Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A245

BY

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

APRIL 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

29395981

Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

April 11, 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 A245 prepared in accordance with our Contract No. 503 agreement dated September 30, 1984 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 A245.

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

Respectfully submitted,

Howard A. [illegible]

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

RMP:i

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

PROFESSIONAL CERTIFICATION

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

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

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

Robert M. Pride
Senior Vice President

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

Howard A. Spellman
Principal Engineering Geologist

Table of Contents

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

LIST OF TABLES

LIST OF FIGURES

Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation and engineering analyses for the A245 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A245 Design Unit consists of the Wilshire/La Brea Station. The Station will be constructed by cut-and-cover methods and will extend in depth up to about 55 feet below the existing ground surface. This report defines the subsurface conditions and provides recommendations for design and construction purposes.

1.1 STATION CONSTRUCTION

The subsurface conditions at the station site consist of 50 to 62 feet of alluvium, primarily silts, clays, clayey sands and silty sands. Underlying the alluvium, the explorations encountered the San Pedro sand and gravel layer varying in thickness between 25 and 30 feet. The San Pedro sand is in turn underlain by interbedded siltstone, claystone and sandstone of the Puente Formation. Ground water was encountered within the alluvium at depths of 12 to 15 feet below the existing ground surface.

Station construction on Wilshire Boulevard will consist of an excavation approximately 550 feet long, 60 feet wide, and up to about 55 feet deep. The Wilshire/La Brea Station excavation will occur almost entirely within alluvial soils. The west end of the excavation may penetrate to the San Pedro Sand Formation.

Temporary support of the Station excavation will be either flexible or rigid type vertical wall systems with internal bracing or external tieback systems. Successful installation of tiebacks will require certain precautions to maintain the stability of the shafts below ground water elevations. Lateral pressures and other guidelines for design of temporary support systems are provided in the report.

Certain fractions of the alluvium are more pervious than other fractions. Therefore, exterior and/or interior dewatering installations are anticipated to be necessary to control ground water seepage and loss of ground along the excavation faces and to maintain the stability of the bottom of the excavation. Dewatering of the alluvium and San Pedro Formation will result in some areal subsidence.

The undisturbed alluvium and the San Pedro Formation will adequately support the permanent reinforced concrete station structure. Design lateral pressures for the permanent structure under varying earth and hydrostatic loading conditions are outlined in the text of the report.

1.2 UNDERPINNING

Guidelines for assessing the need for underpinning of buildings adjacent to the Station construction are discussed in the report. Detailed analyses to identify and recommend which buildings and/or facilities shall be underpinned will be carried out by the section designer for this Design Unit.

1.3 SEISMIC CONSIDERATIONS

Analysis of the gradational characteristics and in-situ relative density of the granular soils indicate that liquefaction of such soils during a maximum design earthquake has a low probability.

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD report entitled "Guidelines for Seismic Design of Underground Structures" dated March 1984. 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 A245 are given in this report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A245, Wilshire/La Brea Station. The work performed for this report includes borings, laboratory tests, engineering analysis, and the development of recommendations and 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 A245.

- "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 11 to 43, dated September 1983; and Preliminary Site Plans, Plans and Sections for Wilshire/La Brea Station, dated February, 1983.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

The Wilshire/La Brea Station site will be located beneath Wilshire Boulevard between Detroit Street and Sycamore Avenue. Development along Wilshire in this area consists primarily of low-rise commercial and retail buildings with the exception of the Mutual of Omaha Building located at the intersection of Wislhire and La Brea. Residential areas are to the north and south of Wilshire. The existing ground surface along Wilshire Boulevard varies from Elevation 194 feet at Detroit Street to Elevation 197 feet at Sycamore Avenue.

The Wilshire/La Brea Station will be a reinforced concrete structure about 550 feet long and 60 feet wide (outside wall dimensions). The station has been planned with a mezzanine, and an entrance located at the northwest corner of Wilshire and La Brea. Ancillary space is proposed at each end of the station. A traction power substation will be constructed at grade adjacent to the Station entrance. The top of rail varies from about Elevation 150 feet at the east end to about Elevation 148 feet at the west end of the station. Assuming the station will be supported on a 4- to 6-foot thick concrete mat, the station area will require an excavation to about Elevation 144 feet. This is approximately 53 feet below the existing grade at the east end of the station, and 50 feet below the existing grade at the west end of the station. After the station is constructed, about 12 to 15 feet of fill will be placed above the majority of the station box. Design loads for this Station structure were not available at the time of this report.

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 A245 investigation, 8 borings were drilled at the proposed station and crossover structure. Subsequent to the completion of the exploration, the crossover structure at this station was omitted. The borings consists of small diameter rotary wash holes numbered 18-1 through 18-7, and a 36-inch diameter man-size auger boring, 17-B. Rotary CEG-18 drilled in 1981 is also included. The locations of the borings are shown on Drawings 2 and 3, and the logs of the borings from the 1981 and 1983 investigations are provided in Appendix A. Ground water observation wells were installed in Borings 17-B, CEG-18, 18-1, 18-3 and 18-7. Section 5.4 presents a summary of ground water level measurements in these wells.

In 1962, Kaiser Engineers drilled 2 borings within the Design Unit A245 Station site. Borings 42 and 43 were drilled about 500 feet apart and ranged from 50 to 80 feet deep at the locations shown on Drawing 2. The Kaiser Boring Logs can be examined at the Southern California Rapid Transit District office in Vol. 4, Books 2 and 3, entitled "Test Boring Program" prepared for the Los Angeles Metropolitan Transit Authority, June 1962.

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 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 and crosshole compression and shear wave velocity surveys were performed in Boring CEG-18 which was drilled during the initial 1981 investigation. The CEG-18 boring was drilled on the north side of Wilshire Boulevard at the A245 Station site. Appendix B summarizes the field 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, triaxial compression tests, resonant column tests, unconfined compression tests, direct shear tests, and permeability tests.

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

4.5 WATER QUALITY ANALYSES

Chemical analyses were performed and selected parameters were evaluated for water samples obtained in Boring 17B. The results of these tests are presented in Appendix D.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

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

Drawings 2 and 4 show generalized subsurface cross sections through the proposed Wilshire/La Brea Station. The subsurface profile at the Station site consists of approximately 1 to 4-1/2 feet of fill over fine-grained Alluvium extending to depths of approximately 50 to 62 feet. Beneath the Alluvium, a layer of very dense San Pedro Sand was encountered. The thickness of this sand layer varied between 25 and 30 feet. The bedrock surface at this Station site was nearly horizontal.

5.2 SUBSOILS

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

- Fill: Fill soils were encountered below surface pavement in five of the eight borings drilled at the site. Fill depths encountered ranged from 1 to 4.4 feet below the surface. The fill generally consisted of relatively clean (no debris) sandy or silty clay which was stiff and moist. At Boring 18-1 however the fill consisted of an upper sandy gravel base course material approximately 1-1/2 feet thick.

- Alluvium: Alluvium soils were encountered to depths of 50 to 62 feet at this site and were primarily fine-grained soils consisting of (in order of decreasing occurrence) sandy clay, silty clay, clayey silt, clayey sand, sandy silt and silty sand. The various soil types encountered were observed to be relatively thin layers ranging from 1 or 2 feet thick to up to about 10 feet thick. Some general trends of the soil stratification, i.e. silt/clay mixtures vs. sand/clay mixtures, can be seen on Drawing 4; however, specific layers generally appeared to be discontinuous. Sampling resistance, SPT results and laboratory test results indicate that these soils are generally stiff to hard and have low compressibility.

- San Pedro Sand: San Pedro Sands encountered below the alluvium at the boring locations were typical for this formation and generally consisted of clean (5%± fines), medium to fine sand with occasional gravelly sand or silty sand lenses. The total stratum thickness generally ranged from 25 to 30 feet thick. Sampling resistance and SPT results in the San

Pedro Sands were very high indicating that this material is very dense and relatively incompressible. At Boring 18-2 within the Station limits a sandy gravel layer approximately 13 feet thick was encountered at the bottom of the San Pedro Sand layer.

5.3 BEDROCK

All but two of the eight borings drilled at the site (Borings 18-6 and 18-7) penetrated into the Fernando Formation bedrock underlying the alluvium. Where encountered, the bedrock consisted of claystone or interbedded siltstone and claystone. The bedrock was little weathered to fresh, thinly bedded to massive. Bedding dip was measured in CEG-18 to be between 10 and 30 degrees. Strike of the bedding could not be determined from the samples obtained. Regional bedding strike is nearly east-west and the dip is south. Sulphur and/or petroleum odors were noted in the bedrock samples from Borings 18-1, 18-2, 18-3, 18-5 and CEG-18.

5.4 GROUND WATER

Regionally, ground water has been measured both at shallow depths within the alluvium and at deep levels within the bedrock. The alluvial ground water occurs at depths of ranging from about 12 to 15 feet below the surface at the Wilshire/La Brea Station site. Ground water within the bedrock below the site is estimated to be about 150 feet below the ground surface.

Table 5-1 presents ground water levels and fluctuations measured in piezometers installed at Borings CEG-18, 18-1, 18-3 and 18-7, and those observed in the man-size Borehole 17B. Based on the measurements presented on Table 5-1, it appears that the ground water level does not vary significantly across the site. Most water level measurements vary between Elevations 177 and 182 and, due to the limited data available for this station, no apparent trends could be used to establish a gradient across the site.

TABLE 5-1
GROUND WATER OBSERVATION WELL DATA*

BORING	GROUND WATER ELEVATION					
	Initial (Date)	01/26/81	10/13/83	11/03/83	12/16/83	3/14/84
17B	180 (10-26-83)	--	--	--	--	--
CEG-18	183 (01-26-81)	179	--	173	--	--
18-1	--	--	182	181	177	178
18-3	--	--	174	178	180	--
18-7	--	--	182	181	178	179

*Rounded to the nearest foot.

Chemical analysis of the water from Boring 17B was made during the 1983 investigation. Results of tests at 17B are presented in Appendix D. Other nearby 1981 borings along Wilshire Boulevard (CEG-16 and CEG-19) indicate that oil field brine could be encountered in this area.

5.5 GAS

No gas analyses were made at this site; however, sulphur and/or petroleum odors from the bedrock samples were noted in borings CEG-18, 18-1, 18-2, 18-3 and 18-5. In addition a sulphur odor from the San Pedro Sand samples was noted in boring 18-2.

5.6 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.6.1 General

For purposes of our engineering evaluations, we have grouped the subsurface materials encountered at the Wilshire/La Brea Station site into three general subsurface units. These subsurface units include Alluvium, San Pedro Sand, and bedrock. This section includes descriptions of each subsurface unit and presents engineering parameters used in our analyses (see Table 5-2). These parameters are based on the laboratory test results, field test results, data from previous investigations, and published data of observed and recorded field behavior from construction projects.

TABLE 5-2
MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	GEOLOGIC UNIT		
	Alluvium	San Pedro Sand	Bedrock[c]
Moist density above ground water (psf)	127	-	-
Saturated density (pcf)	130	130	120
Effective Strength			
ϕ' (degrees)	28	35	35
c' (psf)	400	0	0
Total Strength[a]			
ϕ (degrees)	20	-	10
c (psf)	750	-	5,000
Average Unconfined Compressive Strength (psf)	4000	-	10,000
Permeability (cm/sec)	10^{-3} to 10^{-6}	10^{-2} to 10^{-3}	10^{-6} to 10^{-7}
Poisson's ratio	0.35	0.35	0.35
Initial Tangent Modulus (psf)	$270 \cdot \sigma_v'$[b]	$600 \cdot \sigma_v'$[b]	2.0×10^6

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

[b] σ_v' is the effective overburden pressure (psf) equal to effective density times overburden depth. Moist density should be used to determine σ_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.

[c] Values based on test data from other Design Units.

5.6.2 Alluvium

The alluvium consists of interbeded sandy clays, silty clays, clayey silts, clayey sand and sandy silts. Within this unit, lenses and discontinuous layers of silty sands were also encountered. Standard Penetration Test (SPT) results and laboratory test results indicate that the alluvium is generally stiff to hard, and granular layers are dense to very dense.

Since these soils are generally silty and clayey in nature, both drained (effective) and undrained (total) strength parameters have been developed from results of direct shear and triaxial compression tests. The recommended strength parameters given in Table 5-2 were selected based primarily on the results of tests performed on samples obtained from the Wilshire/La Brea Station site, although strength test results obtained from other nearby design units were also considered.

Young's Modulus or initial tangent modulus were found to be a function of the mean confining pressure at the end of consolidation. Modulus values for the alluvium were therefore normalized to the consolidation pressure. The normalized values recommended for the alluvium are presented in Table 5-2.

Permeability tests performed on triaxial test samples of alluvium obtained from other design units indicate that these soils have permeability ranging from about 10^{-5} to 10^{-8} cm/sec. However, since the soils were found to be interbedded and lenticular, slightly higher permeabilities are recommended for design calculations.

5.6.3 San Pedro Sand

The relatively uniform fine to medium sand, gravelly sand and silty sand of the San Pedro Formation was encountered at the Wilshire/La Brea Station site. Standard Penetration Test (SPT) results and laboratory tests indicate that this sand unit is generally very dense. This unit lies below the alluvium water level.

Since these materials are relatively free-draining, only drained (effective) strength parameters have been recommended for design. The recommended values were based primarily on the test results for this investigation but results from other nearby design units were also considered. Permeability of the sands is expected to range between the fine sand materials (10^{-3} cm/sec) and gravelly lenses or layers (10^{-2} cm/sec) that may be encountered at this station site.

Elastic properties for the sands were based on the results of the laboratory triaxial tests performed as part of this investigation. This data suggest that the modulus does not increase significantly with the increase in confining stress.

