above ground surface				
32		water level fell 2.6' inside steel casing during 1 minute				
34		water level fell 10.8' below top of casing after 10 minutes				
		2-25-84 water level 26.5' below ground surface				
36						
38						
40						
42						
44						

Sheet 2 of 2

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 31-7

Proj: DESIGN UNIT A415 **Date Drilled** 2-25-84 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** M. Schluter **Total Depth** 20.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.5 A.C. PAVEMENT: multiple layers			C	started drilling @ 0730
	SM	0.5-3.0 SILTY SAND: moderate brown; loose-medium dense; moist; clay binder			A	
2						
4		BEDROCK 3.0-20.0 BASALT: brownish black to dusky brown Physical Condition: intensely fractured, closed-very narrow fractured walls, stained, medium-weathered, moderately hard	C-1		DR	
					A	
			C-2		DR	
6		5.0-10.0 very soft to soft, (soil like), highly weathered, clay infilling 1-10 mm. infilling dark yellowish brown			A	
			C-3		DR	
8					RD	drill rig chatter
10		10.0-15.0 degree of weathering variable, highly weathered - medium weathered, medium - highly fractured, random, clay infillings 1-8 mm	C-4		DR	
12					RD	varible drill rig chatter
14						
		15.0-20.0 medium weathered, fresh, hard to very hard rock, highly fractured jagged edges, secondary mineral infilling - chloriate, epidote	C-5		DR	
16						caving and sluffing into hole, added additional bentonite variable drill rig chatter
18						heavy drill rig chatter
		END OF BORING 20.0', finished @ 1100	C-6			
20						

Sheet 1 of 1

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

BORING LOG 32-1

Proj: DESIGN UNIT A415
Date Drilled 1-24/2-24-84
Ground Elev. 865.5'
Drill Rig Failing 250
Logged By D. Gillette
Total Depth 442.0'
Hole Diameter NX
Hammer Weight & Fall N/A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		BEDROCK 0.0-37.0 WEATHERED BASALT: dusky brown to brownish black with dusky yellowish brown streaks (iron oxide stains) contains clay deposits in fractures and exhibits feldspars weathering to clay			A	
2						
					RD	
4			PB-1		PB	
6						
					RD	1.5/2.5 recovery
		Physical Condition: closely to intensely fractured, soft to friable hardness, friable strength, moderate to deeply weathered	Box 1		C	
8						
				1		4.8/5.0 recovery
10						
12						
				2		3.0/5.0 recovery
14						
		14.5-17.0 intensely fractured				
16						
				3		5.0/7.0 recovery
18						
20						

Sheet 1 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20		0.0-37.0 WEATHERED BASALT: continued as previously described	Box 2	3	C	5.0/7.0 recovery
22						
24						
26				4		
28						
30					RD	1-24-84 1-25-84
32					C	
34				5		3.6/8.0 recovery
36						
38		37.0-155.0 BASALT: brownish black, greenish black and dark greenish gray, vesicular				
40		40.0-60.0 minor iron oxide stains along fractures and joint (healed)	Box 3	6		4.9/5.0 recovery
42						
44						

Sheet 2 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44		37.0-155.0 BASALT: continued Physical Condition: moderate to little fractured; moderately hard to hard; moderately strong to strong, little weathered to fresh	Box	6	C	
46						
		47.0-48.0 shear zone				
48				7		7.0/8.0 recovery
50		vesicule basalt fragments set in a green ground massl quartz stringers present; fractures show $CaCO_3$ infilling	Box 4			
52						
54						
56				8		8.0/8.0 recovery
58						
60		40.0-60.0 minor iron stains along fractures and joints (healed)	Box 5			
						61.2 shear zone
62				9		5.0/5.0 recovery
64						
66				10		6.0/6.0 recovery
68						

Sheet 3 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		37.0-155.0 BASALT: continued as previously described	Box 5	10	C	6.0/6.0 recovery
			Box 6			
70		70.0-71.0 fracture zone				
72						
74				11		5.0/5.0 recovery
						1-27-84
		75.0 healed joint, $CaCO_3$				1-28-84
76						
78		78.0-83.0 shear zone, iron oxide staining, contains fault gauge	Box 7			
80				12		7.5/9.0 recovery
82						
84						
86		86.0-88.0 fracture zone		13		very hard drilling
88						
90				14		
92			Box 8			

Sheet 4 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		37.0- 155.0 BASALT: continued 93.0-102.0 zone of vertical fractures - 1/8" - $CaCO_3$ healed	Box 8	14	C	7.0/7.0 recovery 1-28-84
94						1-29-84
96				15		4.0/4.0 recovery
98				16		4.0/4.0 recovery
100			Box 9			
102						
		103.0 slickensided surface				
104						
106		106.0-107.0 randomly oriented fractures and joints		17		9.9/10.0 recovery
108			Box 10			
110						
		111.0-112.0 shear zone				
112						
114				18		
116						

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116		37.0-155.0 BASALT: continued	Box 10		C	
118		118.0-119.0 slickensides	Box 11	19		9.9/10.0 recovery
120		120.0-121.- vertical fractures				
						1-29-84 1-30-84
122						
		123.0-124.0 shear zone with $CaCO_3$ fault gouge				
124						
126			Box 12	20		10.0/10.0 recovery
128						
130						
132						
134						
136				21		10.0/10.0 recovery
			Box 13			
138		138.0-140.0 fracture zone, hackly surfaces, healed $CaCO_3$				
140						

Sheet 6 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
140		37.0-155.0 BASALT: continued 140.0-141.0 shear zone	Box 13	21	C	
142						
144			Box 14			
		145.0-146.0 fracture zone				
146				22		10.0/10.0 recovery
148		148.0-149.0 fracture zone, random orientation, $CaCO_3$				
150						
152						
154			Box 15	23		7.0/7.0 recovery
	40°	155.0-169.0 SHALE: medium gray and dark gray Physical Condition: (155-169) moderately fractured, low hardness, moderate strength, little weathered				
156						
158						1-30-84
		Physical Appearance: well indurated with massive bedding				1-31-84
160						
162						
164						

Sheet 7 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
164		155.0-169.0 SHALE: continued	Box 16	24	C	8.5/10.0 recovery
166						
168						
		169.0-173.4 BASALT: brownish black, greenish black and dark greenish gray, vesicular				
170						
		171.0-179.0 fracture zone, randomly oriented fractures and joints, intensely fractured, well healed				
172				25		9.0/10.0 recovery
		173.4-179.0 SHALE: medium gray and dark gray	Box 17			
174						
		175.0 slickensides				
176						
178						
		179.0-442.0 BASALT: brownish black, greenish black and dark greenish gray, vesicular Physical Condition: closely to moderately fractured, well healed (closed), moderately hard to hard, moderately strong to strong, little weathered to fresh				
180				26		6.9/7.0 recovery
182						
184						
			Box 18			1-31-84
186						2-1-84
		187.0-188.0 fractured zone		27		
188						

Sheet 8 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
188		179.0-442.0 BASALT: continued	Box 18		C	
190		190.0-193.0 fracture zone, $CaCO_3$ infilling, hackly surface		27		10.0/10.0 recovery
192						
194			Box 19			
		195.0 slickensides				
196						
198						
200				28		9.5/10.0 recovery
202						
204		203.5-204.5 fracture zone, intensely fractured	Box 20			
206						
208				29		9.0/10.0 recovery
210						
		211.0-212.0 fracture zone, intensely fractured				
212						

Sheet 9 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
212		179.0-442.0 BASALT: continued	Box 21		C	
214				29		9.0/10.0 recovery
216		216.0-217.0 fractured zone, slickensides				2-1-84 2-7-84
218						
220				30		5.5/8.0 recovery
		221.0-224.0 fractured zone, closely fractured, well healed (closed)				
222						
224			Box 22			
226						
228				31		5.1/8.0 recovery
230						
232						
		233.0-234.0 fracture zone, slickensides		32		
234						
236						

Sheet 10 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
236		179.0-442.0 <u>BASALT</u>: continued	Box 22	32	C	5.2/7.0 recovery
238			Box 23			2-7-84
						2-8-84
240						
242						
		243.0-244.0 fracture zone		33		9.0/10.0 recovery
244						
246			Box 24			
248						
250						
		251.5-252.5 fracture zone, slickensides, closely fractured, well healed (closed)				
252				34		9.3/10.0 recovery
254						
256			Box 25			
258						2-8-84
				35		2-9-84
260						

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
260		179.0-442.0 BASALT: continued	Box 25		C	
262						
				35		9.0/10.0 recovery
264		264.5-265.0 fracture zone	Box 26			
266						
268						268-269 very hard drilling
270						
272						
274			Box 27	36		10.0/10.0 recovery
276						
278						
		279.0 slickensides				
280						
				37		4.0/4.0 recovery
282		282.0 CaCO$_3$ on fracture surfaces				2-9-84
			Box 28	38		2-10-84
284						

Sheet 12 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
284		179.0-442.0 BASALT: continued	Box 28		C	
		285.5-289.0 shear zone, trace clay, slickensides, intensely fractured				
286						
				38		8.5/10.0 recovery
288						
290						
			Box 29			
292						
294						
				39		10.0/10.0 recovery
296		296.0 hackly surface				
298						
		299.5-300.5 fracture zone, closely fractured, healed				
300		Physical Condition (300'-360'): moderately fractured, well healed (closed)				
302			Box 30			2-10-84
						2-11-84
304				40		
306		306.0-308.0 fracture zone, intensely fractured				
308						

Sheet 13 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
308		179.0-442.0 BASALT: continued	Box 30		C	
310				40		9.0/10.0 recovery
		311.0-312.0 fracture zone				
312			Box 31			
314		314.0-315.0 fracture zone				
316				41		10.0/10.0 recovery
318						
320						
322			Box 32			2-11-84 2-12-84
324						
326				42		7.8/8.0 recovery
328						
330						
332				43		

Sheet 14 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
332		179.0-442.0 <u>BASALT</u>: continued	Box 33		C	
		331.5-333.0 fracture zone				
334						
336				43		10.0/10.0 recovery
338						
340						2-12-84 2-13-84
342						
344			Box 34	44		10.0/10.0 recovery
346						
348						
350						
352						
354			Box 35	45		
356						

Sheet 15 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
356		179.0-442.0 BASALT: continued	Box 35		C	
				45		9.9/10.0 recovery
358						
		359.0-371.0 fracture zone Physical Condition (359-370): intensely fractured, well healed (closed)				
360						2-13-84 2-14-84
362						
364			Box 36			
				46		9.3/10.0 recovery
366						
368		Physical Condition (370-424): closely fractured, well healed (closed)				
370						
372						
374			Box 37			
		375.0 slickensides		47		7.0/8.0 recovery
376		378.0-378.5 facture zone, closely fractured				
378						2-14-84 2-15-84
				48		
380						

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
380		179.0-442.0 <u>BASALT</u>: continued	Box 37		C	
		381.0-382.5 fracture zone, intensely fractured, well healed (closed)				
382				48		9.0/10.0 recovery
384			Box 38			
386		386.0 slickensides or hackly surface				
388						
390						
392			Box 39	49		9.0/10.0 recovery
394						
396						
398						2-15-84 2-16-84
400				50		
402			Box 40			
404						

Sheet 17 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
404		179.0-442.0 BASALT: continued 404.5-416.6 fracture zone	Box 40		C	
406				50		9.1/10.0 recovery
408						2-16-84 2-21-84
410						
412			Box 41	51		9.5/10.0 recovery
414						
		415.0 slickensides				
416						
418		418.0-419.0 fracture zone				
420		420.0-424.0 facture zone				
422			Box 42	52		9.0/10.0 recovery
424		Physical Condition (424'-442'): intensely fractured, well healed (closed)				
426		426.0-429.0 fracture zone				
428						

Sheet 18 of 19

Project DESIGN UNIT A415 Date Drilled 1-24/2-24-84 Hole No. 32-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
428		179.0-442.0 BASALT: continued	Box 42	52	C	2-21-84 2-22-84
430						
432			Box 43	53		7.5/8.0 recovery
434						
436		436.5-437.1 fracture zone				
438				54		4.6/5.0 recovery
450		440.0-442.0 fracture zone	44			
440		END OF BORING 442.0'				
442						
444						
446						
448						
452						

Sheet 19 of 19

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-31 for Design Unit A415. Measurements were made at 5-foot intervals from the ground surface to depths of 115 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-31.

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 Figure 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*). Vp* 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 to a depth of 30 feet was found to be approximately 1270 feet per second. To depths of about 115 feet, shear wave velocity estimates generally increased to 4800 feet per second.

B.2 SEISMIC REFRACTION SURVEY

B.2.1 Summary

Seven seismic refraction lines were recorded in the vicinity of Hollywood Bowl during the months of February and March, 1981 at the locations shown on Figure B-3. The purpose of these lines was to delineate the alluvium/bedrock interface to evaluate evidence for fault offset in the area.