5.6.4 Bedrock

For engineering purposes, the claystone and siltstone was considered to be very stiff to hard overconsolidated fine-grained soil. Strength parameters presented in Table 5-2 should be considered to be representative of the relatively fresh bedrock and were based on interpretation of triaxial, unconfined compression and direct shear tests combined with our engineering judgement. The total stress data from laboratory test results indicate a relatively high undrained friction angle. However, experience and principles of soil mechanics predict that the undrained strength of the bedrock should approach that of a pure cohesive material.

Bedrock elastic properties were selected based on consideration of field performance data, laboratory test data and published information combined with engineering judgement. For this study, the bedrock material was considered to have no significant modulus increase within the range of depth affected by the proposed station. The apparent variation of modulus values at low confining pressures indicated by the laboratory data may be due to several factors including the effects of sample disturbance and sample expansion after insitu stresses were removed.

Section 6.0

Geotechnical Evaluation and Design Criteria

6.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA

6.1 GENERAL

In general terms, construction of the A245 Station will involve deep excavations through stiff and dense alluvium to depths of 50 to 55 feet below the ground surface. The existence of high ground water levels will require either dewatering or tight shoring for the construction excavations. The permeable San Pedro Sand layer below the alluvium must be dewatered or cut-off to prevent basal heave or blow-out.

If the site is dewatered, our evaluation indicates that some dewatering-related subsidence will likely occur within a few months over an area extending several hundred feet around the excavation. However, differential settlements due to dewatering subsidence are not expected to cause structural distress to adjacent structures assuming that conditions do not differ significantly from those at the station.

Considering the potential for general areal subsidence, it is our opinion that the combination of areal dewatering and the use of underpinning piles should be avoided where possible due to the potential for "downdrag" on underpinning piles and differential settlements between underpinned foundations and non-underpinned elements. Underpinning may be minimized or eliminated by designing a sufficiently conservative shoring system to limit ground movements adjacent to the shoring to tolerable levels or by utilizing column pick-up techniques during the construction period.

An alternative to the dewatering and conservative shoring approach to the excavation would be a tight shoring system such as slurry wall construction. Such a system could eliminate the need for areal dewatering provided that the shoring is extended into the bedrock to effectively cut-off ground water flow from the San Pedro Sand Formation. If areal dewatering were eliminated, related subsidence would not occur, and underpinning could be used as necessary without unusual risk of "downdrag" on underpinning piles.

The permanent Station structure will, in essence, be a concrete box supported on and retaining the surrounding soils. As shown on Drawing 4, the subgrade condition at the Wilshire/La Brea Station generally will be uniform and therefore estimated angular distortions in the longitudinal direction are small.

The following subsections present our further evaluations and recommendations for design and construction of the A245 Station structure.

6.2 EXCAVATION DEWATERING

6.2.1 <u>General Evaluation</u>

The construction of the Wilshire/La Brea Station will require an excavation extending 35 to 40 feet below the measured ground water levels and may require areal construction dewatering if tight shoring is not used. As discussed in

Section 5.0, the subsurface conditions at the site generally consist of predominately fine-grained alluvium, overlying the San Pedro Sand Formation which in turn overlies siltstone bedrock. The bedrock surface and overlying San Pedro Sand strata are relatively flat lying. The bottom of excavation may extend to the top of the San Pedro Sand at the east end but generally it will be within the fine-grained alluvium a few feet above the San Pedro Sands (see Drawing 4).

The dewatering system must relieve the hydrostatic pressures within the San Pedro Formation to prevent basal heave or "blow-out" of the excavation. Ground water inflow to the dewatering system will, therefore, be primarily from the permeable San Pedro Sand Formation. Drawdown within the San Pedro Formation will probably occur within a few weeks; however, complete drawdown within the overlying clayey alluvium may require a few months. The shape of the drawdown surface is expected to be characteristic of the more permeable San Pedro Sand than the clayey alluvium. A relatively flat drawdown surface is expected which may extend 500 feet beyond the excavation. Geologic discontinuities, i.e., major variations in the alluvium or San Pedro could cause variations in the phreatic surface especially during the early stages of dewatering.

The approximate estimates of drawdown time and area of influence were necessarily based on assumed hydraulic properties and uniform conditions. Actual hydraulic properties and possible variations in subsurface conditions could significantly alter drawdown characteristics at the sites from those estimated. In our opinion, the best way to evaluate effects of possible subsurface variations and obtain reliable aquifer properties is by pump test(s) with separate observation wells (piezometers) in the San Pedro Sand and alluvium where the degree of hydraulic connection and the probable effect of the dewatering on the phreatic surface could be directly assessed. The test well(s) should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

Changes in vertical pressures within the alluvium due to the reduction of buoyant forces via dewatering are estimated to result in significant surface settlement within the expected one year or greater construction period. Our settlement calculations based on laboratory consolidation tests indicate that total surface settlements due to dewatering would be 1 to 2 inches for 40 feet of drawdown and 1/2 to 1-1/3 inches for 20 feet of drawdown. Actual total settlements will depend on variations in subsurface conditions and the duration of construction (dewatering). Due to the expected gently sloping ground water drawdown curve, settlements should be relatively uniform (assuming uniform subsurface conditions), and differential settlements were estimated to be about 1/4 inch per 100 feet for locations more than 20 feet from the well.

It will be essential that the dewatering wells be properly designed (and installed) to prevent piping of soil into the wells. Uncontrolled piping into the wells will result in loss of ground (settlement).

As an alternative to dewatering, tight shoring such as slurry wall construction penetrating into the bedrock underlying the A245 site could provide an effective ground water barrier. Chemical grout may also be considered to establish a ground water cut off within the San Pedro Sands in conjunction with a soldier pile system.

6.2.2 Possible Dewatering System

Local practice in the site vicinity generally has been to use conventional deep well dewatering systems without apparent unfavorable subsidence effects. Considering this, it is our opinion that a deep well system could be used for site dewatering. Pumping test(s) should be performed prior to dewatering. A possible dewatering system might consist of the following:

- Deep wells around the perimeter of the excavations pumping from the San Pedro Sands.
- Vertical drains through the alluvium which penetrate to the San Pedro Sands. These should be strategically located to drain known sand zones within the alluvium.
- Supplementary ditch drains and sumps within the excavation to handle localized inflows; e.g. from sand layers.

6.2.3 Criteria for Dewatering Systems

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

- The system should maintain ground water levels low enough to provide stability of the bottom of the excavation against a "blow-out" failure at all times during construction.
- To adequately draw down the water table, the dewatering system should be installed and in operation for a sufficient time period prior to excavating below the static ground water level. This period will depend on the pumping rate of the system and the hydraulic characteristics of the site.
- The dewatering system should maintain the ground water levels low enough to prevent piping of the alluvial soils into the excavation. Inflow seepage should be reduced to quantities which can be accommodated by a drain/sump system and which allow excavation and construction to proceed.
- Wells must be designed and developed to eliminate loss of ground from piping of soils near the wells. The well operations should be constantly monitored for evidence of piping.

- The system should operate continuously. Emergency power and backup pumps should be required to ensure continual excavation dewatering.

6.3 UNDERPINNING

6.3.1 Common Underpinning/Support Methods

Several methods for underpinning are commonly used. These include jacked piles, slant drilled piles, and hand-dug pit or pier underpinning. Another technique which has been used is the "column pick-up" method which provides a means of jacking up selected columns if settlements occur. These various techniques are discussed below.

- Jacked Piles: These piles generally consist of H-sections or open end pipe piles 6 to 18 inches in diameter. These sections generally are preferred due to their relatively low volume of soil displacement which facilitates placement. Open end pipe sections have the additional advantage of permitting clean-out to reduce point and shaft resistance during installation. The piles are normally placed in 4- to 5-foot long sections by jacking against the underpinned footing. Jacked piles are commonly pre-loaded individually to 150% of the design load and then locked off.

- Slant Drilled Piles: This method consists of placing a steel pile in a shaft (generally 12- to 24-inch diameter) drilled from the side of the foundation. The shaft is drilled at a small angle or slant under the foundation and then back-reamed to provide a vertical slot below the foundation. A steel pile is placed under the foundation, and the shaft is filled with concrete. The actual connection to the footing can be made by shimming or "drypack" concrete. Pre-loading could be accomplished using jacks and shims similar to jacked piles. In weak soils or in ground subject to sloughing, this method can result in settlement if there is loss of ground into the drilled hole.

- Hand-Dug Pits: This method consists of excavating an approach pit adjacent to and beneath the footing and advancing square or rectangular shafts, normally 3 to 5 feet wide, down to the bearing stratum. The shaft excavations are lagged for the entire depth with the lagging normally left in place permanently. Reinforcement is placed, and concrete is tremied into the shaft(s). In some cases, this process may be repeated until the entire plan area of the footing is supported on the deep bearing stratum.

- Column Pick-Up: This technique provides a method of releveling specific structural elements without underpinning in the event that excessive settlements occur. A structural break is made between the column (or wall) and its foundation. Special connections are made to transmit loads around the structural break and jacking, or other means, is used to relevel the column or wall. After completion of the excavation, a permanent connection between the building and foundation is re-established. Since this method does not transfer foundation loads to a lower stratum, both shoring and permanent walls must be designed for surcharge loads imposed by the existing structure.

6.3.2 Underpinning Considerations

The need to underpin and the appropriate type of underpinning for specific buildings adjacent to the proposed excavation depend on many factors related to both engineering and economics and cannot be generalized. Thus each structure needs to be evaluated separately. The following discussions and evaluations are presented strictly from an engineering standpoint. Economic considerations are beyond the scope of this investigation.

From an engineering standpoint, the need to underpin is evaluated on the basis of expected ground movements and potential for structural damage. Figure 6-1 presents general guidelines for evaluating if a structure may be within the influence zones of the excavation; however, further evaluation of expected ground movements should be made based upon the type of shoring proposed. Section 6.4.5 discusses the anticipated ground movements in the vicinity of the excavation due to shoring movement. A conservatively designed shoring system (higher design lateral pressures) could be constructed to reduce ground movements due to shoring and thereby reduce the need to underpin.

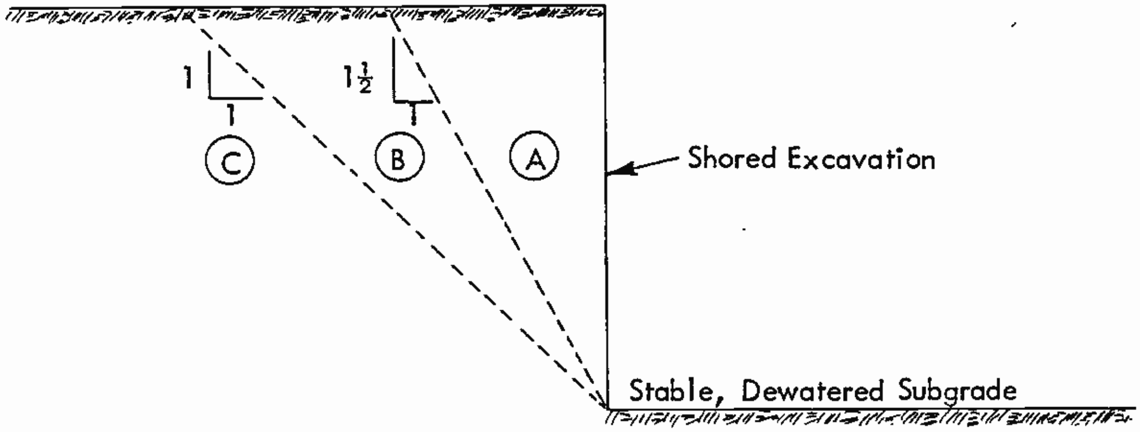
Due to contributing factors discussed in sections 6.1 and 6.2, if site dewatering is performed, the need to underpin and possible effects on and of underpinning should be carefully evaluated. Dewatering is expected to result in areal subsidence extending for hundreds of feet beyond the excavation limits. Effects of areal subsidence would include downdrag forces on underpinning piles and possible differential settlement between underpinned foundations and non-underpinned foundations. If dewatering is planned, underpinning should be avoided if possible, i.e., conservative shoring, or the effects of subsidence on the underpinned structure should be accommodated in the design. The "column pick-up" method described in 6.3.1 may be better adapted to the condition of areal settlement than the more conventional underpinning methods.

6.3.3 Design Criteria

Figures 6-2 through 6-4 present design criteria for jacked piles and slant drilled piles (without downdrag loads). Figure 6-2 illustrates the procedures for determining the geometry of the support zones and the total capacity of the underpinning pile. No support should be allowed within any existing fill soils encountered or within the "no support" zone shown on Figure 6-2. Figures 6-3, and 6-4 present design parameters for underpinning based on the expected subsurface conditions at the Wilshire/La Brea Station.

If jetting or other methods which remove soil ahead of the pile are used, no shaft frictional resistance should be allowed. To ensure proper end bearing, jetting must not be used for the final 5 feet of penetration. Group action of piles or piers should be considered and an appropriate reduction factor applied to determine the effective group capacity. An appropriate reduction factor is presented in the Los Angeles City Building Code, Section 91.2808b.

Total capacity of hand-dug, lagged piers should be limited to end bearing only and must extend below the "no support" zone shown on Figure 6-2. All piers are assumed to be 36-inch square or larger in section. For design, an allowable bearing pressure of 7 ksf may be used for piers which bear on undisturbed



NOTE: 1. These guidelines are applicable only for stable ground conditions. Other conditions would require special evaluation.

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

(A) 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.

(B) Generally No Special Provisions Required:

Properly designed shoring system generally adequate without underpinning unless underlain by poor soils or adjacent to especially sensitive structures.

(C) No Special Provisions

Approved for publication 4/84 by [signature]

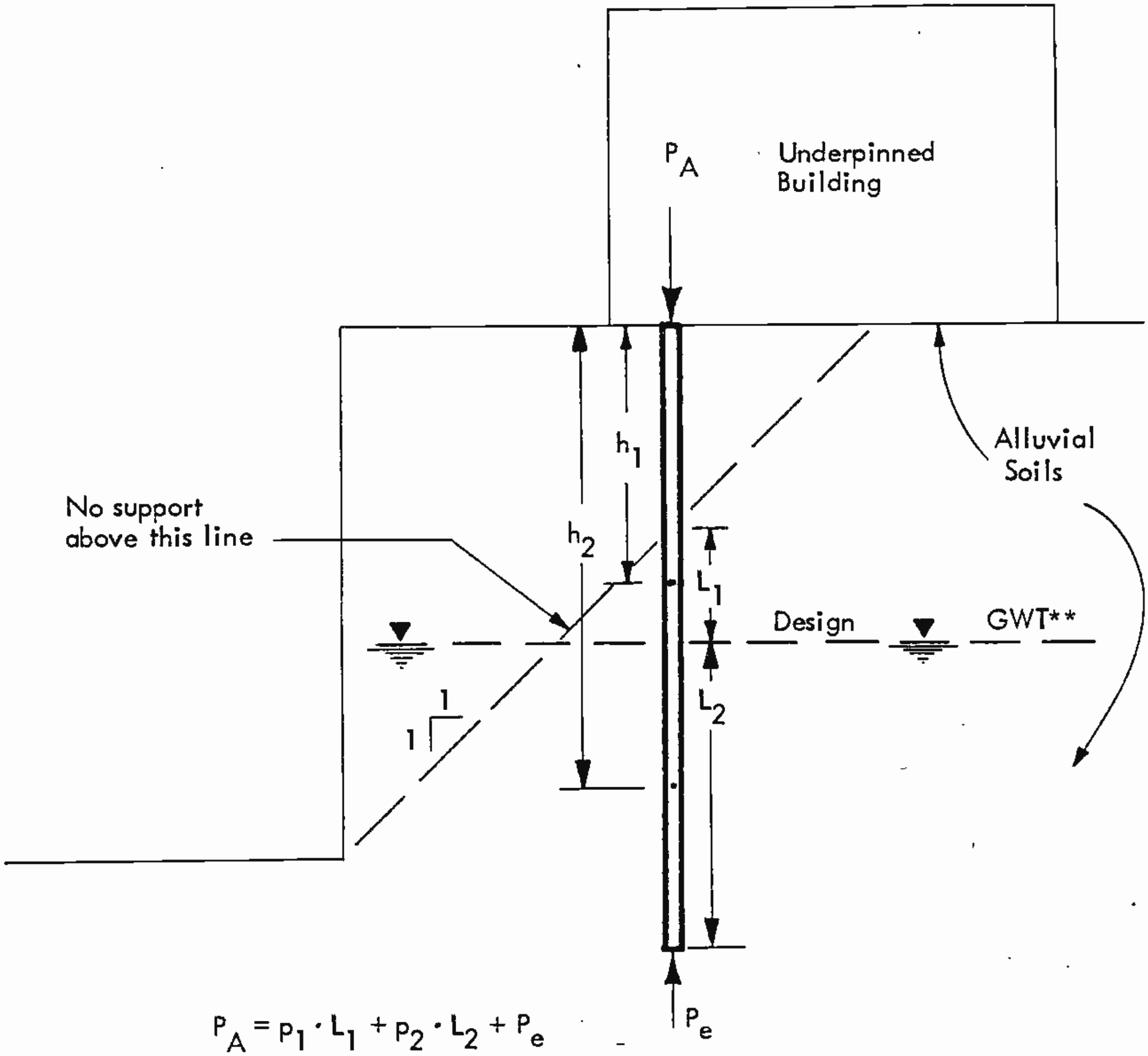
UNDERPINNING GUIDELINES

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 6-1



$$P_A = p_1 \cdot L_1 + p_2 \cdot L_2 + P_e$$

WHERE: $^*p_1$ = average frictional resistance at h_1
$^*p_2$ = average frictional resistance at h_2
$^{**}P_e$ = end resistance

* See Figure 6-3 through 6-6 for values of p_1 and p_2.

** For alluvium use P_e values given on Figure 6-3 and 6-4.
For San Pedro Sands use $P_e = q \times A_e$
where: $q = 80D$ (ksf)
D = pile diameter (ft.)
A_e = pile tip area (ft.^{2})

UNDERPINNING - DESIGN CAPACITY CRITERIA

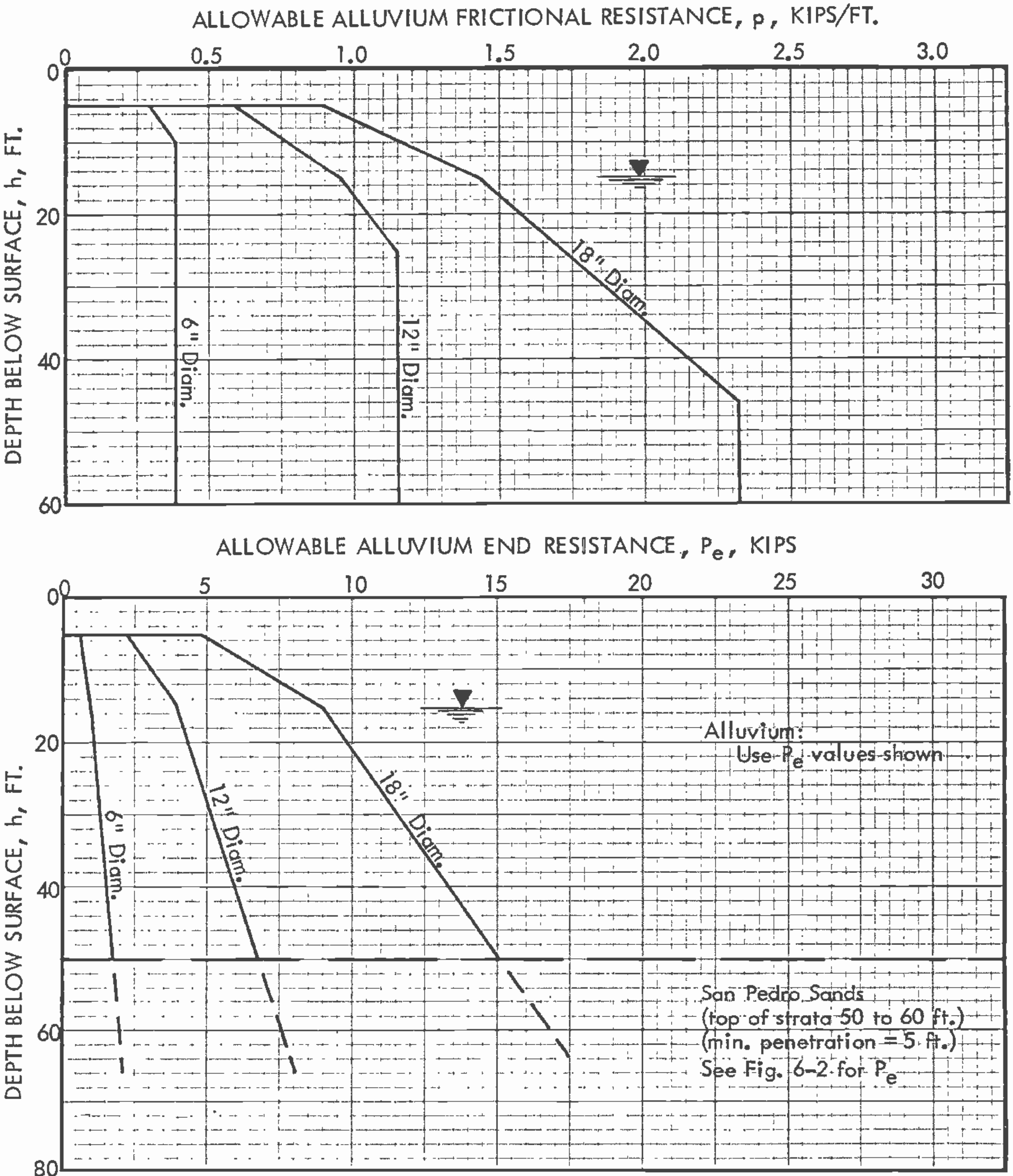
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No 83-1140

Figure No. 6-2

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 3/84 by [signature]



NOTES: 1) See Figure 6-2 for Determination of P_e in San Pedro Sands and Total Capacity.
2) Jacked piles are assumed to be circular pipe piles filled with concrete.

UNDERPINNING - JACKED PILE DESIGN PARAMETERS

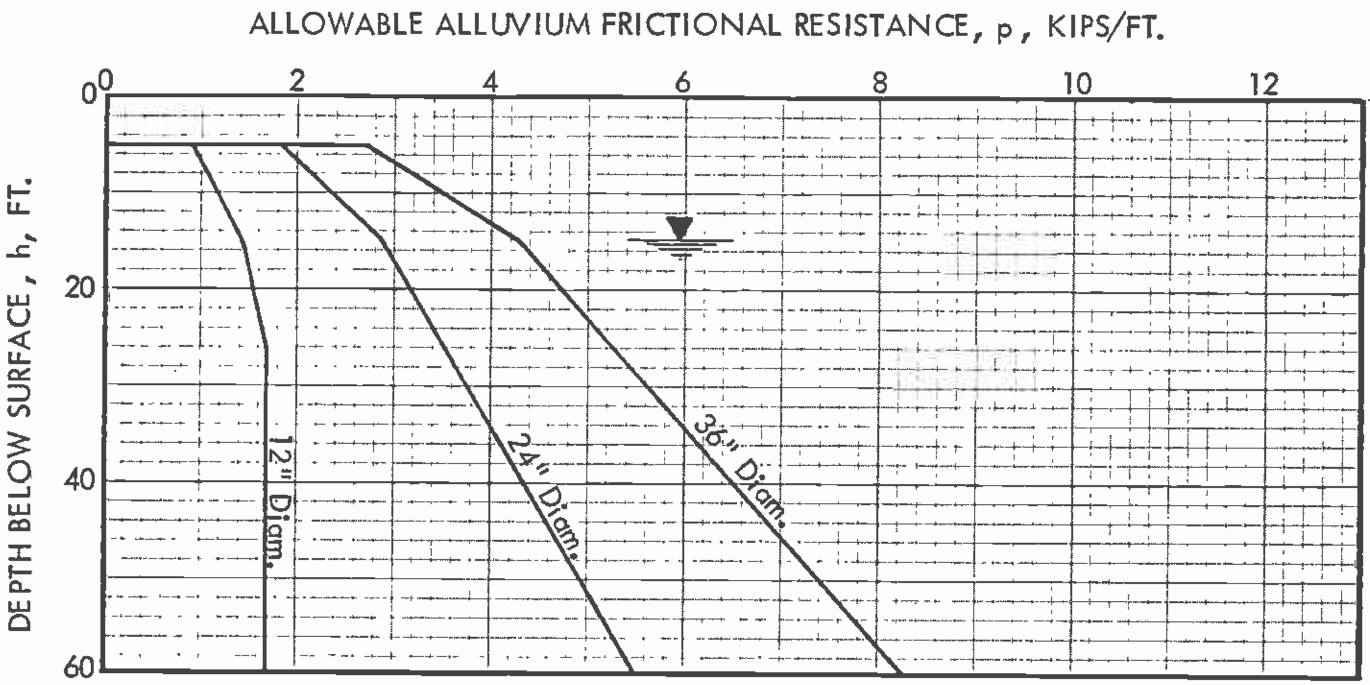
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

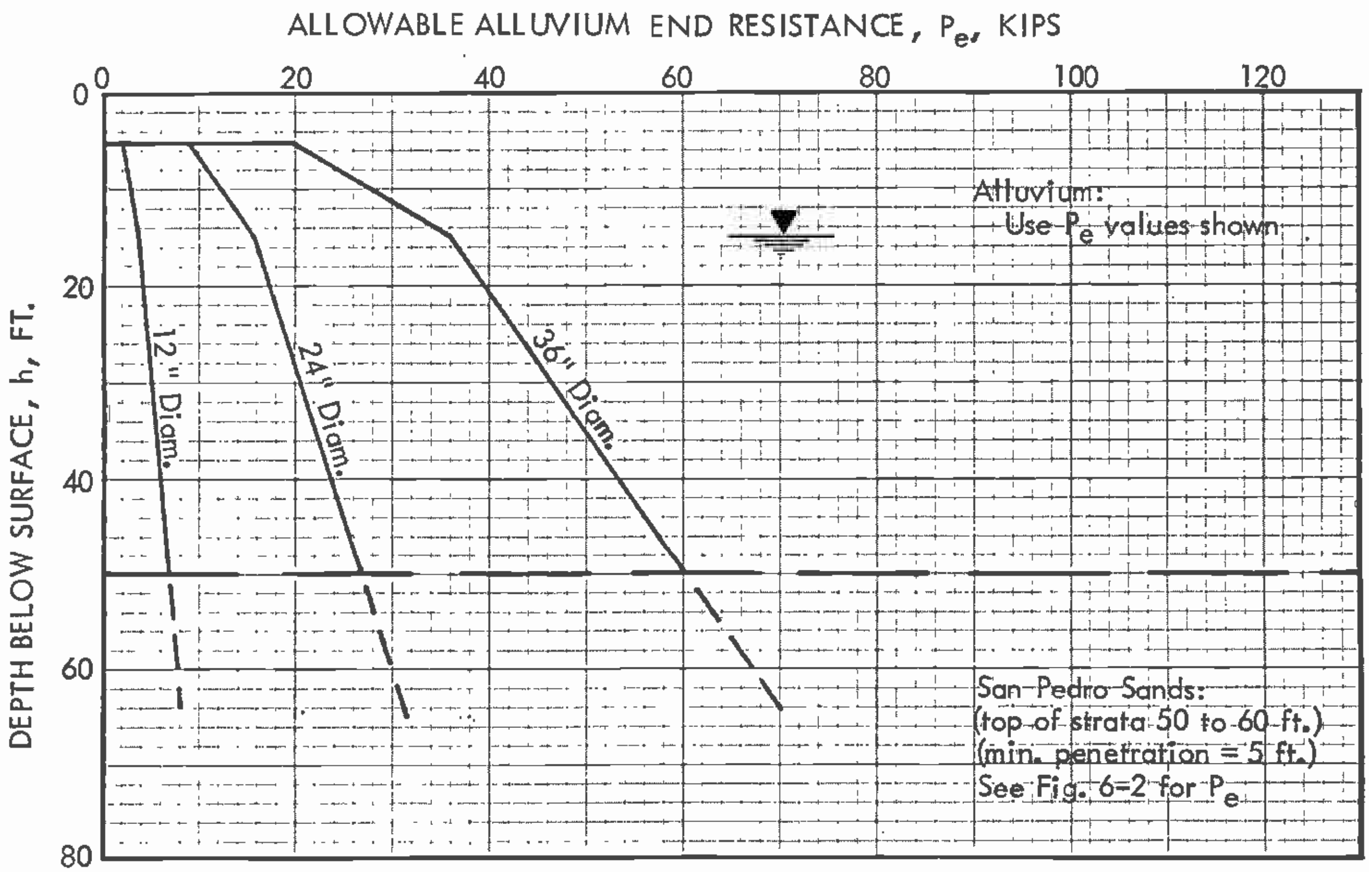
Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. 6-3

Approved for publication ______ by ______





See Figure 6-2 for Determination of P_e in San Pedro Sands and Total Capacity.