Seismic readings were recorded in both forward and reverse directions along all lines. Profiles showing subsurface velocity zones were constructed from interpretations of the data, and are presented in Figures B-4 through B-6.

A map showing the locations of the seismic refraction lines is presented on Figure B-3 of this Appendix.

Interpreted results indicate that Lines S-31, S-32, S-51 and S-52 were recorded in areas of near-surface weathered rock, and Lines S-35, S-36 and S-44 were recorded in areas underlain by alluvial deposits. The ground water table was observed at depths of 24 to 40 feet in the near-surface weathered rock areas and 8 to 34 feet beneath the ground surface in the alluvial areas surveyed. The only significant seismic refraction anomaly indicative of possible fault offset was observed beneath the northwest end of Line S-51.

B.2.2 Detailed Description

Seismic refraction Lines S-31, S-51 and S-52 were recorded in the more hilly portions of the Hollywood Bowl area.

As shown on the subsurface velocity profiles of Figures B-4 and B-5, the area beneath these lines is underlain by low velocity material (1,000 to 1,250 ft/sec) to depths of 3 to 14 feet beneath the ground surface. The low velocity zone is underlain by low to medium velocity material (2,160 to 2,830 ft/sec) to depths of 24 to 40 feet where medium velocity material (4,260 to 5,000 ft/sec) is encountered. The medium velocity zone extends at least to depths of 40 to 80 feet beneath Lines S-51 and S-52 and is underlain by high velocity material (12,120 to 12,990 ft/sec) at depths of 62 to 103 feet beneath Lines S-31 and S-32. The high velocity zone beneath Lines S-31 and S-32 extends to the maximum depth-limit of information obtained (about 105 feet).

The low velocity material is interpreted to represent residual soil and colluvial deposits. The low to medium velocity zone represents weathered bedrock, and the medium velocity zone represents saturated weathered bedrock. The high velocity zone is interpreted to represent competent bedrock. A vertical step anomaly with the northwest side up was observed beneath the northwest end of Line S-51.

Seismic refraction Lines S-35, S-36 and S-44 were recorded in the alluvial portion of the Hollywood Bowl area.

As presented on the cross sections of Figures B-5 and B-6, the area beneath these lines is underlain by low velocity material (970 to 1,330 ft/sec) to depths of 8 to 34 feet beneath the ground surface. Medium velocity material

(5,000 to 6,150 ft/sec) underlies the low velocity zone and extends to the depth-limit of information obtained (40 to 50 feet beneath Lines S-36 and S-44, and to depths of 66 to 85 feet beneath Line S-35. High velocity material (10,000 ft/sec) underlies the medium velocity zone beneath Line S-35 to the maximum depth-limit of information obtained (about 85 feet).

The low velocity zone is interpreted to represent unconsolidated alluvial deposits and fill. The medium velocity zone is interpreted to represent saturated alluvial deposits (and perhaps weathered bedrock beneath Line S-44). The high velocity zone represents competent bedrock.

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*
31	15- 30	3922	1253	196	5	3922±1450	1273	333	64	4	1270±400
	30-115	8788	1195	439	17	8790±1630	4842	190	240	18	4840±215

$\bar{V}p$ = 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.

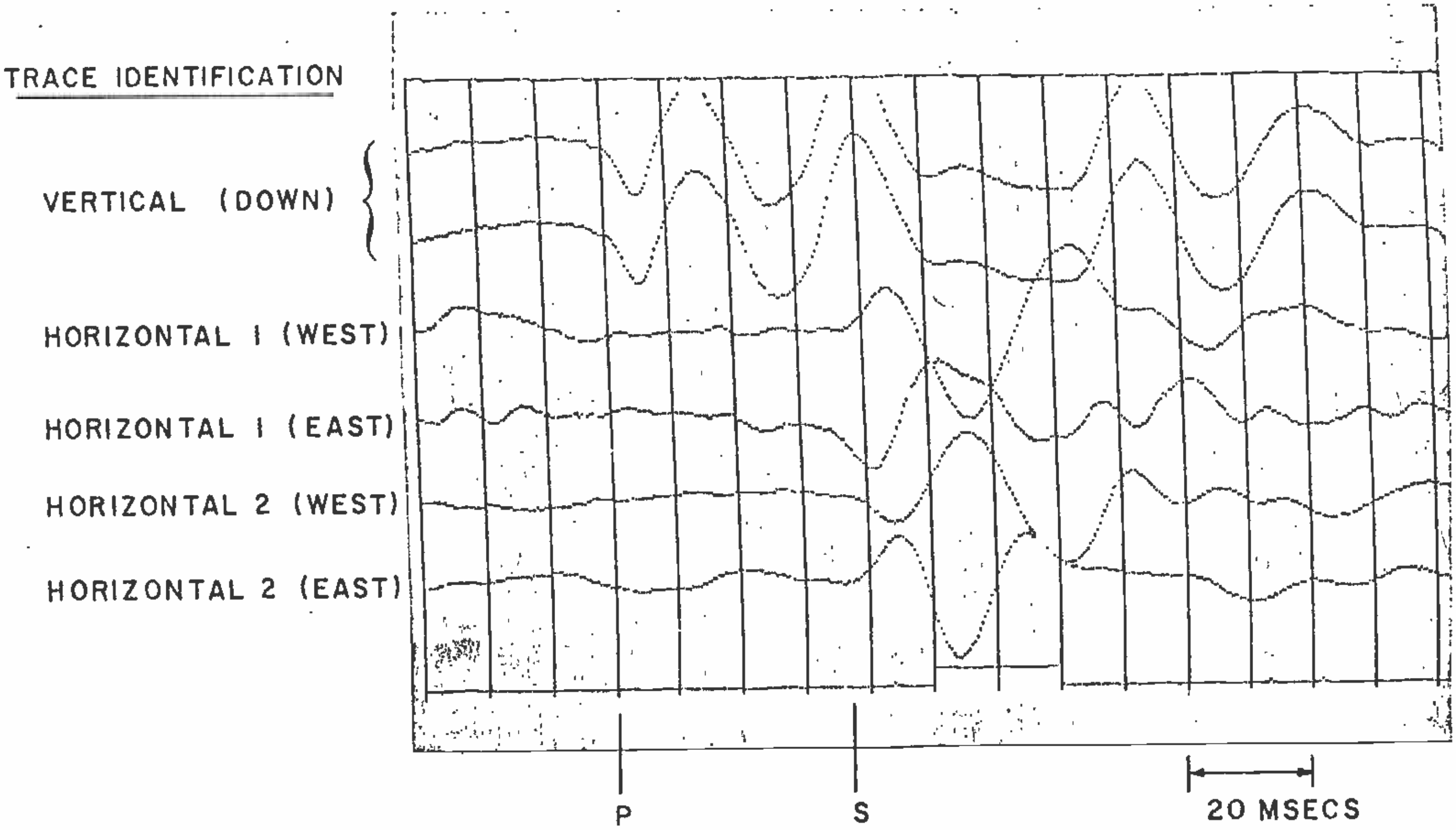
Np = number of points used for straight line fit of compressional wave.

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



BOREHOLE: 13

DEPTH: 70 FT

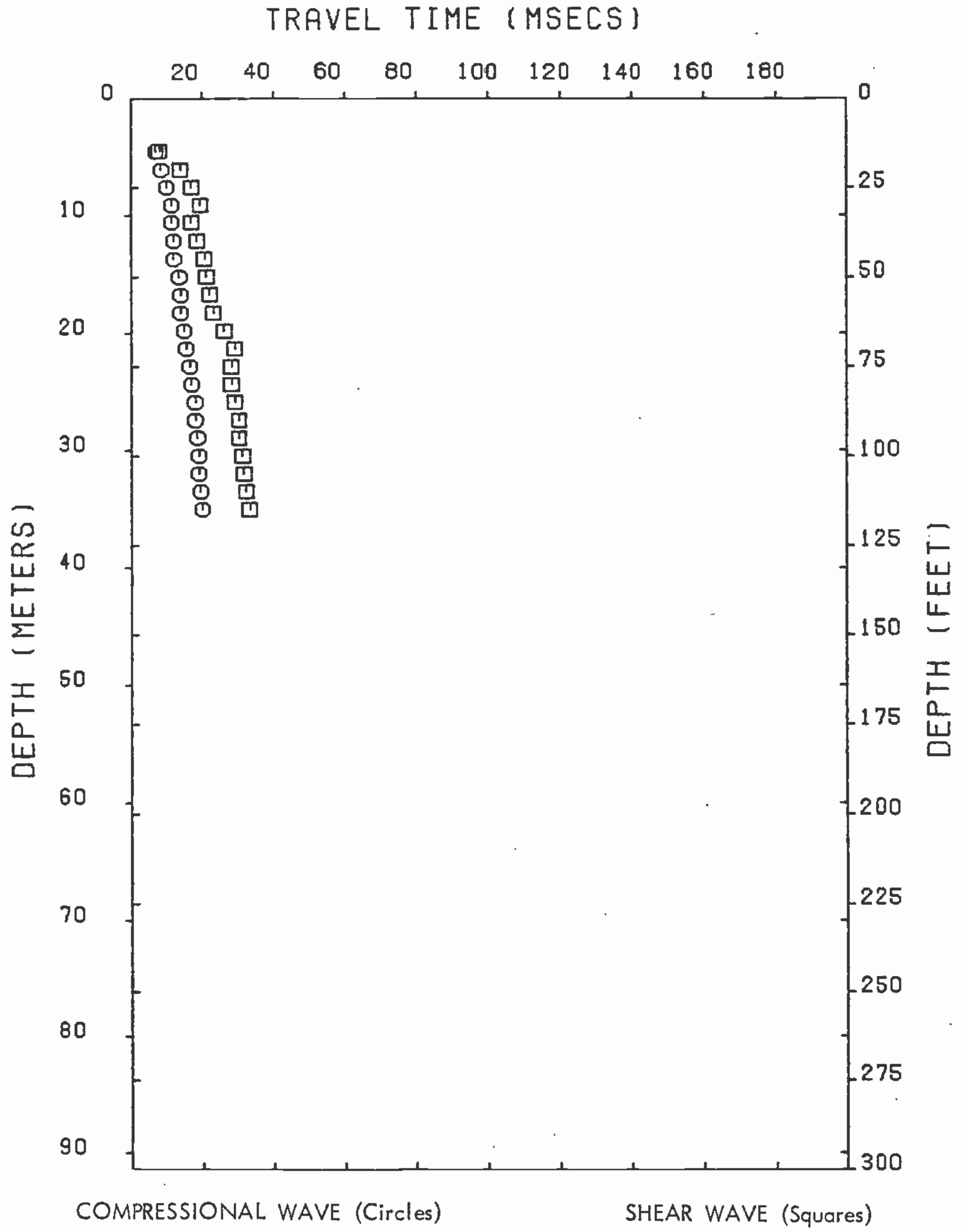
DOWNHOLE SAMPLE RECORD

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Project No. 83-1140

Figure No. B-1



DOWNHOLE TRAVEL TIME PROFILE - BORING 31

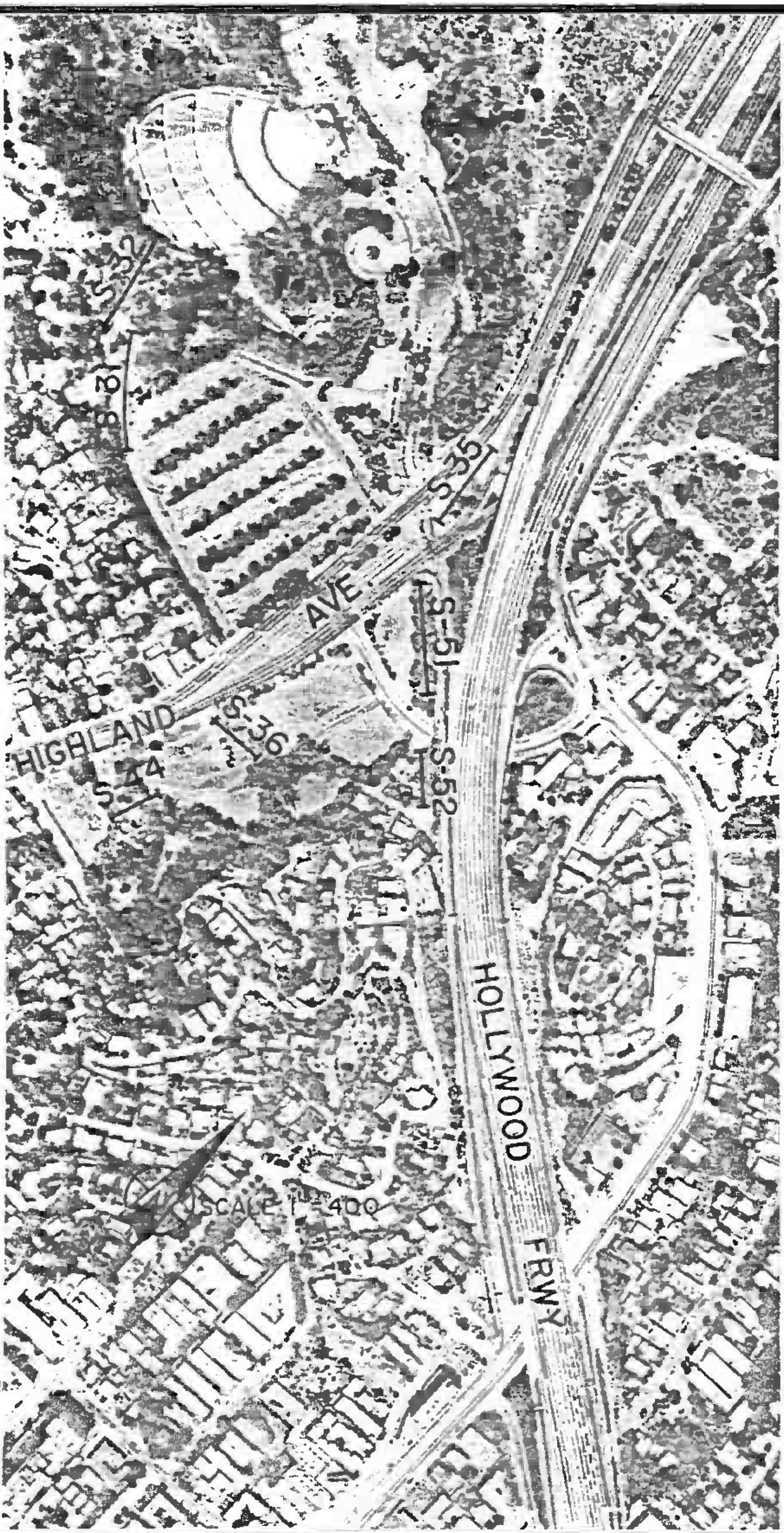
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B-2