UNDERPINNING-CAST-IN-PLACE PILE DESIGN PARAMETERS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No.
6-4

Approved for publication 4/84 by [signature]

alluvium and penetrate at least 10 feet below the ground surface. For piers which penetrate at least 5 feet into the San Pedro sand but are at least 5 feet above the bedrock surface, an allowable bearing pressure of 20 ksf may be used. Piers bearing on bedrock may be designed based on 15 ksf. These values apply only if the bearing surface is properly prepared and approved by a qualified engineer.

Surface subsidence due to dewatering and lateral ground movements adjacent to the excavation are discussed in Sections 6.2.1 and 6.4.5, respectively. The capability of the existing structure and underpinning system to sustain these movements should be evaluated. If dewatering is planned, the effects of downdrag due to surface subsidence should be included in underpinning design. For computation of downdrag loads, the following procedure may be used:

1. The upper 3/4 of the alluvium thickness (including soils within the "no load" zone) should be assumed to be the downdrag zone. The alluvium thickness may be estimated from Drawing 4 and <u>should not include</u> the San Pedro Sands.

2. No positive (upward) frictional resistance should be used in the downdrag zone, instead a negative (downward) frictional load equal to twice the allowable frictional resistance within the zone (as determined from Figures 6-3 and 6-4) should be added to the design load.

   The negative frictional load is based on full soil strength (safety factor = 1.0) while the positive allowable frictional resistance is based on a safety factor of 2.0.

6.3.4 Underpinning Performance

Underpinning is not a guarantee that the structure will be totally free from either settlement or lateral movement. Some settlement may occur during the underpinning process. Additional vertical and/or lateral movement may occur during the construction of the main excavation, depending on the performance of both the shoring and underpinning elements. Effects of subsidence may result in differential settlements between underpinning elements and non-underpinned elements.

6.3.5 Underpinning Instrumentation

Prior to construction, elevation reference points should be established on each foundation element to be underpinned. The points should be monitored on a regular basis consistent with the construction progress (readings may be required daily). Maximum allowable movements should be established for each element by the engineer prior to underpinning. If it appears that these limits may be exceeded, immediate measures should be taken such as restressing underpinning elements, adding more supports or changing installation procedures.

Where a group of three or more jacked piles is used to underpin a foundation element, load relaxation of previously installed piles can occur. Methods should be implemented to evaluate this problem and re-load piles if necessary.

6.4 TEMPORARY EXCAVATIONS

6.4.1 General

The required A245 station excavation will extend approximately 50 to 55 feet below the existing ground surface and 35 to 40 feet below the water table. A primary consideration in the selection of the shoring system should be the effects of dewatering as discussed in Sections 6.1 and 6.2. Dewatering of the site may result in areal subsidence in the site vicinity which could cause downdrag and differential settlements of underpinned structures. However, this condition could be mitigated by a conservatively designed shoring system which could minimize underpinning or by a "tight" shoring system which could eliminate the need for site dewatering. There are several currently used shoring methods which include soldier piles and lagging, slurry wall construction and sheet piles. Bracing systems are generally either tieback anchors or internal bracing. We understand that the excavation 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.

The fine-grained alluvial soils at the site will generally be favorable for construction of shoring systems. However, caving may occur within the zones of granular alluvium and within the San Pedro Sands. In addition, gravel and cobble zones may be encountered, especially near the base of San Pedro Sand.

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

6.4.2 Soldier Pile Shoring Systems

A soldier pile and lagging shoring system consisting of soldier piles installed in predrilled holes is a common method of shoring deep excavations in the Los Angeles area. Both conventional and conservative soldier pile shoring systems may be used at the station site. A conservative wall would be designed for higher soil loads to reduce ground movements behind the wall.

Soldier piles have been installed in the Los Angeles area in soils similar to those encountered at the proposed A245 Station site. In granular soils, particularly below the ground water table, caving can be a problem. The contractor should recognize that caving conditions may be encountered in construction of soldier piles or other drilled shaft elements.

Granular soil layers within the alluvium at the site will require support between soldier piles to eliminate loss of ground. Typically, wooden lagging is used although precast concrete or steel panels could also be used.

6.4.3 Shoring Design Criteria

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

Specific shoring design criteria include:

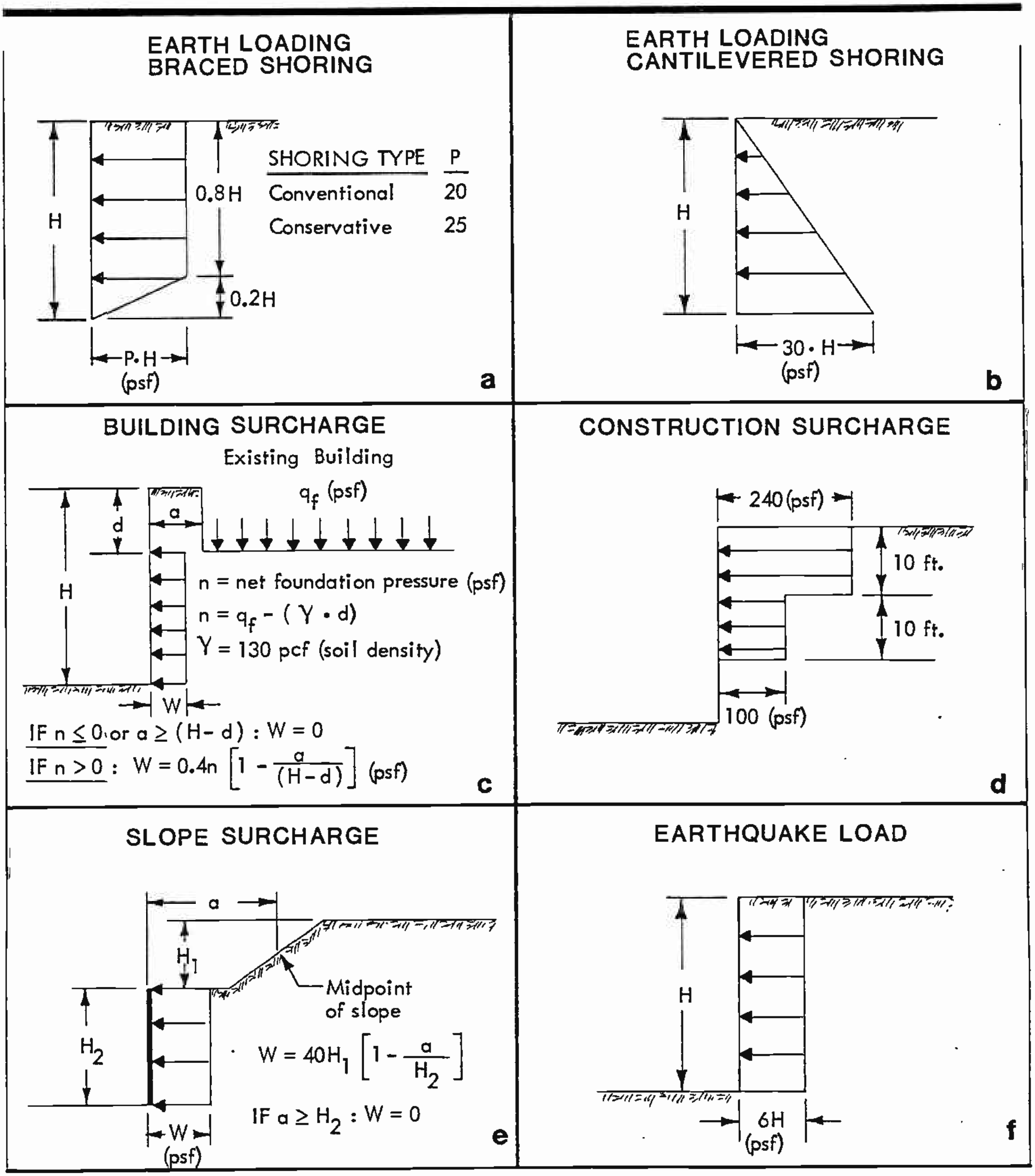
- Design Wall Pressure: Figures 6-5a and 6-5b present the recommended lateral earth pressure on the temporary shoring walls. Design lateral pressures for both conventional and conservative shoring systems are presented in Figure 6-5a. Figure 6-5e also includes the case of partial slope cuts. Appendix D.2 provides technical support for the recommended seismic pressures of Figure 6-5f. The full loading diagram above the bottom of excavation should be used to determine the design loads on tieback anchors and the required depth of embedment of the soldier piles. For computing design stresses in the soldier piles, the computed values can be multiplied by 0.8. For sizing lagging, the earth pressures can be reduced by a factor of 0.5.

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

  The required depth of embedment to satisfy vertical loading should be computed based on allowable vertical loads shown on Figure 6-6. Maximum depth of penetration restrictions shown on Figure 6-6 is based on consideration of the depth to bedrock below the excavation.

  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-5 minus the support from tiebacks or internal bracing. The required depth of embedment to satisfy lateral loads should be computed based on the net allowable passive resistance (total passive resistance of the soldier pile minus the active earth pressure below the excavation). Due to arching effects, it is recommended that the effective pile diameter be assumed equal to 1.5 pile diameters or half of the pile spacing, whichever is less. Figure 6-7 indicates the recommended method to compute net passive resistance.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. At the A245 Station site the alluvial soils encountered were generally clayey.



NOTE: All distances should be in feet.

LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

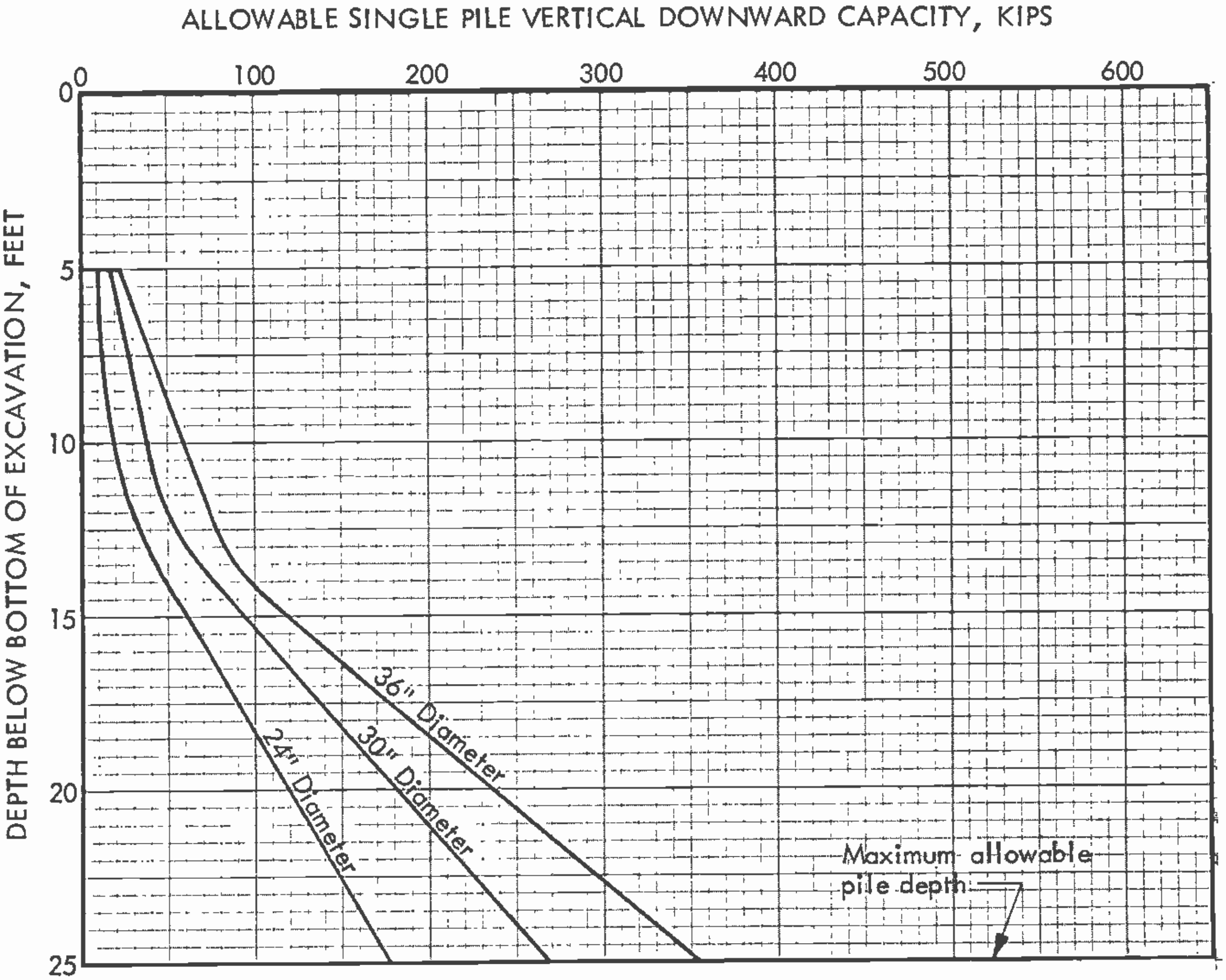
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. 6-5

Approved for publication ______ by ______



NOTES:

1) All piles must penetrate at least 5 feet into the San Pedro Sand.
2) Pile tips should not penetrate more than the indicated maximum depths below the bottom of excavation.
3) For seismic design, capacities may be increased 33%.
4) Water level assumed at the bottom of excavation.
5) Piles are assumed to be cast-in-place concrete.

Approved for publication 4/84 by JAD

VERTICAL CAPACITY OF PILES FOR SHORING AND DECKING

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

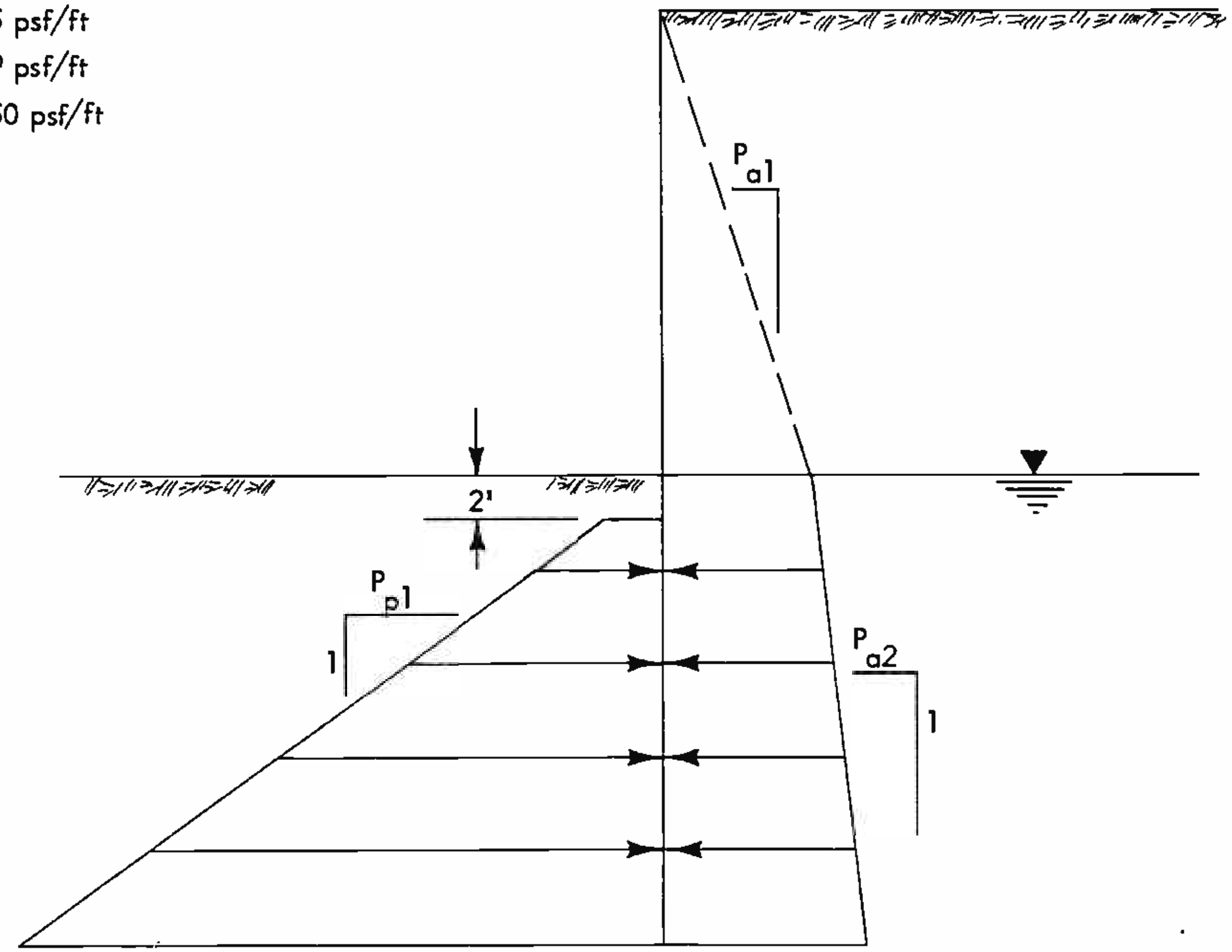
Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 6-6

Recommended Unit Pressures

P_{a1} = 35 psf/ft
P_{a2} = 19 psf/ft
P_{p1} = 350 psf/ft



Where: P_p = Total Allowable unit passive pressure

P_a = Unit Active pressure

NOTE:
1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive - Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-5

Approved for publication 4/84 by [signature]

SOLDIER PILE PASSIVE RESISTANCE

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. 6-7

Converse Consultants Geotechnical Engineering and Applied Sciences

However, occasional granular layers may be exposed and these soils 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 be placed to minimize ravelling of soils and loss of ground between soldier piles. The contractor should limit the temporarily exposed soil height to less than 3 feet to control ravelling problems, especially in the dewatered zone.

- 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.

6.4.4 Internal Bracing and Tiebacks

6.4.4.1 General: Tiebacks and/or internal bracing may both be suitable to support the temporary shoring wall for the proposed excavation. Tiebacks have the advantage of producing an open excavation which can significantly simplify the excavation procedure and construction of the permanent structure. However, there may be an opportunity to install used pipe and WF sections from other projects as struts and to salvage these for use elsewhere. This often makes the employment of internal bracing more attractive to the contractor than tiebacks. Obtaining permission to install tiebacks under adjacent properties and encountering obstructions from adjacent below grade structures (such as basements) can also affect the economics and feasibility of tiebacks.

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

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

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

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

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to at least 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.

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

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

$$P = \pi DLq$$

Where:

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

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

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

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

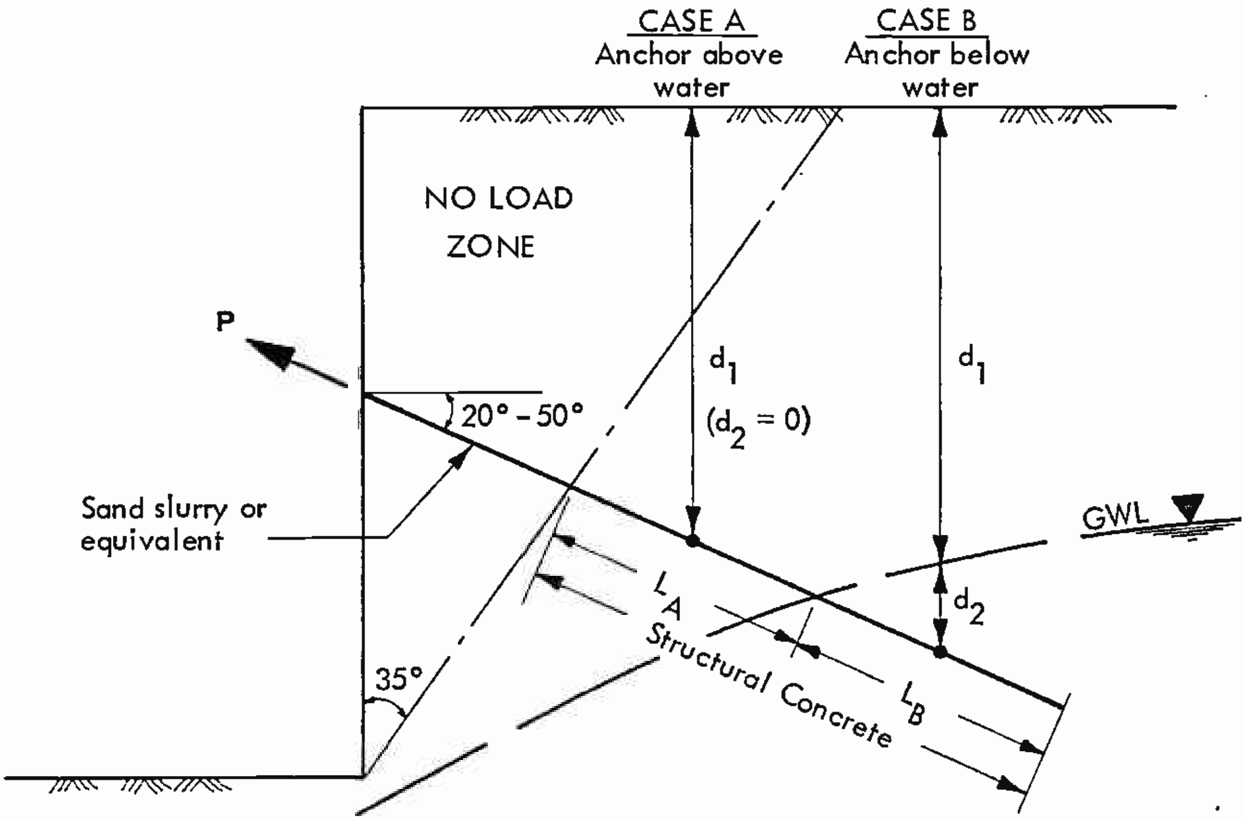
Where:

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

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

Figure 6-8 illustrates the tieback anchor 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.



NOTE:

The design adhesion value, q, can be evaluated by

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

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

The total anchor capacity can be estimated by:

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

See also Section 6.4.4.4 for further discussion

Approved for publication 3/8/84 by JAD

STRAIGHT SHAFT TIEBACK ANCHOR CAPACITY

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 6-8

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

The anchors may be installed at angles generally between 20° to 50° below the horizontal. Based on specific site conditions, these limits could be expanded to avoid underground obstructions. Structural concrete should be placed in the lower portion of the anchor up to the limit of the no-load zone. Placement of the anchor grout should be done by pumping the concrete through a tremie or pipe extending to the bottom end of the shaft. The anchor shaft between the no-load zone and the face of the shoring must be backfilled with a sand slurry or equivalent after concrete placement. Alternatively, special bond breakers can be applied to the strands or bars in the no-load zone and the entire shaft filled with concrete.

For tieback anchor installations, the contractor should be required to use a method which will minimize loss of ground due to caving. The majority of the anchors should not experience significant caving problems. However, caving from sand layers within the alluvium could occur due to vibration from the drilling equipment and/or ground water effects. Caving problems should be expected where anchors penetrate sands below the water table. Caving not only causes installation problems but could result in surface subsidence and settlement of overlying buildings. To minimize caving, casing could be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, the hole could be maintained full of slurry or a hollow stem auger could be used.

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.

6.4.5 Anticipated Ground Movements

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

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

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

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

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

6.4.6 Historical Shoring Pressure Diagrams - Los Angeles

Appendix E.1 summarizes the design shoring pressures for nine shoring systems in the Los Angeles vicinity. To our knowledge there are no data on field measurements of actual lateral soil pressures for shored excavations in the Los Angeles area and, therefore, the design pressures of Appendix E.1 have not been directly verified.

6.5 SUPPORT OF TEMPORARY DECKING

Where temporary street decking requires center support piles, the piles should extend below the maximum proposed excavation level for support. At these depths, the piles would be founded within the San Pedro layer or the bedrock. These materials are suitable for supporting such pile loads.

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

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed A245 Station excavation should be instrumented to reduce liability (by having documentation of performance), to validate design and construction requirements, to identify problems before they become critical, and to obtain data valuable for future designs.

We recommend the following instrumentation program:

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

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

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

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

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

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

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

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

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

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

- 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 and immediately reported as frequently as necessary to determine the behavior being monitored. For ground movements this should be no greater than one to two-week intervals during the major excavation phases of the work. Strut load measurements should be more frequent, possibly even daily, when significant construction activity is occurring near the strut (such as excavation, placement of another level of struts, etc.).

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

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

6.7 EXCAVATION HEAVE AND SETTLEMENT OF THE STATION STRUCTURES

The proposed A245 excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed 50- to 55-foot excavation will decrease the net vertical ground stresses by about 4800 psf. Stress reduction caused by the excavation will result in rebound or heave of the alluvium and bedrock below the excavations. This response is not due to the occurrence of any swelling type of soils but simply the response to stress unloading. In addition, even with a suitable shoring system, shear stresses will develop tending to cause the soil adjacent to the walls to heave upward. Since the excavations will be open for an extended period, the heave is expected to be completed prior to construction of the station. The station structure and subsequent backfilling will reload the soil. We estimate that the net station loads will be about 2000 to 4000 psf. Such a load will cause the ground to reconsolidate. Thus, even though the weight of the excavated soil exceeds the weight of the final structure, the structure will experience some ground settlement due to recompression of the elastic rebound.

We estimate that the maximum heave at the center of the excavation will be on the order of 1½ to 3 inches. We also believe that the majority of this will occur while the excavation is being made. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (consolidation heave) within the San Pedro Sand and bedrock underlying the proposed excavation. Due to the dense and hard consistency of the sand and bedrock, the majority of the deformation will be elastic rebound. These values agree well with observed behavior in similar excavations in the Los Angeles area (Evans, 1968).

It was computed that the estimated imposed loads from the structures and backfill will induce settlements on the order of 1 to 2-1/2 inches. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/3 inches over the width of the structures. This correlates to an angular rotation of only about 1:1100. Differential settlement between the alluvial supported east end and the San Pedro Sand supported west end could be one inch. However, the maximum longitudinal angular distortion is estimated to be only about 1:2400.