Approved for publication 5/84 by



Approved for publication 5/84 by JD

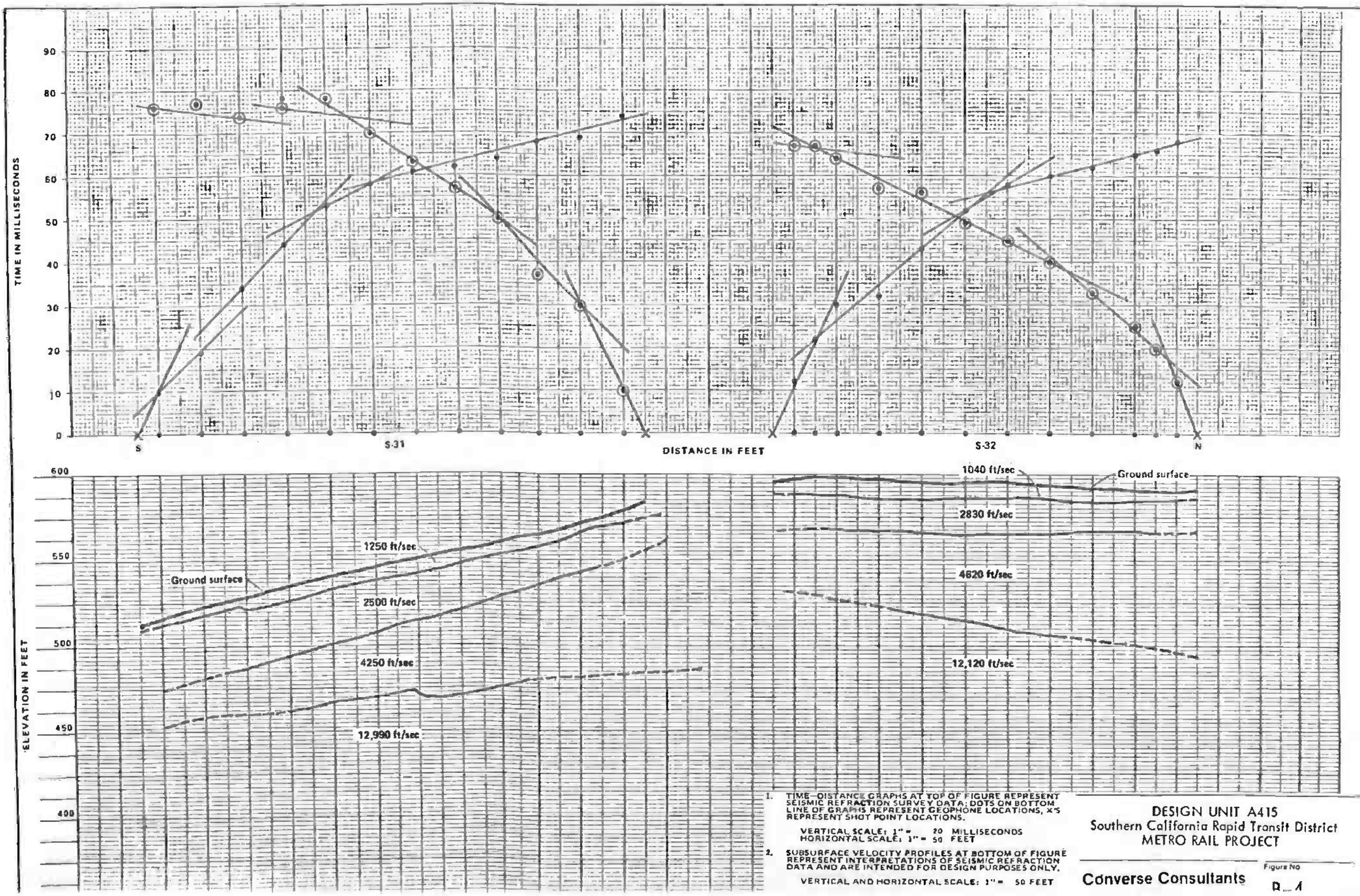
SEISMIC REFRACTION SURVEY - AREA 5

DESIGN UNIT A250
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B-3



1. TIME—DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

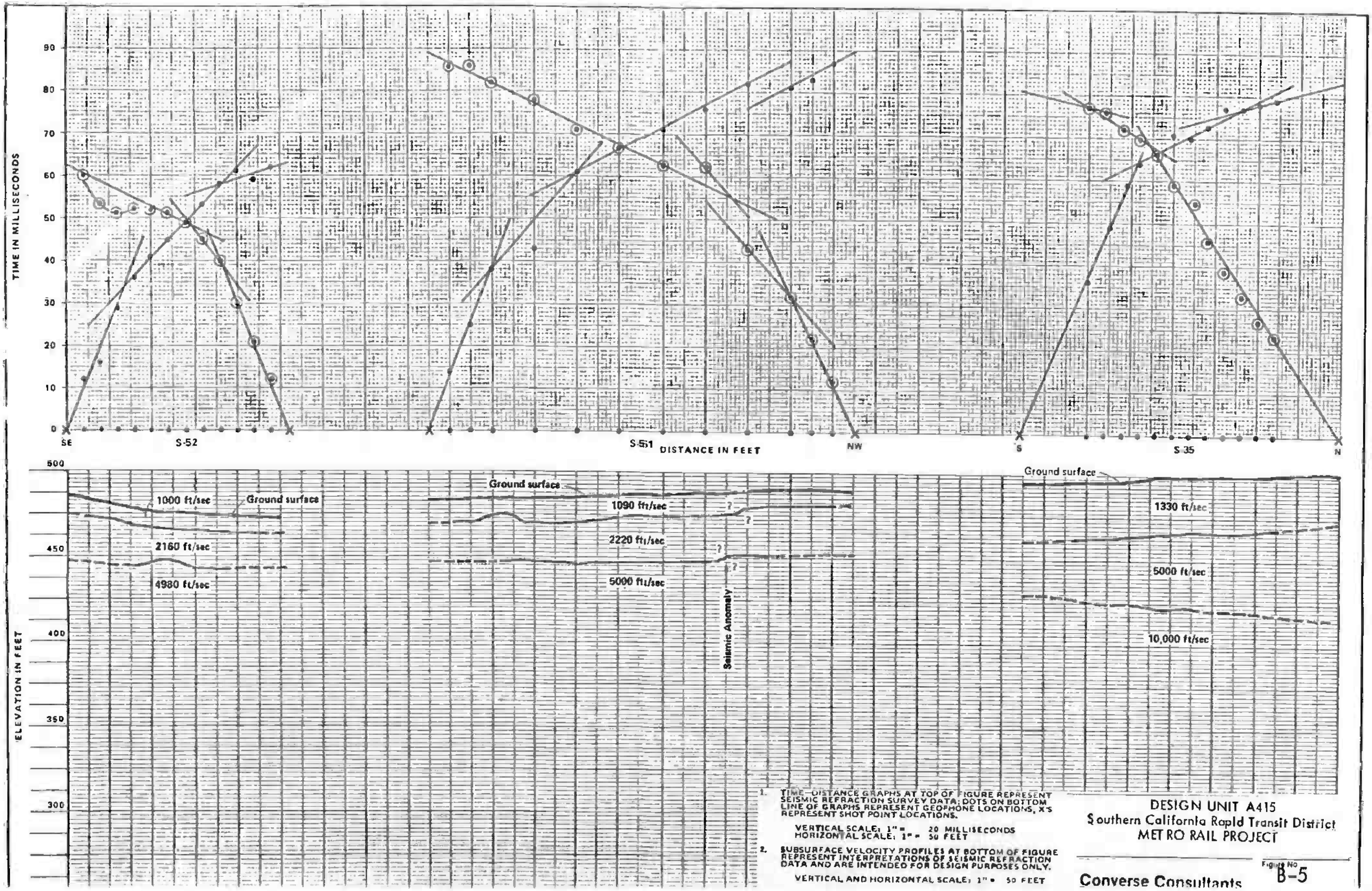
2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants

Figure No
B-4



1. TIME–DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

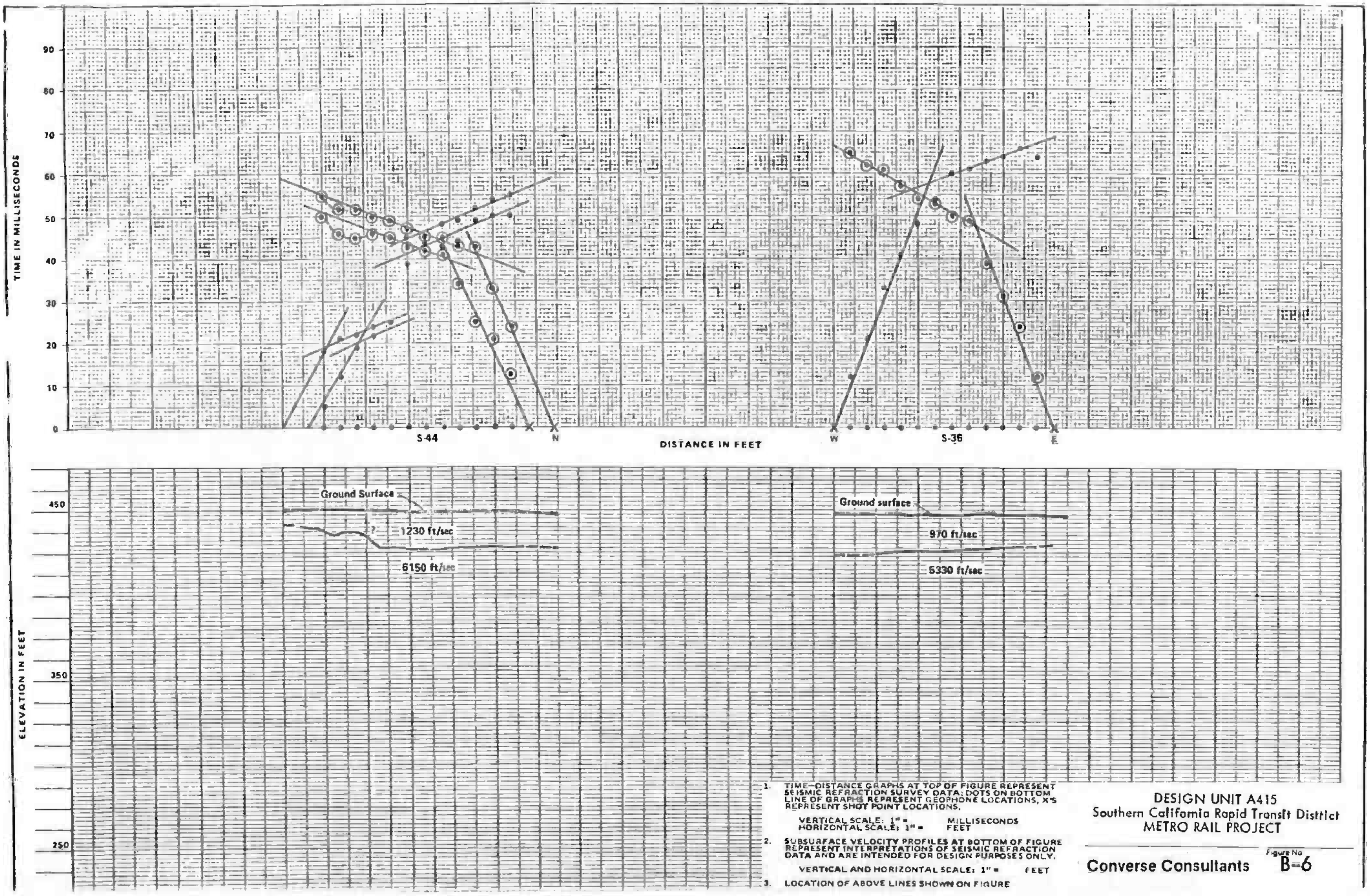
2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants

Figure No
B-5



1. TIME—DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = MILLISECONDS
   HORIZONTAL SCALE: 1" = FEET

2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = FEET

3. LOCATION OF ABOVE LINES SHOWN ON FIGURE

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants

Figure No
B-6

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 Bowl 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 triaxial compression, direct shear, consolidation and permeability.

The laboratory test data from the present investigation are presented in Table C-1. The geologic units listed in these tables are described in Section 5.0 of the report. Figures C-1 through C-5 summarize strength and modulus data for coarse-grained alluvium and the weathered 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-6 through C-8.

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 Figure C-9 and Table C-1.

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 a modified version of the ASTM D-2261 test method. Test results are presented on Table C-1.

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 Table C-1.

C.3 ENGINEERING PROPERTIES: STATIC

C.3.1 Triaxial Compression

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

C.3.2 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-10 and C-11.

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 Table C-1 and Figure C-3.

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 Table C-1 and Figure C-3.

C.3.4 Unconfined Compression

Unconfined compression tests were performed on selected samples of the basalt bedrock from the test borings for the purpose of evaluating the unconfined strength of the basalt. Results of the unconfined compression tests are presented on Table C-2 and on Figure C-5.

C.3.5 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 Figure C-12.

C.3.6 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 Table C-1.

TABLE C-1 LABORATORY TEST DATA FOR ALLUVIUM

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		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
31-2	1	3	Sandy Silt	A_2	89	14	43	11										
	2	8	Silty Sand	A_1	92	23					29	0.20			X	X		
	3	13	Sandy Silt	A_2	95	21					32.5	0.45					X	
	4	18	Silty Sand	A_1	99	22									X	X		X(3)
	5	23	Silty Sand	A_1	104	20			3.7×10^{-4}		45	1.10			X			
31-3	J1	8	Sandy Silt	A_2	-	22									X			
	J2	16	Sandy Silt	A_2	-	25	38	8							X	X		
	J3	27	Sandy Silt	A_2	-	28	39	11							X	X		
	J4	34	Silty Sand	A_1	-	24									X			
31-4	1	3	Silty Sand	A_1	98	15	33	3			46	0.24			X	X		
	2	8	Sandy Silt	A_2	104	16	NP				39.5	0.75					X	
	3	13	Sandy Silt	A_2	116	15					45.5	0.75					X	
	4	18	Sandy Silt	A_2	98	27									X	X		X(3)
	5	22	-Disturbed-		-	-												

TABLE C-1 LABORATORY TEST DATA FOR ALLUVIUM

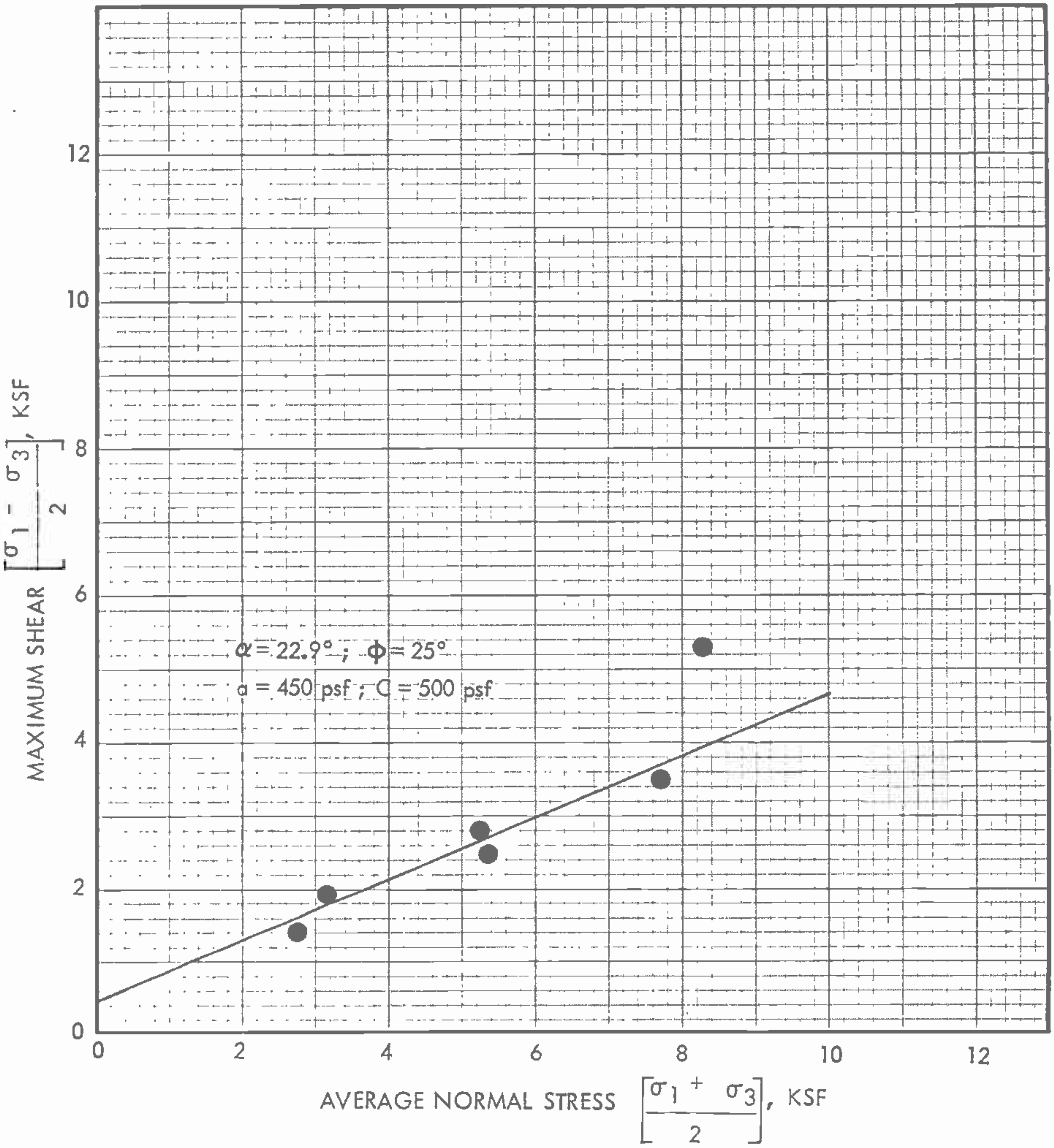
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
31-6	1	3	Silty Sand	A_1	93	16												
	2	9	Sandy Gravel	A_1	109	17												
	3	13	Silty Sand	A_1	109	10												
	4	19	Silty Sand	A_1	103	15												

TABLE C-2 - UNCONFINED COMPRESSION TESTS ON BASALT NX CORES

BORING No.	DEPTH (ft)	UNCONFINED COMPRESSIVE STRENGTH (ksf)
31-2	43	211
	54	114
	65	256
	68	240
	71	244
31-3	56	69
	61	122
	65	320
	67	268
	70	244
	73	142
	74	268
	76	49
	79	365
31-4	68	179
	69	130
	74	207
	76	61
	80	65
	84	61
	87	146
	89	284
	93	106
	96	256
31-5	102	284
	104	142
	107	414
	110	402
	117	532
	120	361
	125	581

NOTES: 1) Data from Triaxial Compression Tests

2) $\tan \alpha = \sin \phi$



Approved for publication 5/1/11 by

SUMMARY OF TOTAL STRENGTH DATA - ALLUVIUM

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

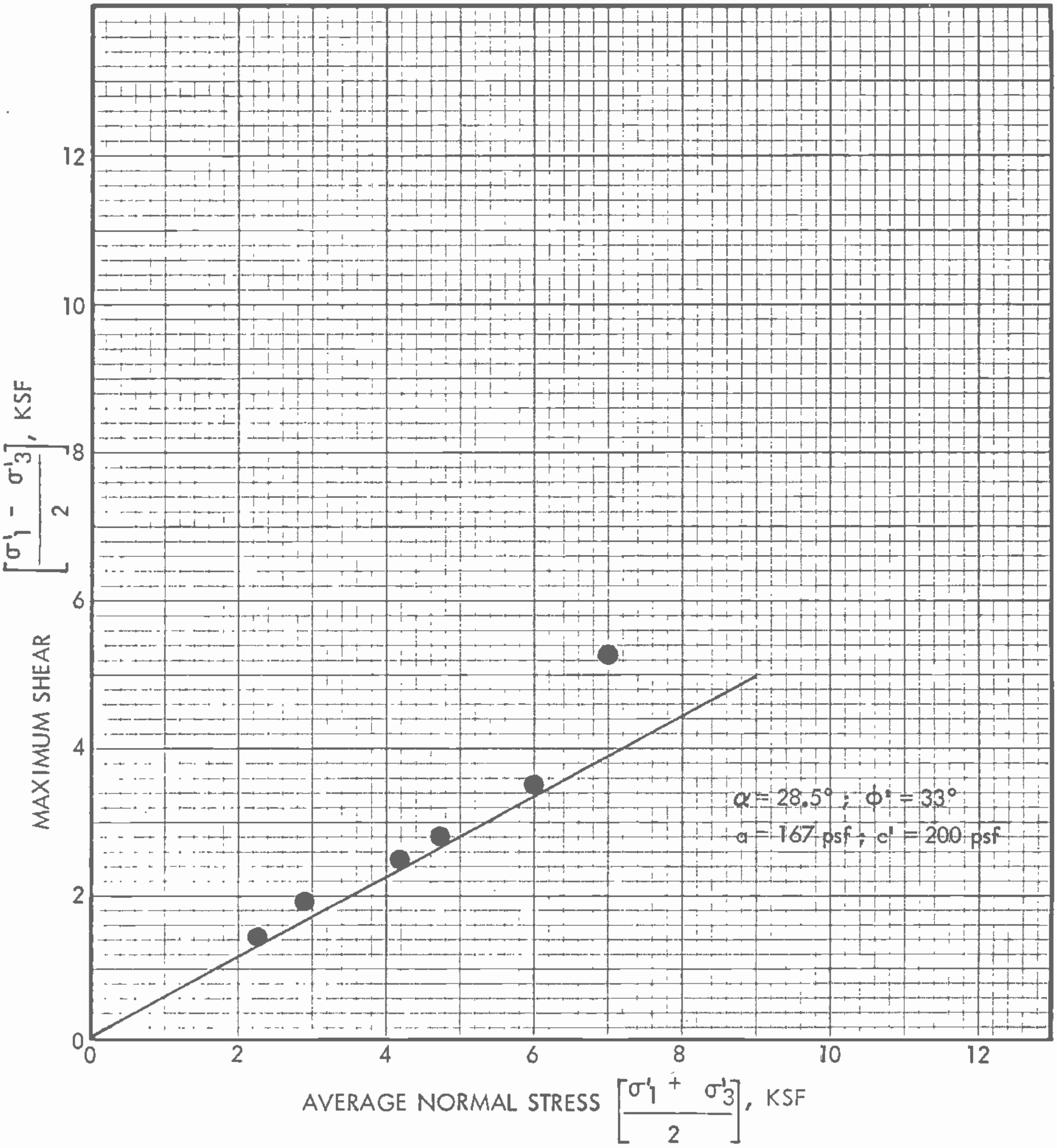
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-1