These calculations are based on a uniform foundation bearing pressure which could result only from a uniformly loaded and perfectly flexible structure. We understand that the station structure will be relatively stiff. Thus the actual differential settlement will be less than for the theoretical flexible foundation assumed.

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

6.8 FOUNDATION SYSTEMS

6.8.1 Main Station

It is understood that the proposed A245 Station will be supported on thick base slabs which will function as massive mat foundations. We estimate that the net mat foundation bearing pressures will be about 2000 to 4000 psf. In our opinion the station can be adequately supported on a mat foundation. Section 6.7 presents estimated settlements for the proposed station structure.

6.8.2 Support of Surface Structures

Surface structures can be generally supported on conventional spread footings founded on undisturbed stiff or dense natural soils. If suitable natural soils do not exist at the surface structure site, footings may be founded on a zone of properly compacted structural fill (see Appendix E). Allowable bearing pressures and estimated total settlements of spread footings bearing on the natural alluvium or compacted structural fill can be determined based on Figures 6-9 and 6-10. These figures are based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed information on structural loadings and site conditions at the surface structure location. Detailed site specific studies should be performed to provide final design recommendations for specific 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 6-9 and 6-10 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 6-9 and 6-10, 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 300 psf/ft may be used for the sides of footings poured neat against dense or stiff alluvium or properly compacted fill. The maximum passive pressure should not exceed 3000 psf. Frictional resistance at the base of foundations should be determined using a frictional coefficient of 0.35 with dead load forces.

6.9 PERMANENT GROUND WATER PROVISIONS

We understand that the stations will be designed to be water-tight and to resist the full permanent hydrostatic pressures. We recommend that the entire structure be fully waterproofed due to the high design water levels.

6.10 LOADS ON SLAB AND WALLS

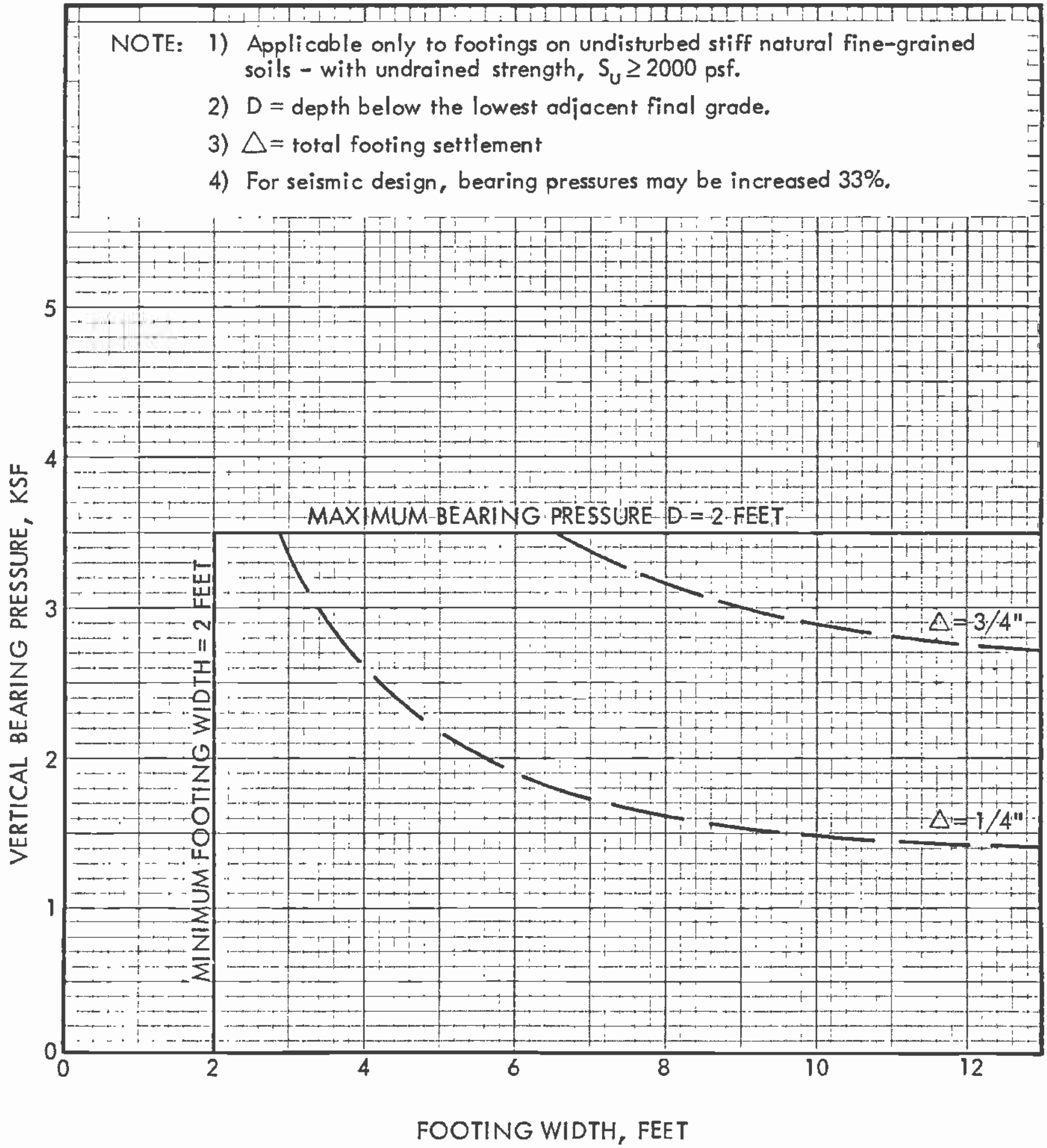
6.10.1 Hydrostatic Pressures

As discussed in Section 5.4, the existing ground water levels as measured at the boring locations were about Elevation 180 to 182 at the Wilshire/LaBrea site. The winter of 1983 was one of the five wettest years in the past 100 years and, therefore, the measured levels are considered to represent near maximum levels. It is recommended that the long-term design ground water level be assumed to be Elevation 187 for determining hydrostatic pressures.

6.10.2 Permanent Static Earth Pressures

Figure 6-11 presents lateral earth pressures recommended for design of permanent subsurface walls.

Vertical earth pressures on the roof should be assumed equal to the full moist and/or saturated weight of overburden soil plus surcharge, which is to be provided by the Section Designer.



Approved for publication 11/3/11 by FAD

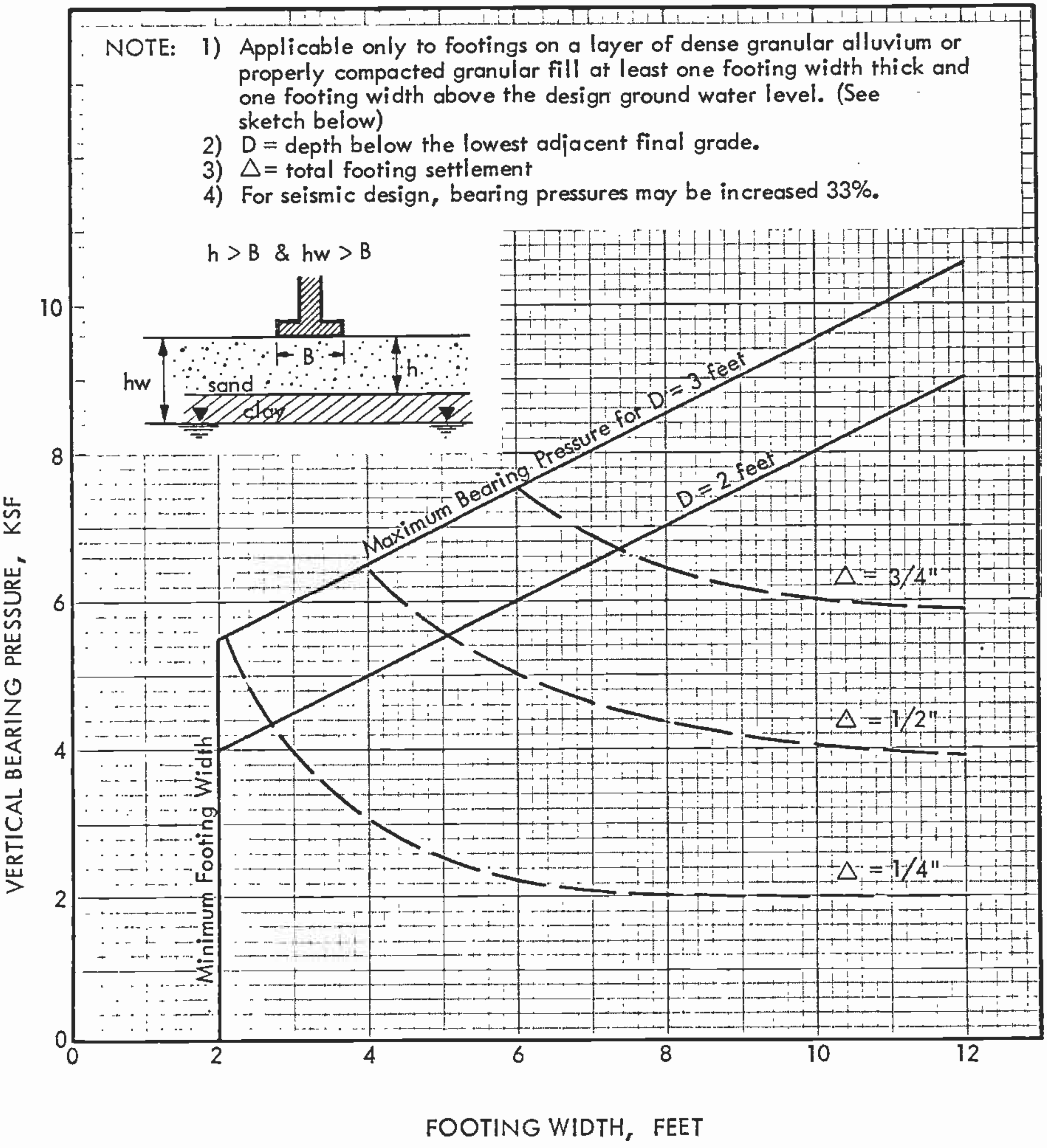
ALLOWABLE BEARING & SETTLEMENT FOR SPREAD FOOTING ON FINE-GRAINED SOILS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 6-9



Approved for publication 4/84 by [signature]

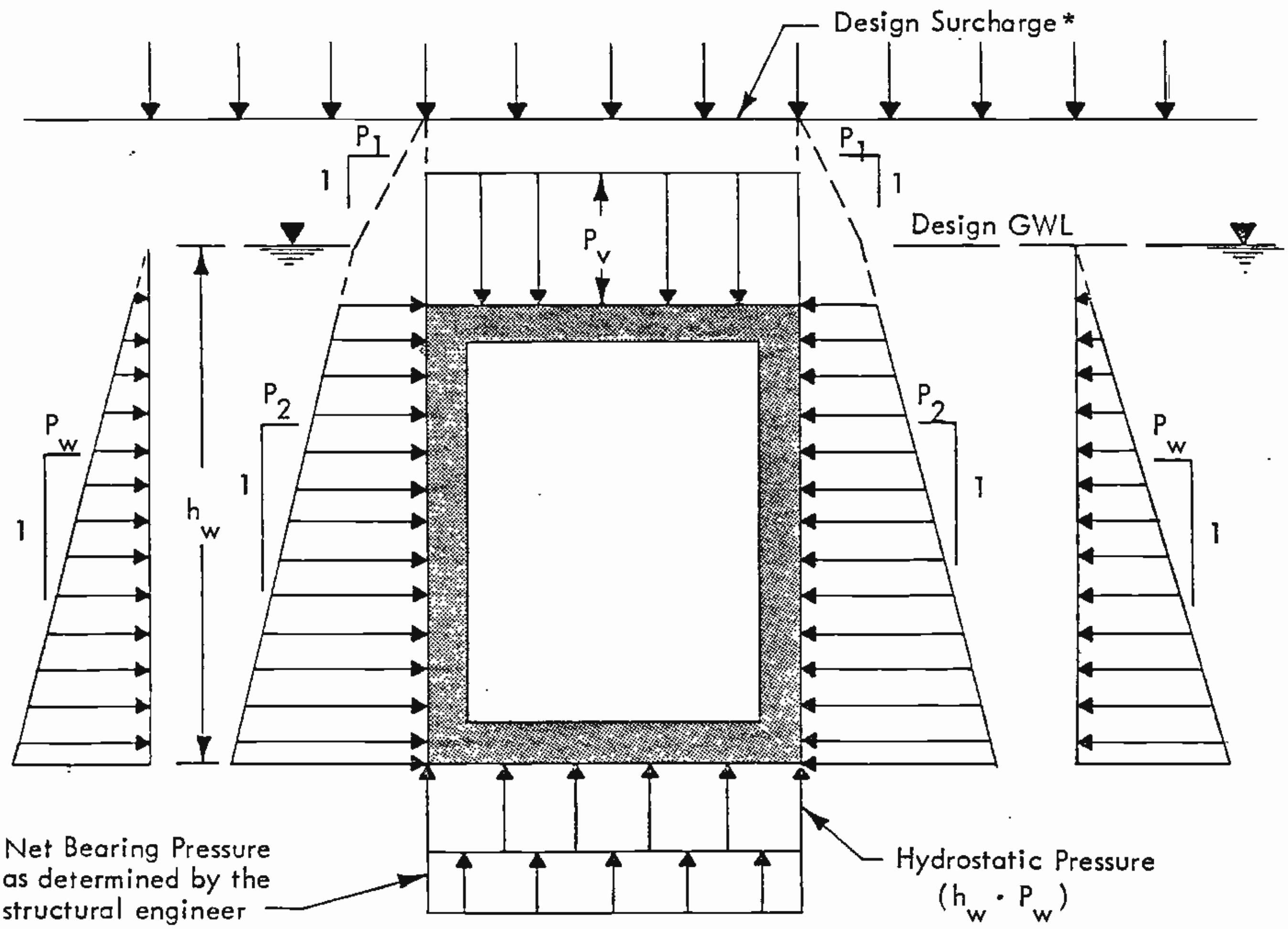
ALLOWABLE BEARING & SETTLEMENT FOR SPREAD FOOTING ON GRANULAR SOILS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. 6-10



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	30	16	62.4	*	**
Long Term	55	30	62.4	*	Elev. 187 ft.
Side sway †	30/55	16/30	62.4	*	**

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

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

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

Approved for publication 4/18/84 by [signature]

LOADS ON PERMANENT WALLS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. 6-11

6.10.3 Surcharge Loads

Lateral surcharge loads from existing buildings not underpinned must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-5. Vertical surcharge loads due to 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.