NOTES: 1) Data from Triaxial Compression Tests

2) $\tan \alpha = \sin \phi$



Approved for publication 5/84 by

SUMMARY OF EFFECTIVE STRENGTH DATA - ALLUVIUM

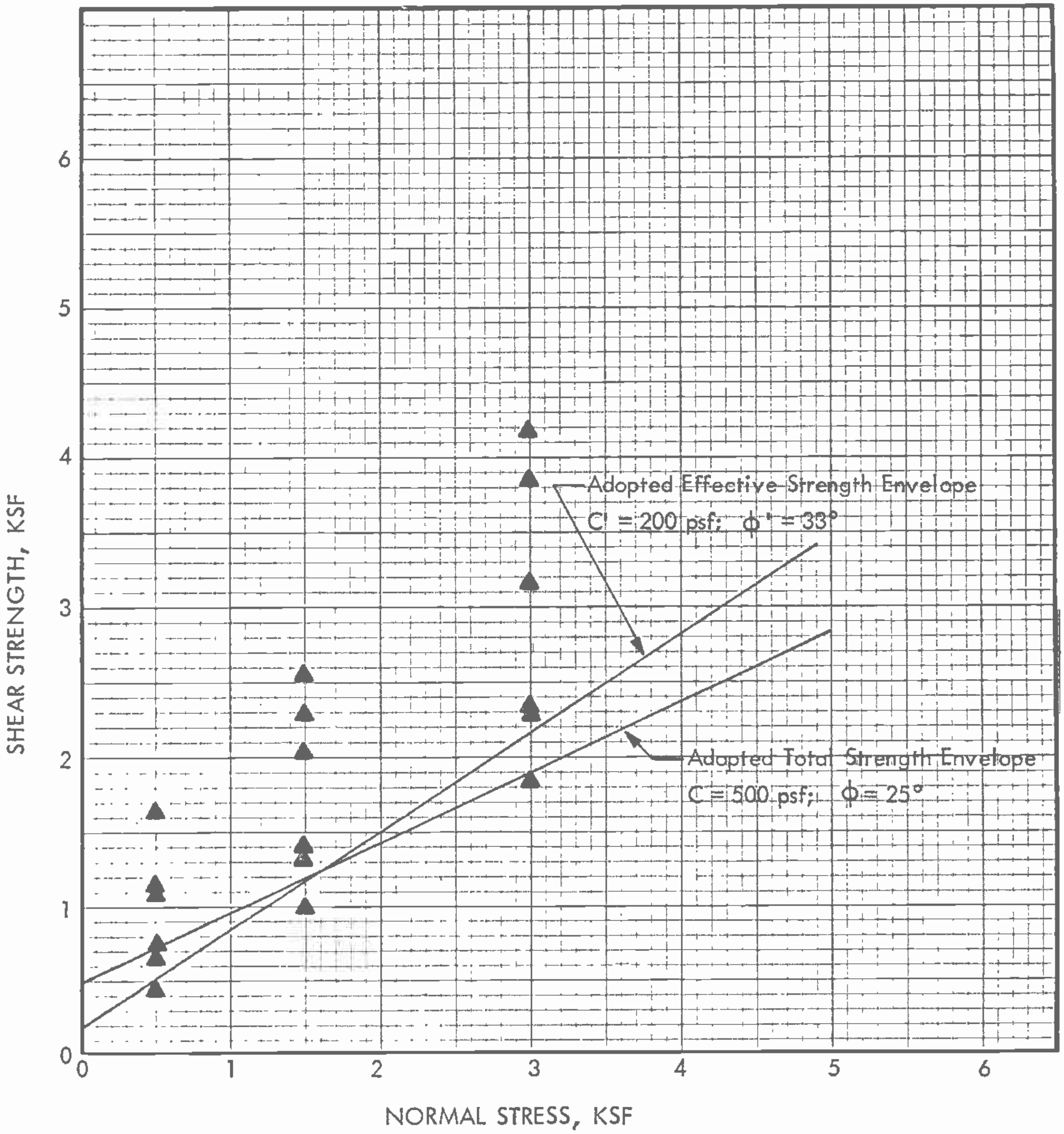
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-2

Approved for publication 5/14 by JAD



DIRECT SHEAR TEST DATA - ALLUVIUM

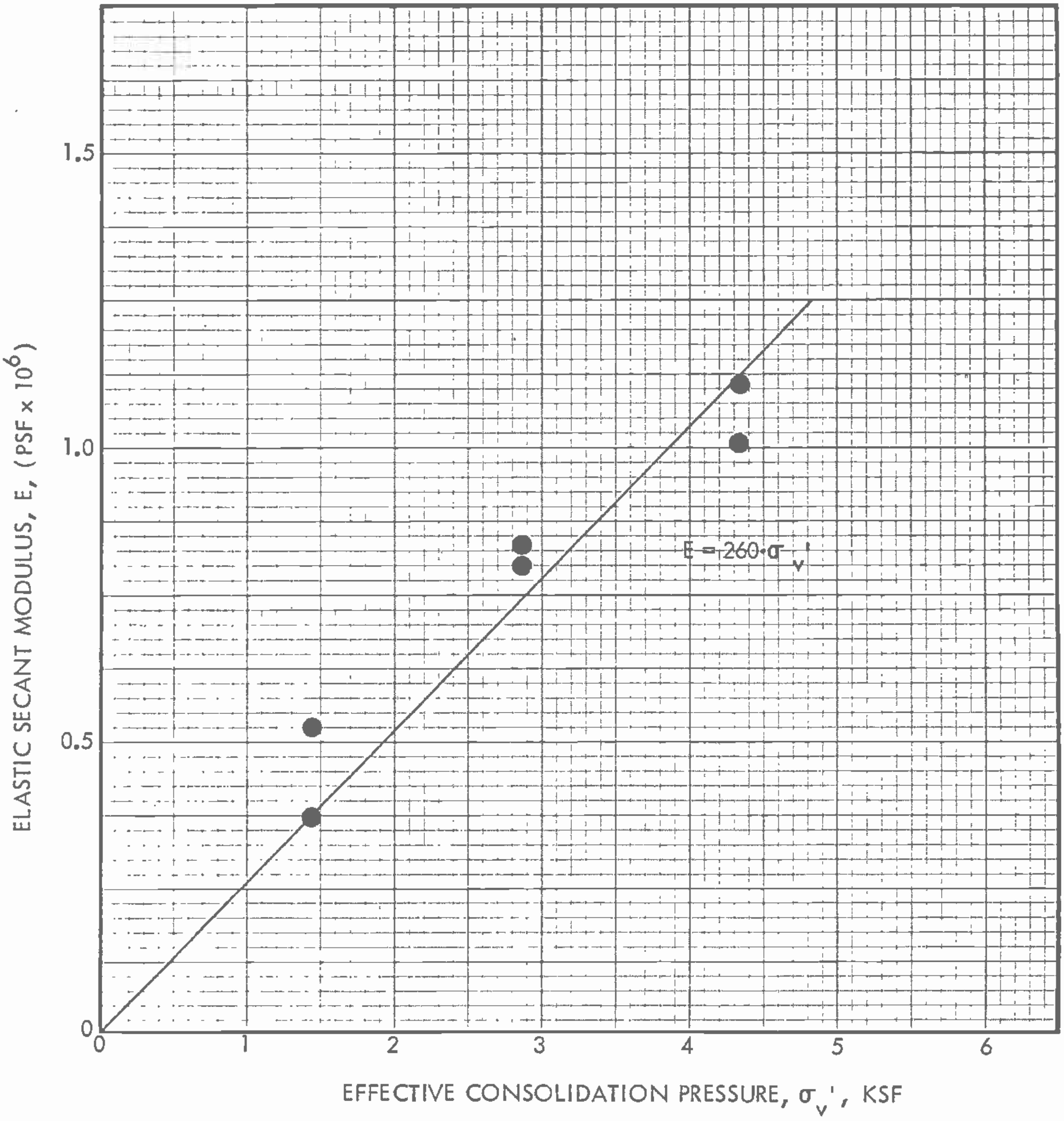
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-3

NOTE: Secant Modulus values are based on 0.5% axial strain.



Approved for publication 5/84 by

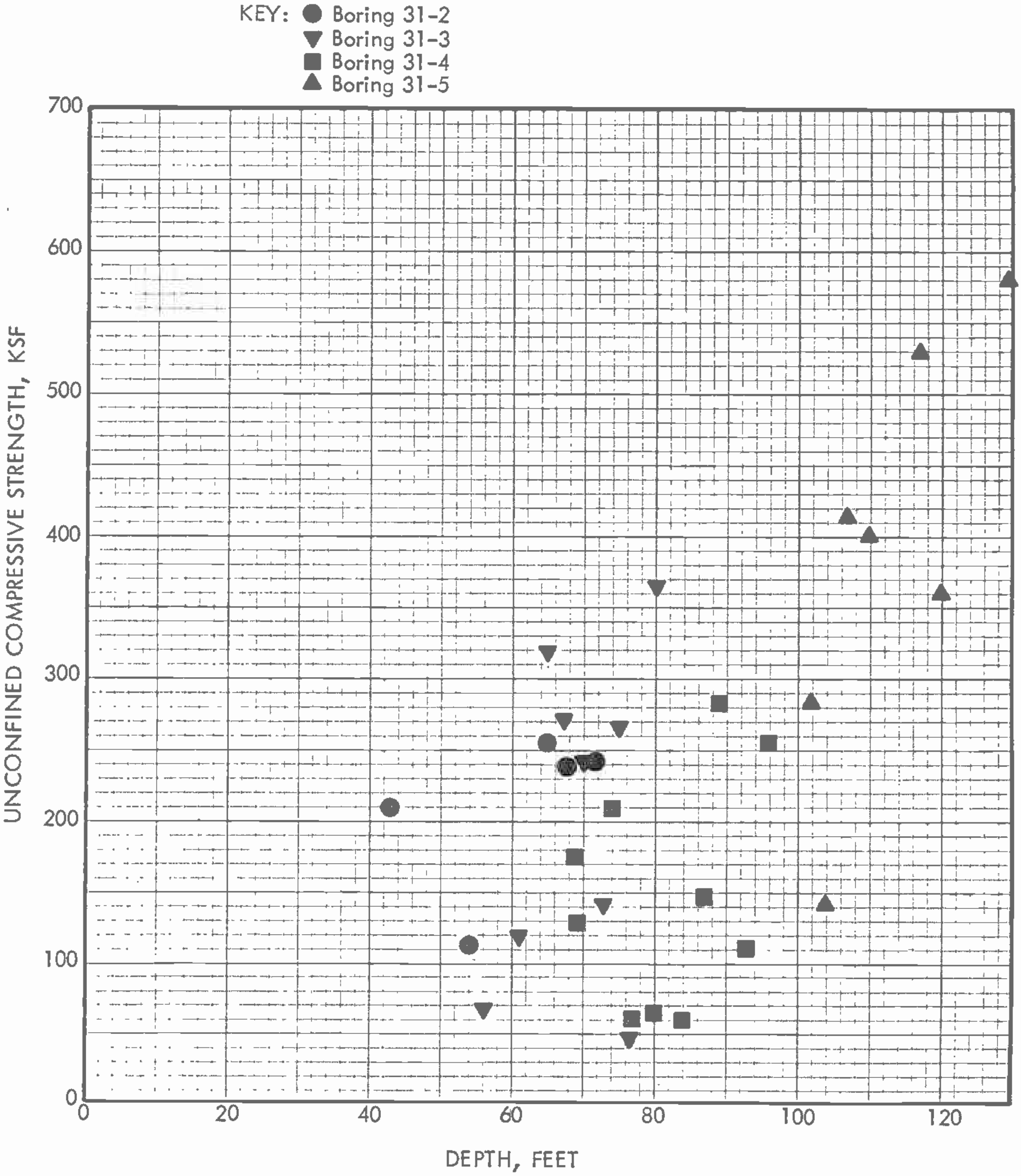
SUMMARY OF MODULUS DATA - ALLUVIUM

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-4



SUMMARY OF UNCONFINED COMPRESSION TESTS - BASALT

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

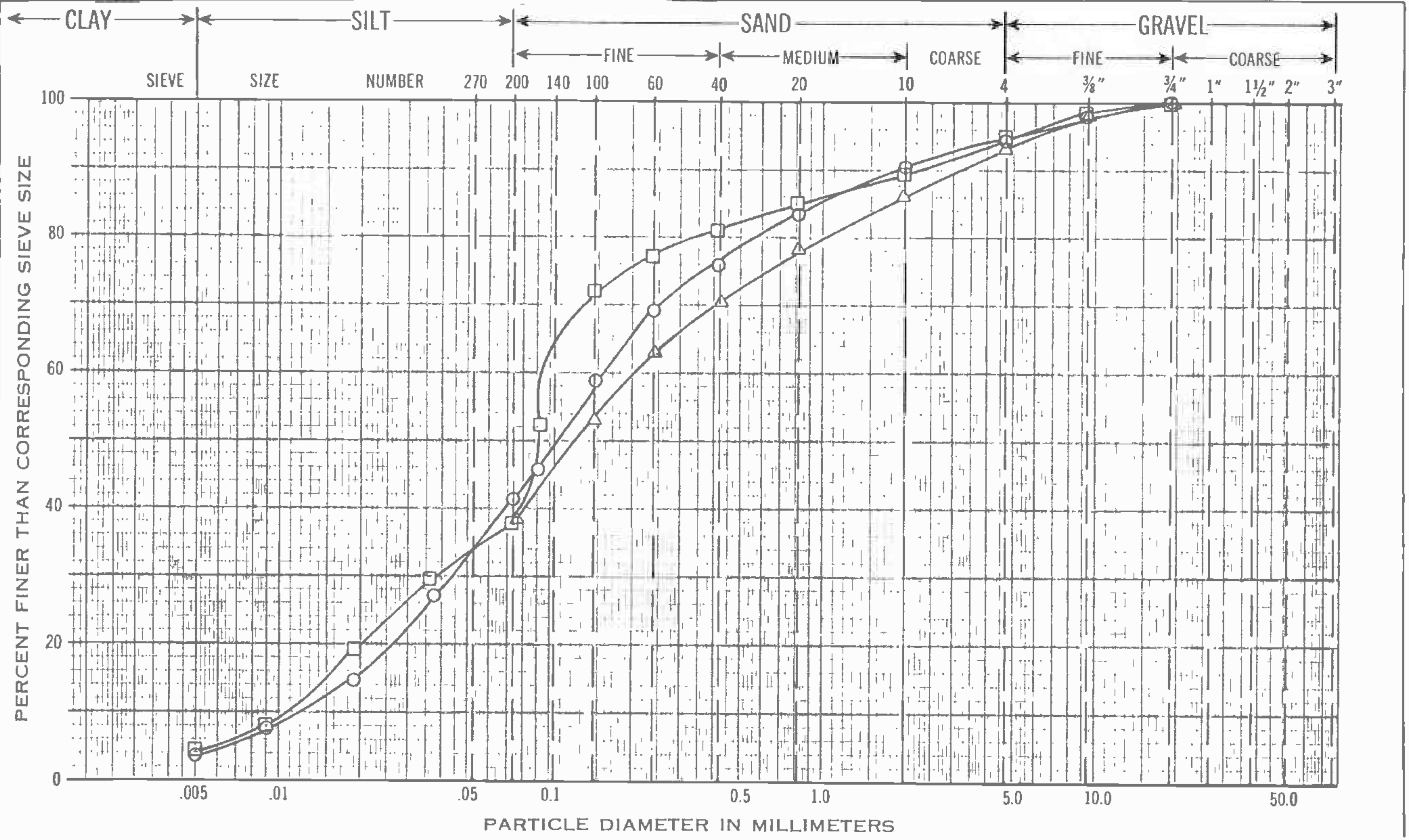
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-5

Approved for publication 5/1/84 by

FORM NO. D-3-1 Approved for publication 5/841 by



SYMBOL	BORING	SAMPLE	DEPTH
—○—	31/2	C-2	7'-7"
—□—	31/2	C-4	17'
—△—	31/2	C-5	23'

GRAIN-SIZE DISTRIBUTION CHART

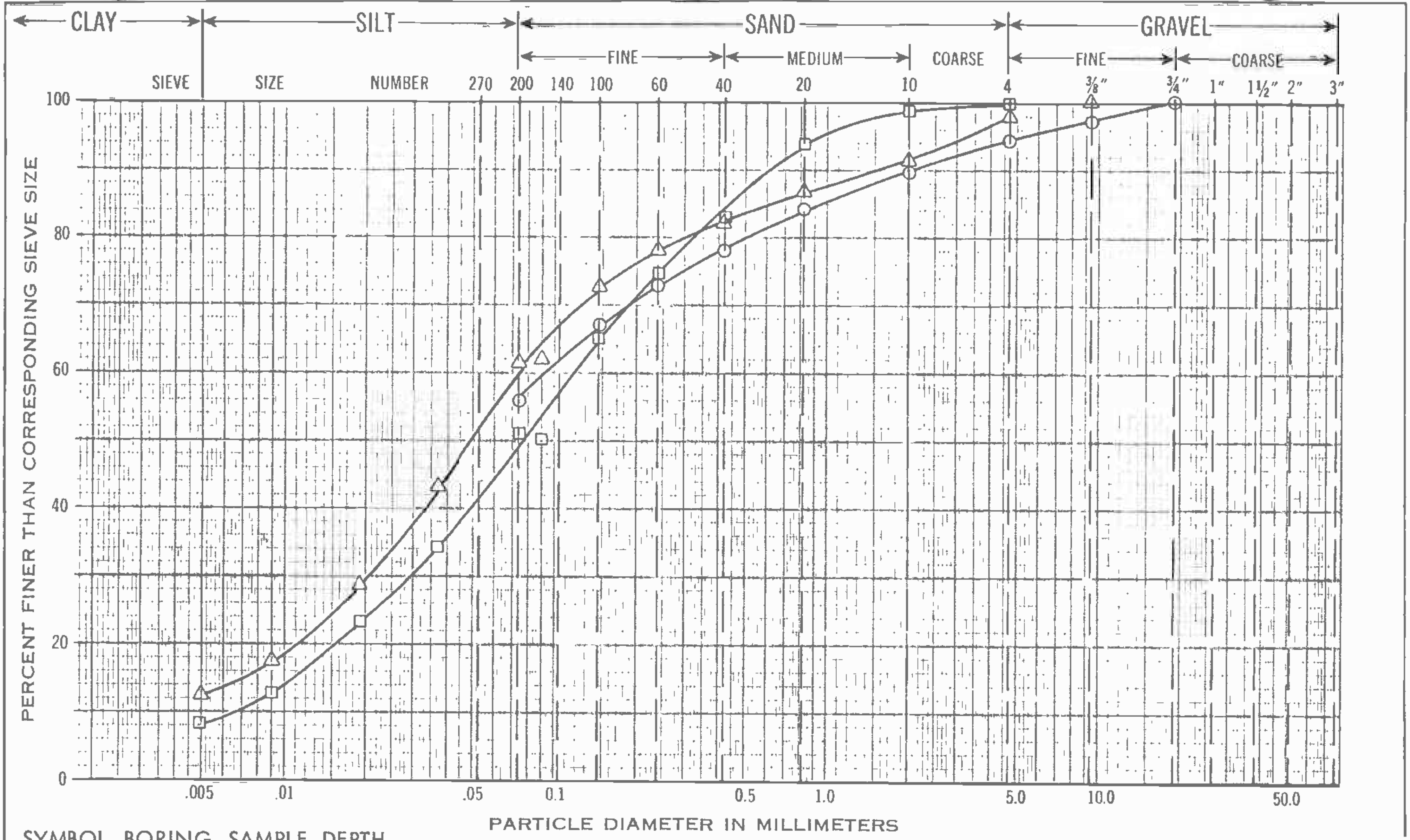
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Drawing No.
C-6

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



SYMBOL	BORING	SAMPLE	DEPTH
—○—	31/3	J/1	8.0'
—□—	31/3	J/2	16.0'
—△—	31/3	J/3	27.0'

GRAIN-SIZE DISTRIBUTION CHART

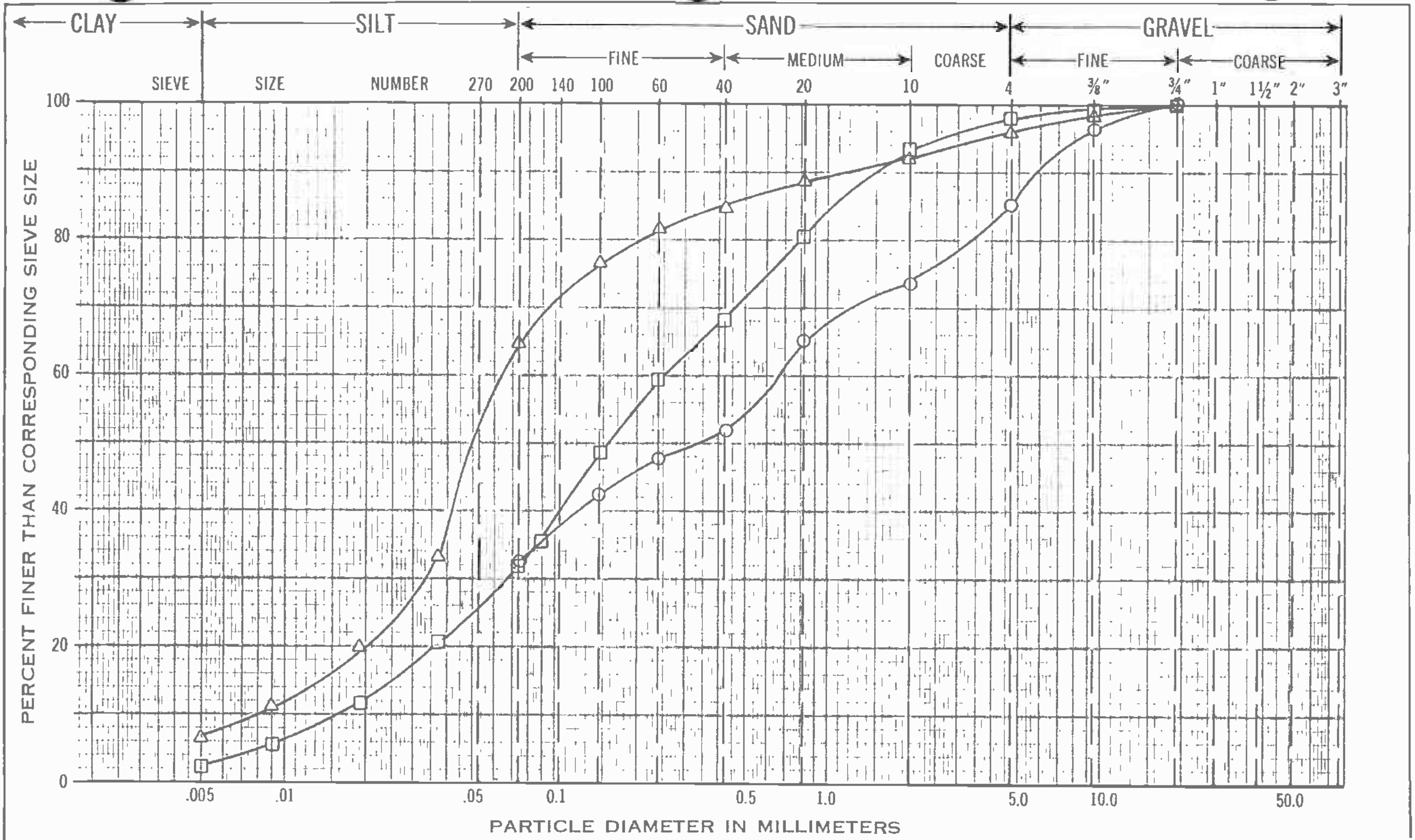
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Drawing No.
C-7

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



SYMBOL	BORING	SAMPLE	DEPTH
—○—	31/3	J-4	34'
—□—	31/4	C-1	3'
—△—	31/4	C-4	17'

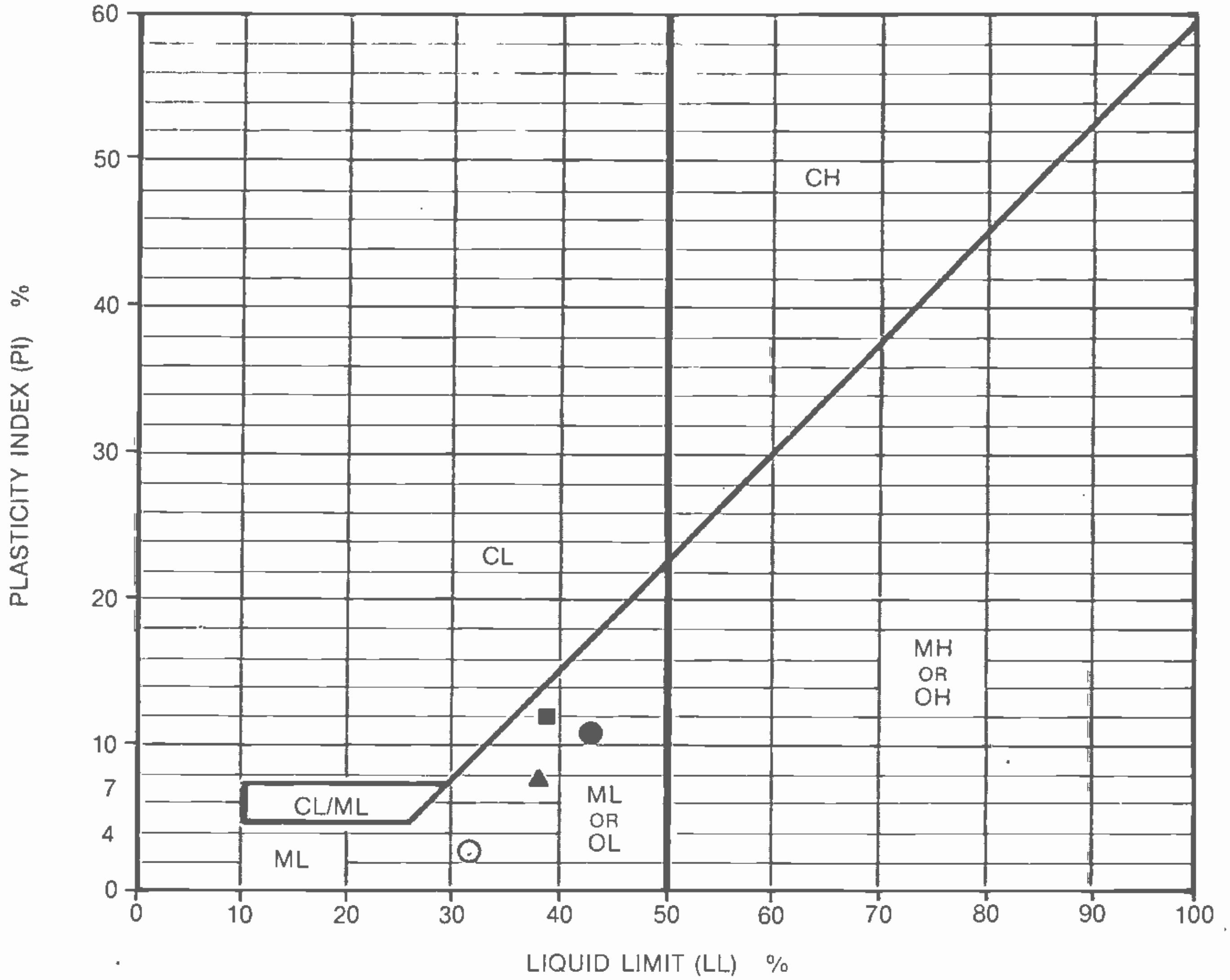
GRAIN-SIZE DISTRIBUTION CHART

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Drawing No.
C-8



Symbol	Classification and Source				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
●	ML	31-2	C-1	3'	43	32	11	
▲	ML/SM	31-3	J-2	16'	38	30	8	49
■	ML	31-3	J-3	27'	39	28	11	60
○	SM	31-4	C-1	3'	33	30	3	32