6.11 SEISMIC CONSIDERATIONS

6.11.1 General

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

6.11.2 Dynamic Material Properties

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

TABLE 6-1

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		ALLUVIUM	SAN PEDRO SAND	PUENTE BEDROCK
Average Compression Wave Velocity, V_c (ft/sec)	- moist	4000		5700
	- saturated	5000	5000	
Average Shear Wave Velocity, Vs (ft/sec)		1100	950	1300
*Poisson's Ratio		0.35	0.35	0.35
**Young's Modulus, E, (psi)	- moist	207,000		530,000
	- saturated	185,000	185,000	
**Constrained Modulus, E_c, (psi)	- moist	450,000		850,000
	- saturated	700,000	700,000	
**Shear Modulus, G_{max}, (psi)		34,000	25,000	45,000

* For saturated alluvium, use value of 0.45.

** All modulus values are for low strain levels (10^{-6}).

The compression and shear wave velocities are based on interpretation of limited downhole geophysical surveys performed in Boring CEG-18 and other borings in similar materials during the 1981 investigation. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish modulus values at low strain levels.

The variation of dynamic shear modulus, with shear strain is presented in Figure 6-12 for the various geologic units. Variation of the dynamic shear modulus is expressed as the ratio of the strain compatible modulus (G) to the very low strain modulus (G_{max}). Similar relationships for soil hysteretic damping are presented in Figure 6-13. The modulus and damping curves are based on dynamic laboratory tests performed during our 1981 investigation. Dynamic test results are presented in Vol. II, Appendix H of our 1981 report.

6.11.3 Liquefaction Potential

The generalized subsurface cross section has been described in Section 5.0 and is shown in Drawings 2 and 4. The ground water levels were at about Elevation 180 to 182. These ground water elevations correspond to a depth of about 15 feet below the ground surface. The soils which are below the ground water level and, therefore, must be evaluated for liquefaction potential include the alluvial soils and the San Pedro Formation Sand.

Our liquefaction evaluation was based on procedures and correlations published by Seed et al (1983) which utilized index soil properties and performance data for soils during previous earthquakes. Field Standard Penetration Tests (SPT), available field geophysical data from CEG-18, and laboratory classification test data were all used in our evaluation of liquefaction potential (see Appendix E).

The referenced procedures include correlations of SPT data and liquefaction potential for granular soils. Measured SPT "N" values in the San Pedro Sands were all greater than 100 blows (refusal) and, therefore, these materials are considered to have a very low liquefaction potential even under the maximum design earthquake. Corrected "N" values (normalized to 2 ksf overburden pressure) for 12 SPT tests in saturated granular alluvium ranged from 29 to 60 with an average of about 45. Determination of dynamic strength was based on an M7.0 (maximum design) earthquake event. Based on these SPT values, the potential for liquefaction of the granular alluvium was considered low.

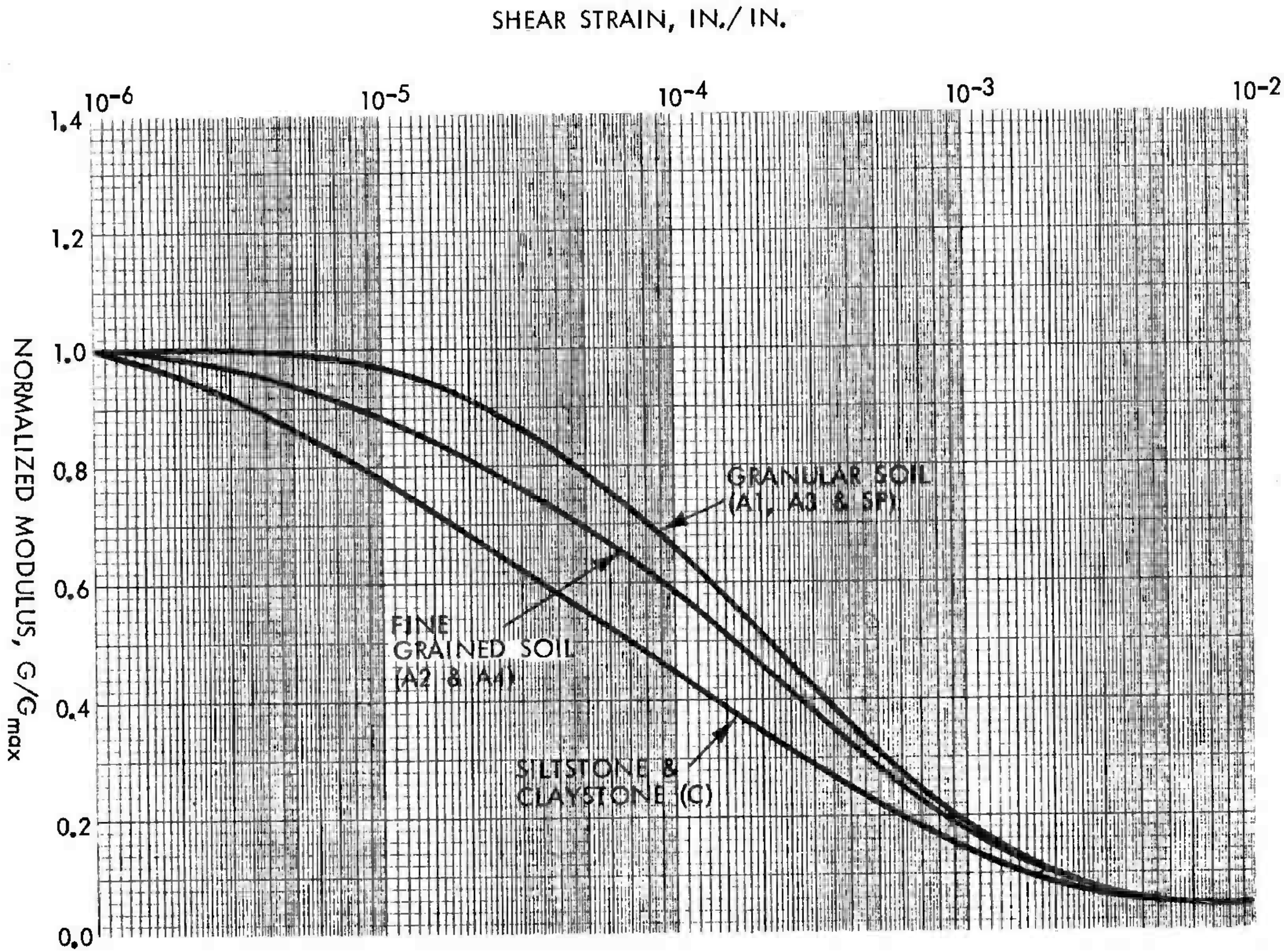
Clayey soils are generally considered non-liquefiable, but there are correlations between classification tests (Atterberg Limits, moisture content,and grain size distribution) and liquefaction potential of clayey soils. Index property tests of the clayey alluvium compared with index properties of soils vulnerable to liquefaction indicated these materials to be essentially non-liquefiable.

Considering the above discussed results, it is our opinion that the potential for liquefaction at the A245 Station site is low for the maximum design earthquake.

6.12 EARTHWORK CRITERIA

Site development is expected to consist primarily of excavation for the subterranean structure but will also include general site preparation, foundation

Approved for publication 4/84 by



Converse Consultants
Geotechnical Engineering and Applied Sciences

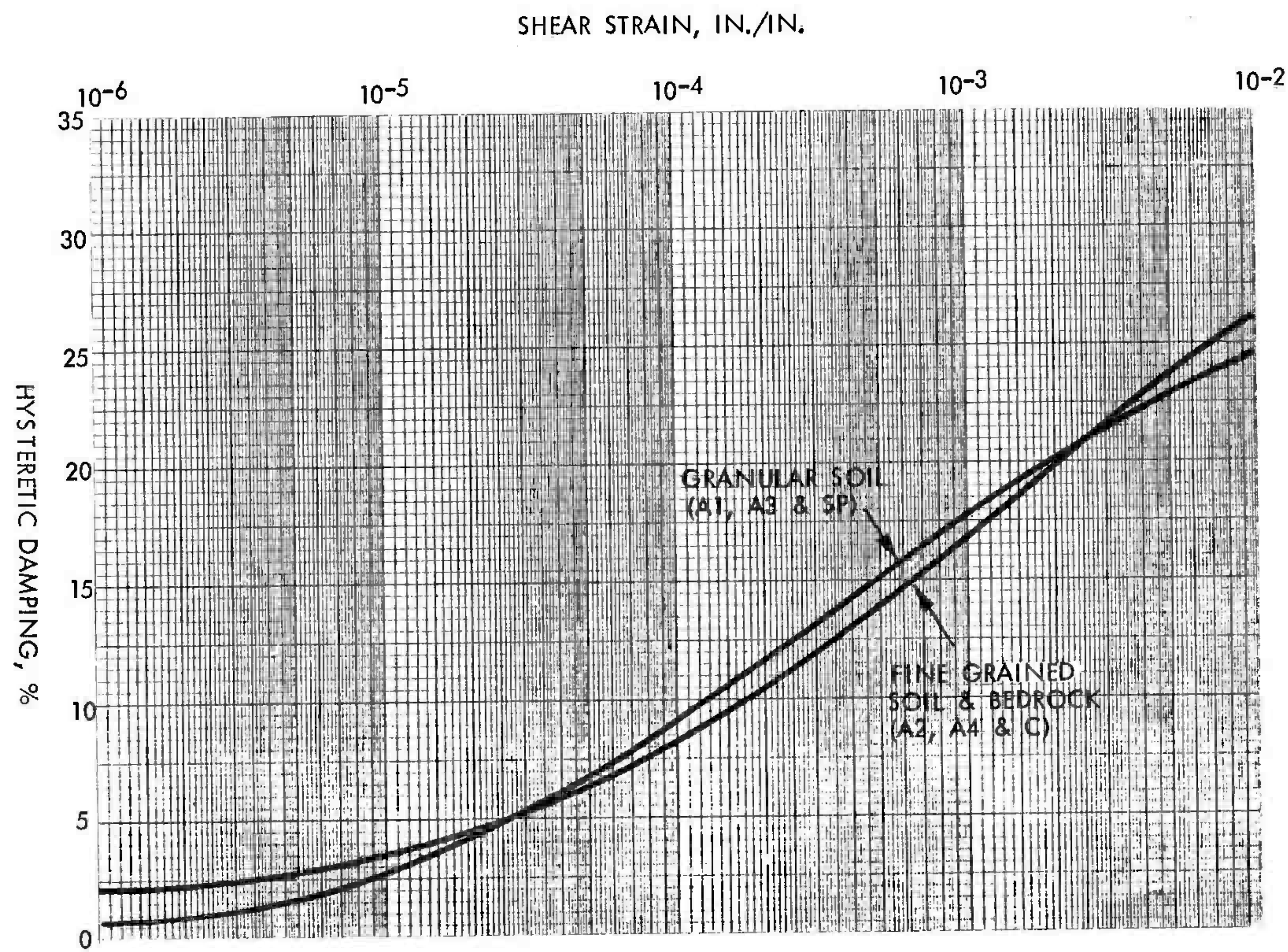
RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. 6-12

Approved for publication 4/84 by [illegible]



Converse Consultants
Geotechnical Engineering and Applied Sciences

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. 6-13

preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations and dewatering are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill 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, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. The excavated existing fills, fine-grained soils and bedrock material are not considered suitable because these fine-grained materials will make compaction difficult and could lead to fill settlement problems after construction. If the granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the geotechnical engineer.

6.13 PAVEMENT DESIGN

Minimum flexible pavement sections for assumed Traffic Index (TI) values of 5.0, 7.0 and 9.0, and a subgrade R-value of 40 were developed using CALTRANS design method. Pavement sections provided below include the recommended thickness of compacted subgrade, base course and asphaltic concrete for the three Traffic Index values.

ASSUMED TRAFFIC INDEX (TI)	THICKNESS (in inches): A.C. with Base Course – A.C.	A.C. with Base Course – Base Course	Full Depth Asphaltic Concrete	Compacted Subgrade (R ≥40)
5.0	2.0	6.5	4.5	24.0
7.0	3.0	8.5	7.0	36.0
9.0	4.0	11.0	9.5	36.0

We understand that the City of Los Angeles requires a minimum pavement section along major streets (such as Wilshire Boulevard) consisting of 8 inches of asphaltic concrete over 12 inches of base course. Therefore, the City of Los Angeles should be consulted regarding final selection of the replacement pavement sections.

Subgrade soil preparation should include processing of any disturbed subgrade areas, and excavation and replacement as required to provide a properly compacted subgrade of select granular material ("R" Value ≥40) to the depths indicated above. Subgrade fill compaction should be performed in accordance with recommended specifications presented in Appendix F.

Base course material should be Type II aggregate base conforming with Section 26-1.023 of CALTRANS' Standard Specifications (1978).

6.14 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- Pump Test: It is recommended that a pumping test be performed at the A245 Station site to evaluate the pumping and dewatering characteristics. The test well should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

- 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 stations. The purpose of these studies would be to determine site specific subsurface conditions and provide site specific final design recommendations for these peripheral structures.