Approved for publication by

PLASTICITY CHART

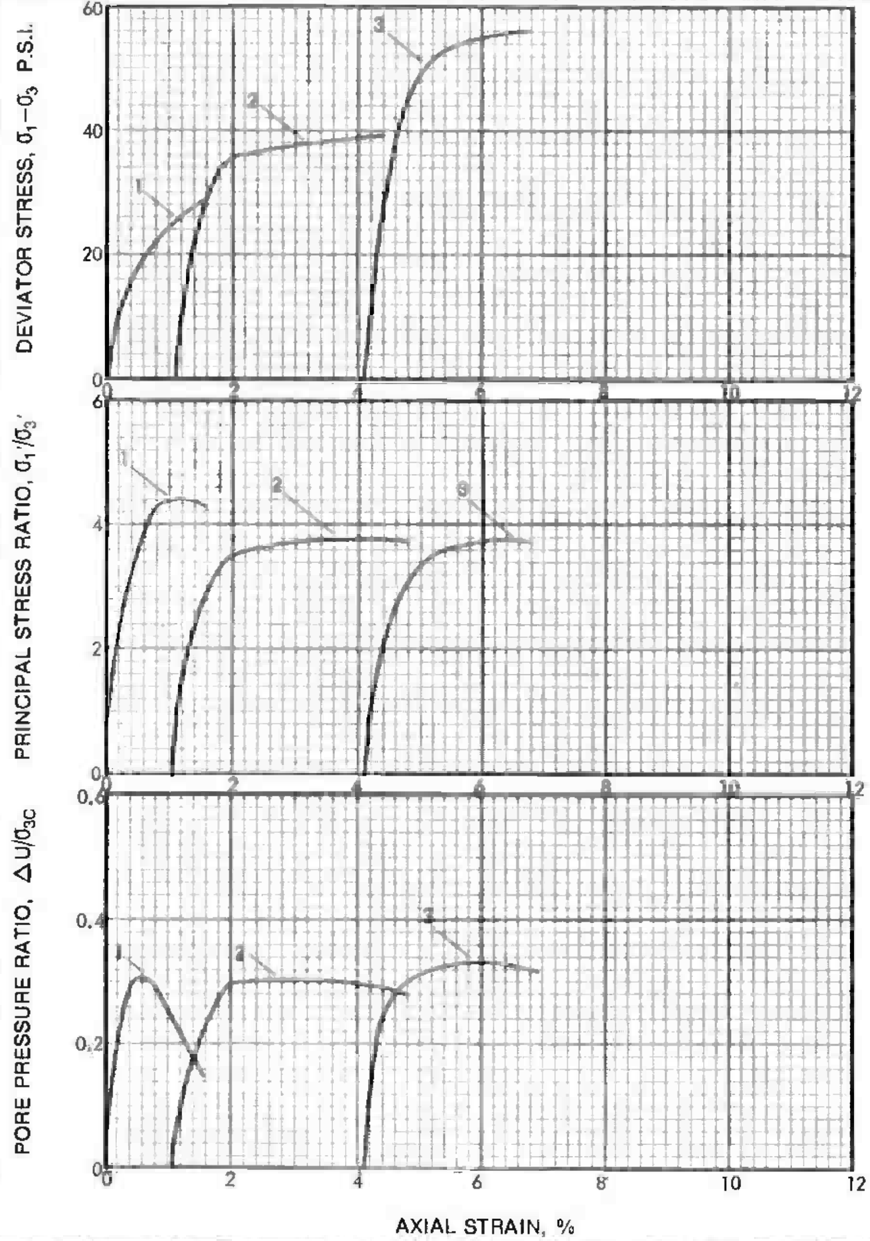
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

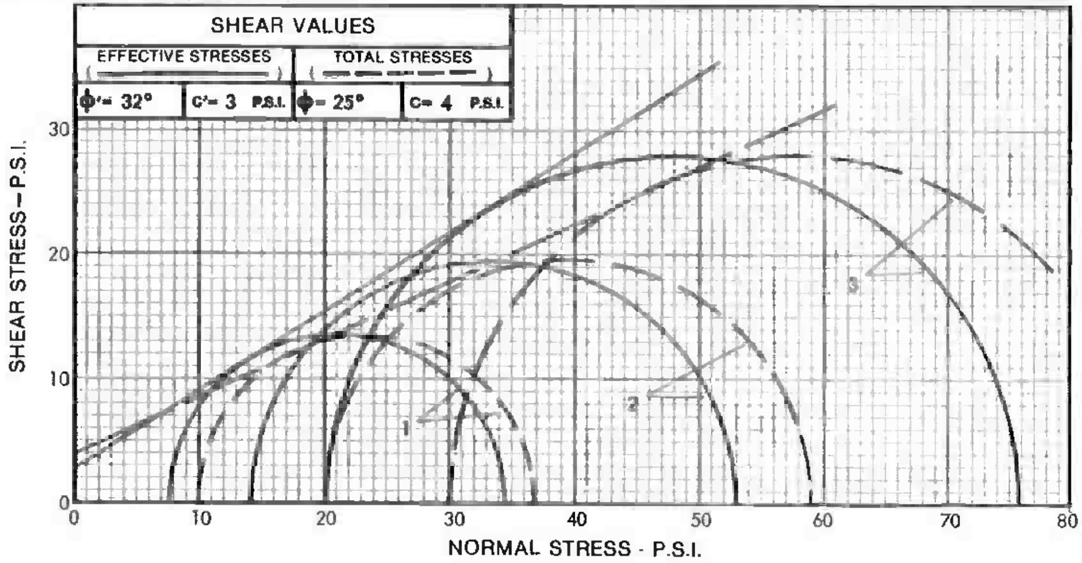
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-9

Approved for publication 5/5/4 by





SHEAR VALUES

EFFECTIVE STRESSES		TOTAL STRESSES	
ɸ'= 32°	C'= 3 P.S.I.	ɸ= 25°	C= 4 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)	
	31-2	C-4	18	ML	5.0	2.42	98.5	22.3	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 $\sigma_1-\sigma_3$ (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	26.5	2.2	7.8	34.3	CUE
	2	20	39.0	5.8	14.2	53.2	CUE
	3	30	55.5	9.9	20.1	75.6	CUE

TRIAXIAL COMPRESSION TESTS

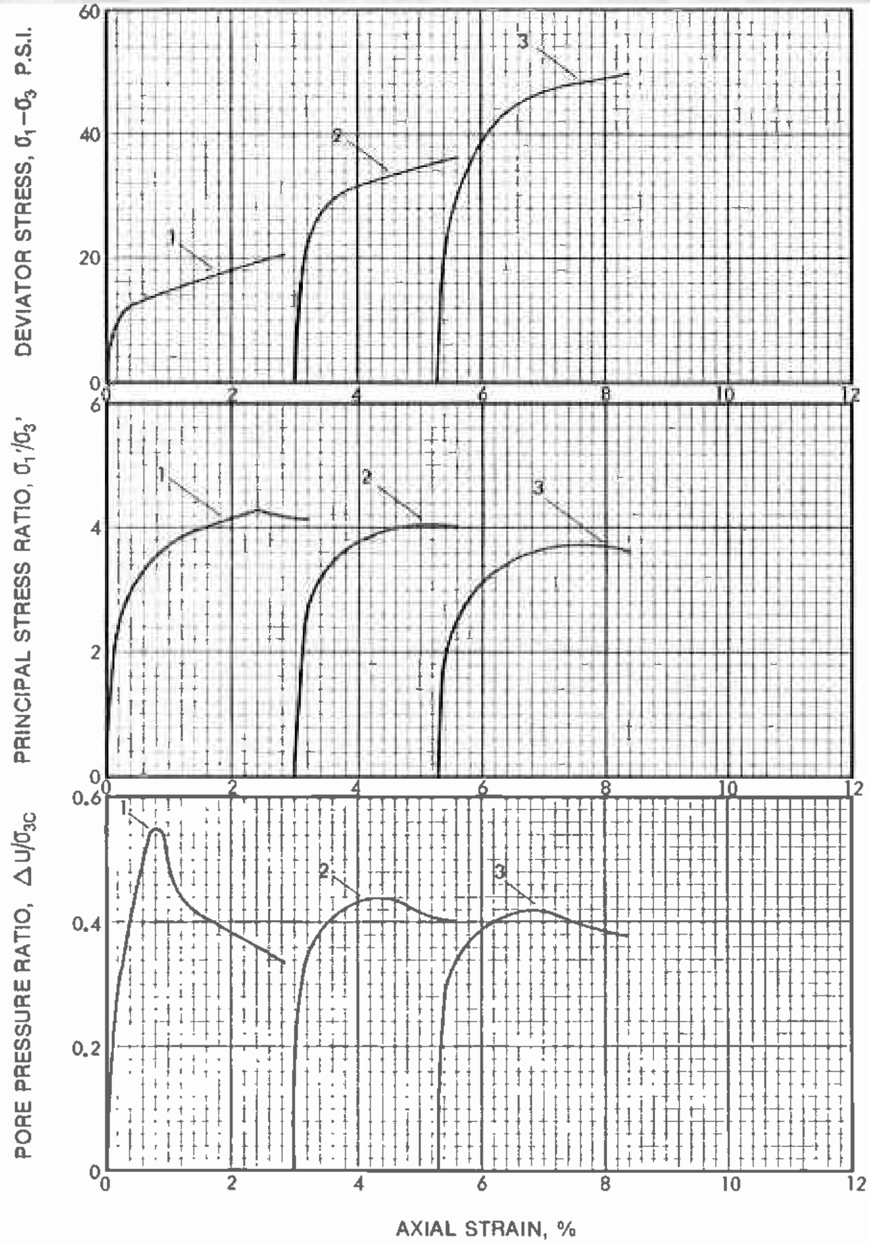
DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

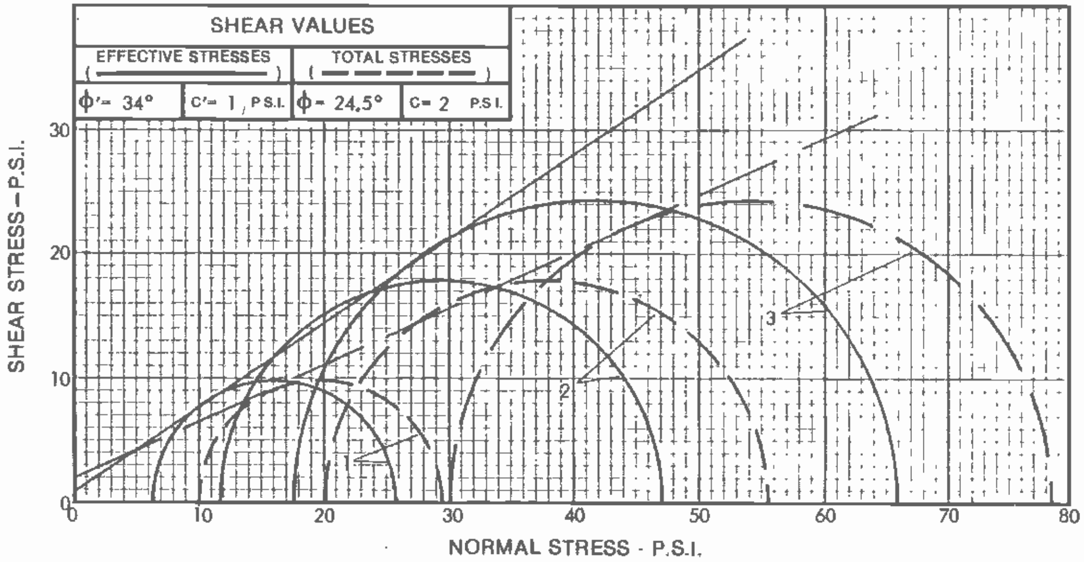
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-10

Approved for publication 5/84 by TKO





SHEAR VALUES			
EFFECTIVE STRESSES (———————)		TOTAL STRESSES (— — — — —)	
φ'= 34°	C'= 1 P.S.I.	φ= 24.5°	C= 2 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)	
	31-4	C-4	17-18	SM	5.0	2.42	98.3	26.7	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 $\sigma_1-\sigma_3$ (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	19.5	3.6	6.4	25.9	CUE
	2	20	35.5	8.2	11.8	47.3	CUE
	3	30	48.5	12.5	17.5	66.0	CUE

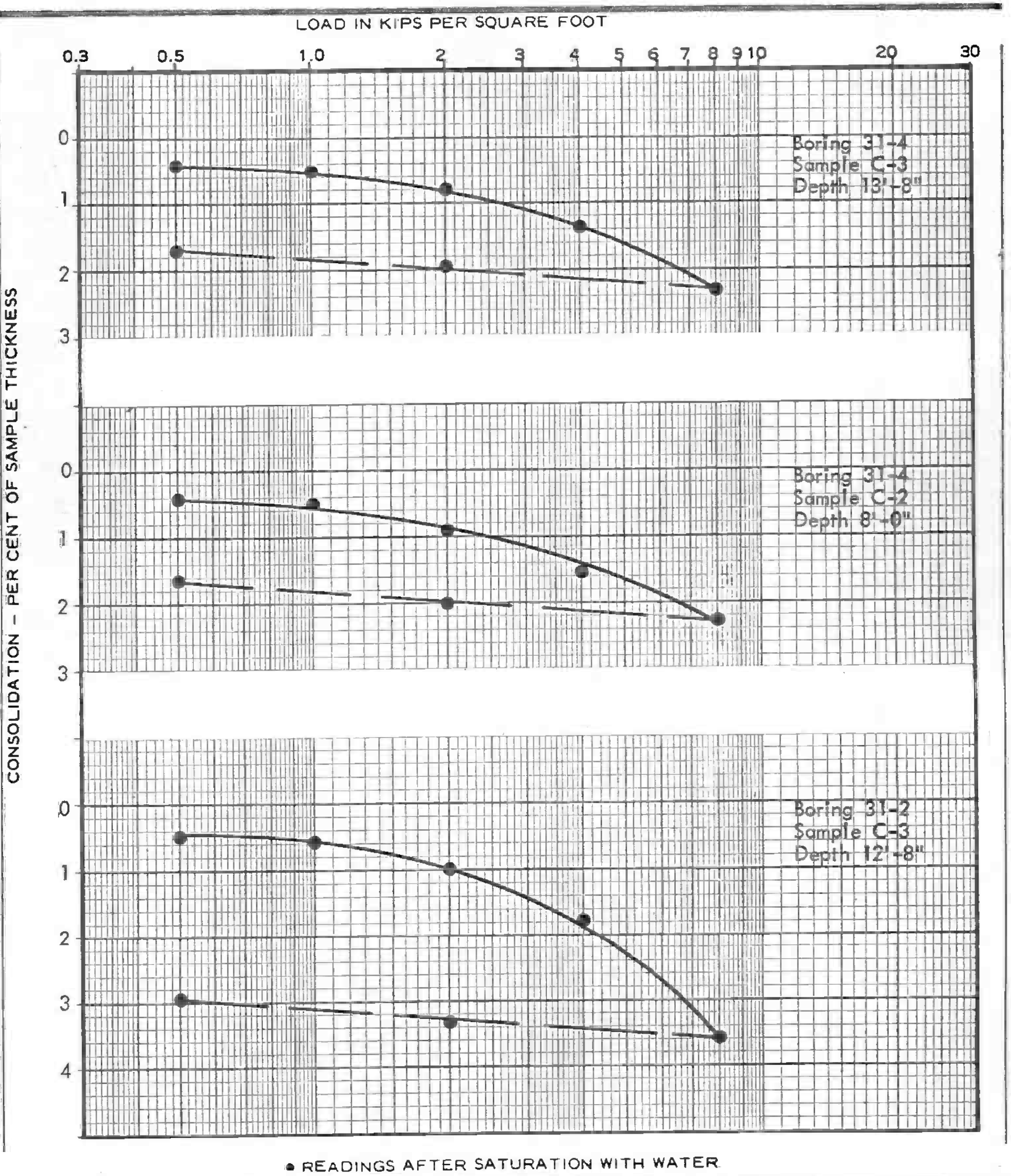
TRIAXIAL COMPRESSION TESTS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No
C-11



Approved for publication 5/84 by JKD

CONSOLIDATION TESTS

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Drawing No.
C-12

Appendix D

Water Quality Analysis

APPENDIX D WATER QUALITY ANALYSIS

D.1 RESULTS

Water samples were taken from Boring CEG-31 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 a piezometer. 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. P81-03-017-1

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 3-3-81

Sample labeled: HOLE 31-2"

Conductivity: 811 µ mhos/cm

Turbidity: NTU

pH 8.6 @ 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	15	0.75
Magnesium, Mg	1.8	0.15
Sodium, Na	157	6.83
Potassium, K	3.0	0.08
	Total	7.81
Anions determined:		
Bicarbonate, as HCO_3	167	2.74
Chloride, Cl	50	1.41
Sulfate, SO_4	161	3.35
Fluoride, F	0.9	0.05
Nitrate, as N	2.4	0.17
	Total	7.72
Carbon dioxide, CO_2, Calc.	< 1	
Hardness, as $CaCO_3$	45	
Silica, SiO_2	25	
Iron, Fe	2.12	
Manganese, Mn	< 0.01	
Boron, B	0.58	
Total Dissolved Minerals, (by addition: $HCO_3 \to CO_3$)	511	

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:

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

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

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.

SPT data was obtained in four borings (31-2, 31-3, 31-4 and 31-6) drilled within the station limits. Due to the limited alluvium thickness, only nine tests were obtained below a depth of 15 feet in the three borings. Six SPT tests were performed in silty sand soils in Borings 31-2, 31-4 and 31-6. Most of the soils in Boring 31-3 were classified as sandy silt with clay (PI = 8 to 11); therefore, the two SPT tests in these soils in Boring 31-3 were not considered for our evaluation. However, the SPT test at 33 feet in Boring 31-3 was considered valid because the laboratory classification was silty sand with only slight plasticity.

Our analysis of the SPT data was performed in accordance with the simplified procedures of Seed et al (1983). Corrected "N" values (normalized to 2 ksf overburden pressure) for the five SPT tests in silty sand soils from Borings 31-2 and 31-4 ranged from a minimum of 31 to a maximum exceeding 75+, with an average of about 50+. The corrected "N" values for silty sands in Borings 31-3 and 31-6 were only 11 and 21, respectively. Determination of dynamic strength was based on an M7.0 earthquake event. Results of the analyses indicated that, where corrected "N" values equaled 30 or greater, the soils would not liquefy during the maximum design earthquake (MDE).

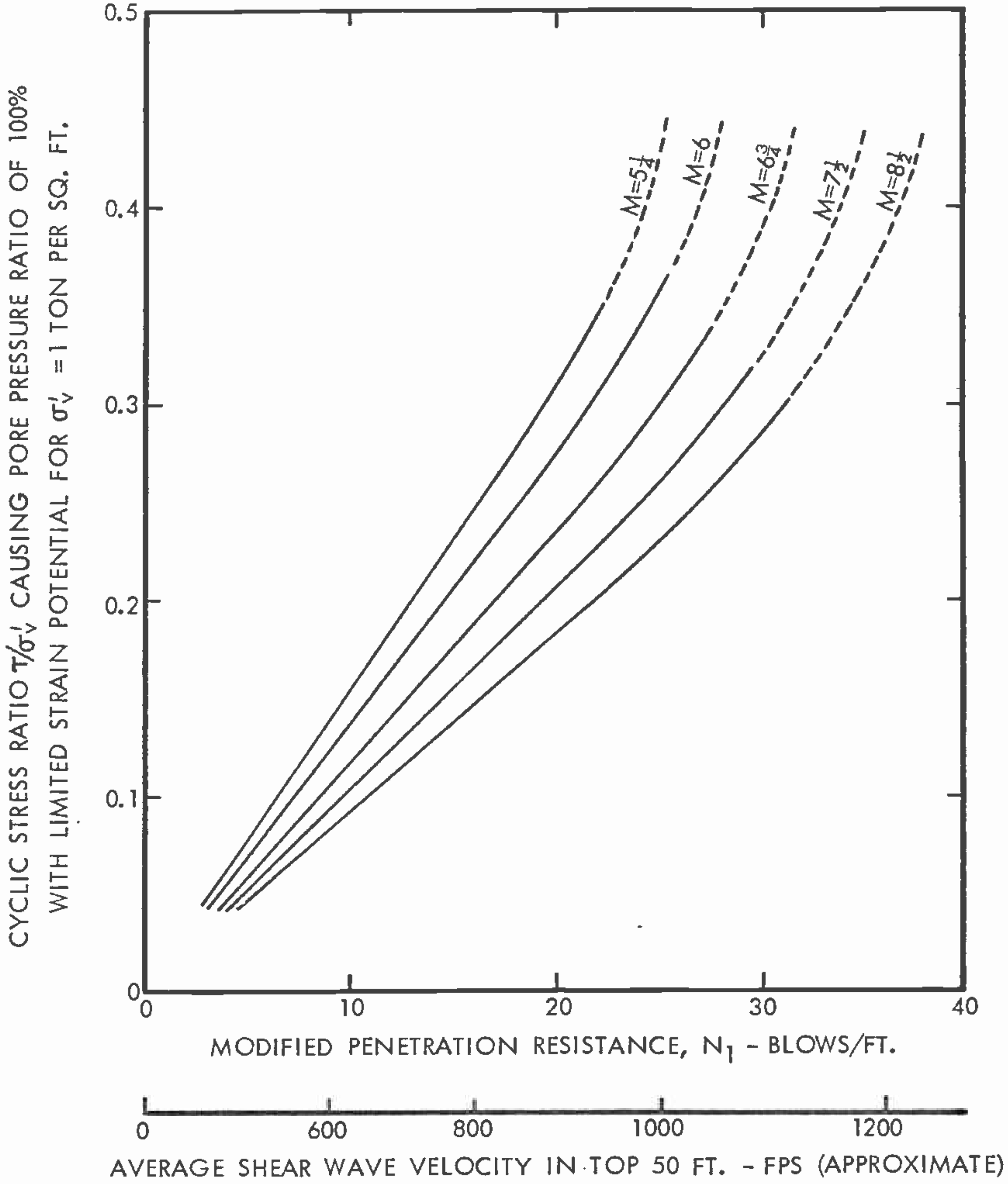
E.3.2 Shear Wave Velocity Measurements

Downhole 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. The downhole survey was performed at Boring CEG-31. Shear wave velocities measured in the Alluvium (approximately the upper 30 feet of the borehole) was 1270 fps.

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 very low based on the shear wave velocities measured at CEG-31.

E.3.3 Conclusions

Based on the above considerations and comparisons, it is our judgement that the limited alluvial soil deposits would have low liquefaction potential during ground shaking from the maximum design earthquake. The low liquefaction potential of the alluvial soils is anticipated due to the low potential for high ground water combined with the generally high SPT blow counts of the sand soils and the clay content of the silt soils.



(after Seed, 1983)

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

DESIGN UNIT A415
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
E-1

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 4/1/84 by

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.

° Site Preparation (surface structures): 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.

° Minor Construction Excavations: 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.

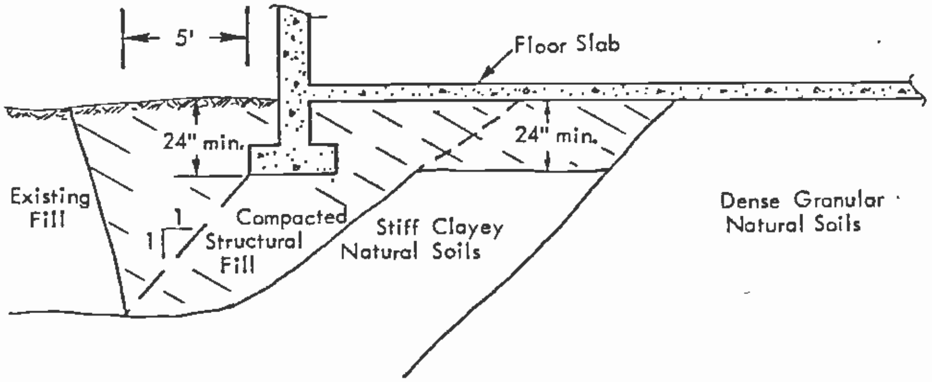
° Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed 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 for near-surface structures within the alluvium may be supported directly on undisturbed stiff or 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".

° Recommended Specifications for Fill Compaction: 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 6 inches. 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 granular 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.