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

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.

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.

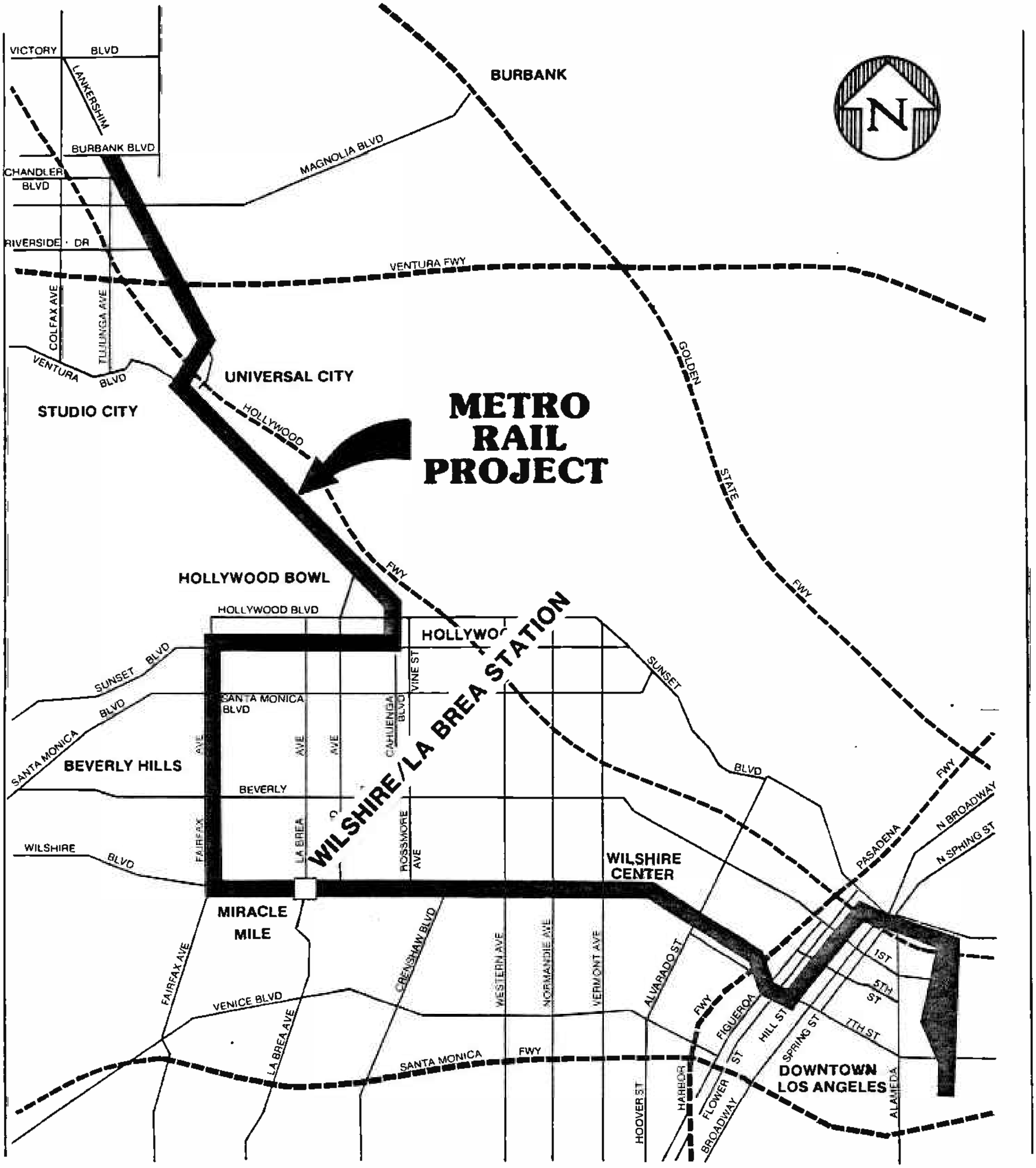
SEED, H.B, and Whitman, R.V., 1970, Design of earth retaining structures for dynamic loads in Ground and Design of Earth Retaining Structures: ASCE Specialty Conference on Lateral Stresses New York, p. 103-148.

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

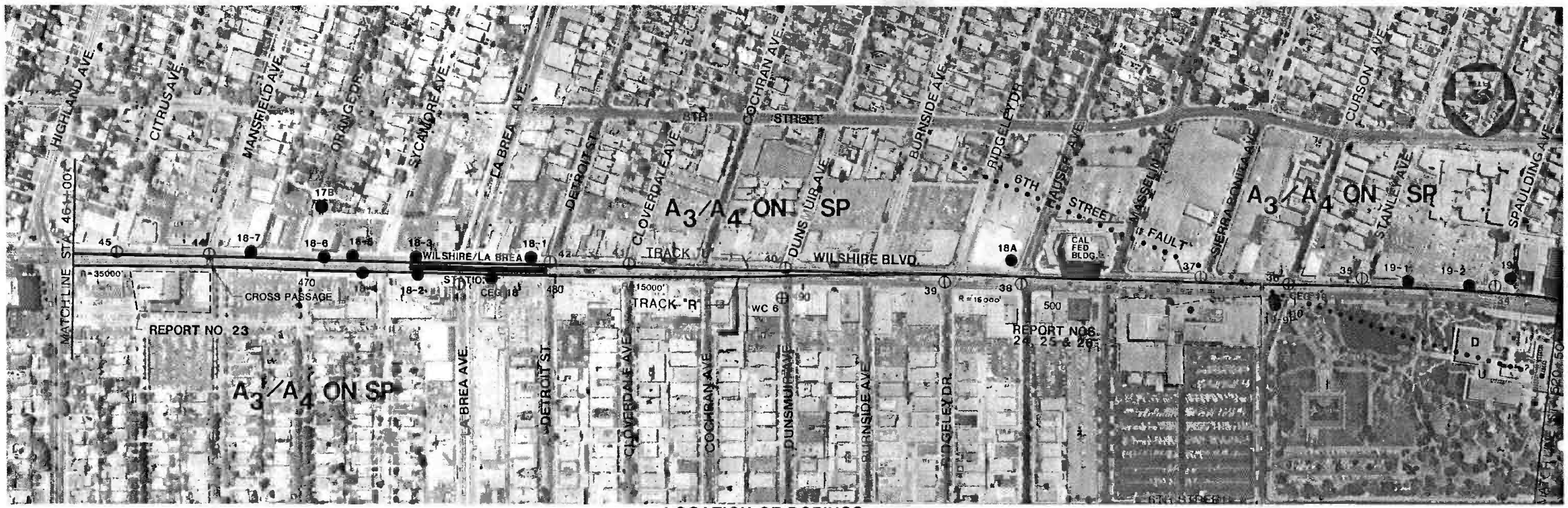
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No **83-1140**

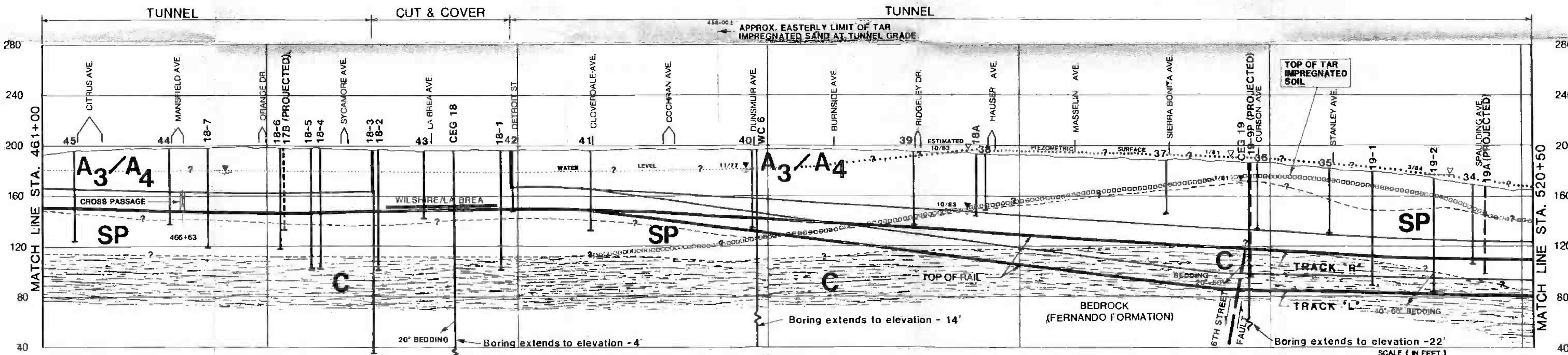
Drawing No **1**

Converse Consultants — Geotechnical Engineering and Applied Sciences

D-10 Approved for publication 4/84 by [handwritten initials]



LOCATION OF BORINGS



GEOLOGIC SECTION

REFERENCE:

MILESTONE 10 SHEET 9 OF 21 ALIGNMENT PLAN & PROFILE STATION 461+00 TO STATION 520+50 DATED NOV. 1983

NOTE:

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

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

3.) THIS DRAWING WAS PREPARED AS AN AID IN DEVELOPING DESIGN RECOMMENDATIONS. SUBSURFACE INFORMATION PRESENTED ON THIS DRAWING IS BASED ON INTERPOLATION AND EXTRAPOLATION OF SUBSURFACE DATA BETWEEN AND BEYOND BORING LOCATIONS. ACTUAL CONDITIONS ENCOUNTERED DURING CONSTRUCTION MAY BE DIFFERENT.

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP	DESCRIPTION

DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

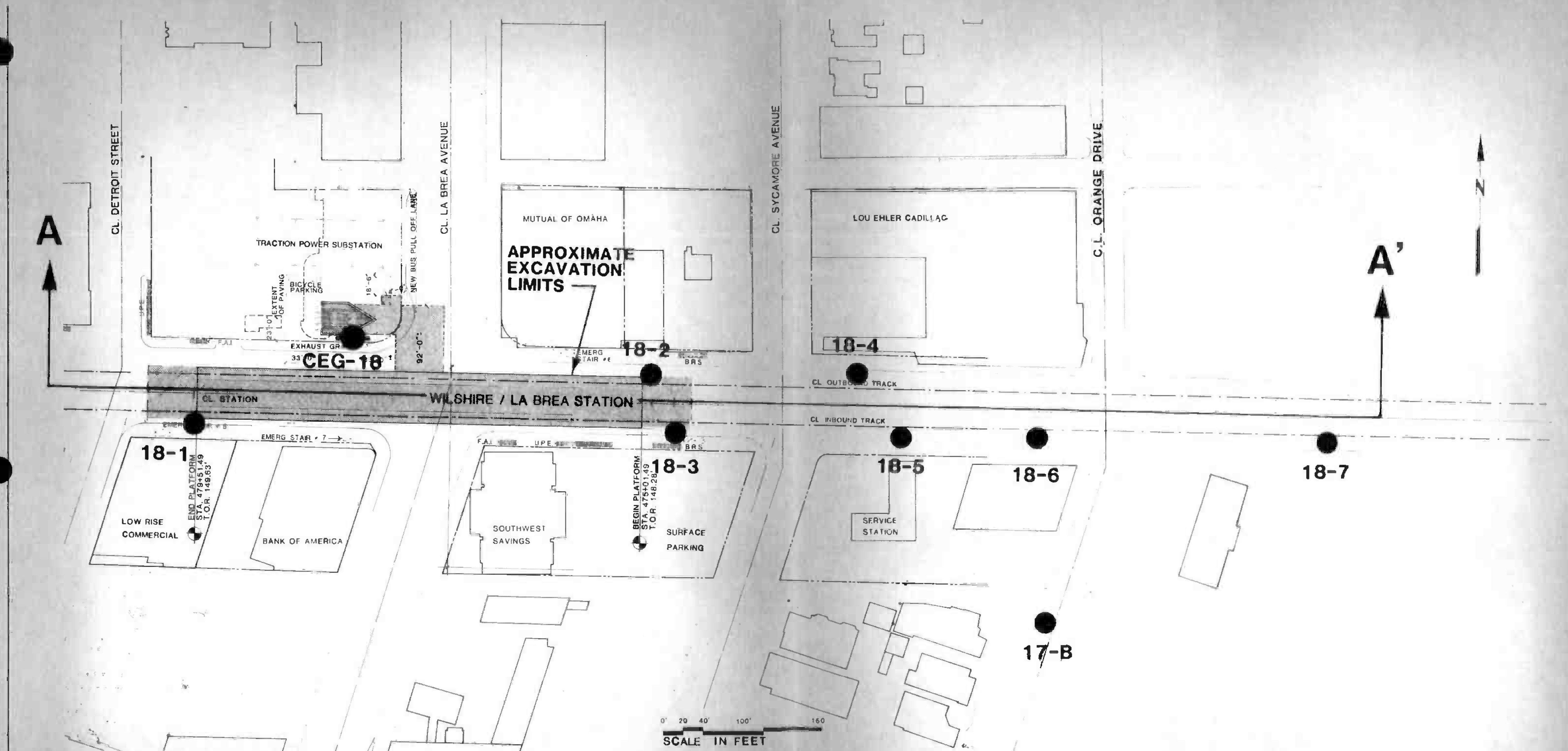
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted [signature] Date Mar 1984

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

DESIGN UNIT A245
LOCATION OF BORINGS AND GEOLOGIC SECTION

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



REFERENCES: "PRELIMINARY WILSHIRE/LA BREA STATION SITE PLAN" DRAWING NO. A-38 SHEET NO.9 OF 21 PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40', DATED JUNE 1983.

"CBD TO NORTH HOLLYWOOD LINE PLAN," WILSHIRE/LA BREA STATION STA. AR470+00 TO AR480+00 DRAWING NO. AP-16AAA-C-141 SHEET NO. 6 OF 21, PREPARED BY DMJM/PBQD, ORIGINAL SCALE 1" = 40', DATED JULY 1983.

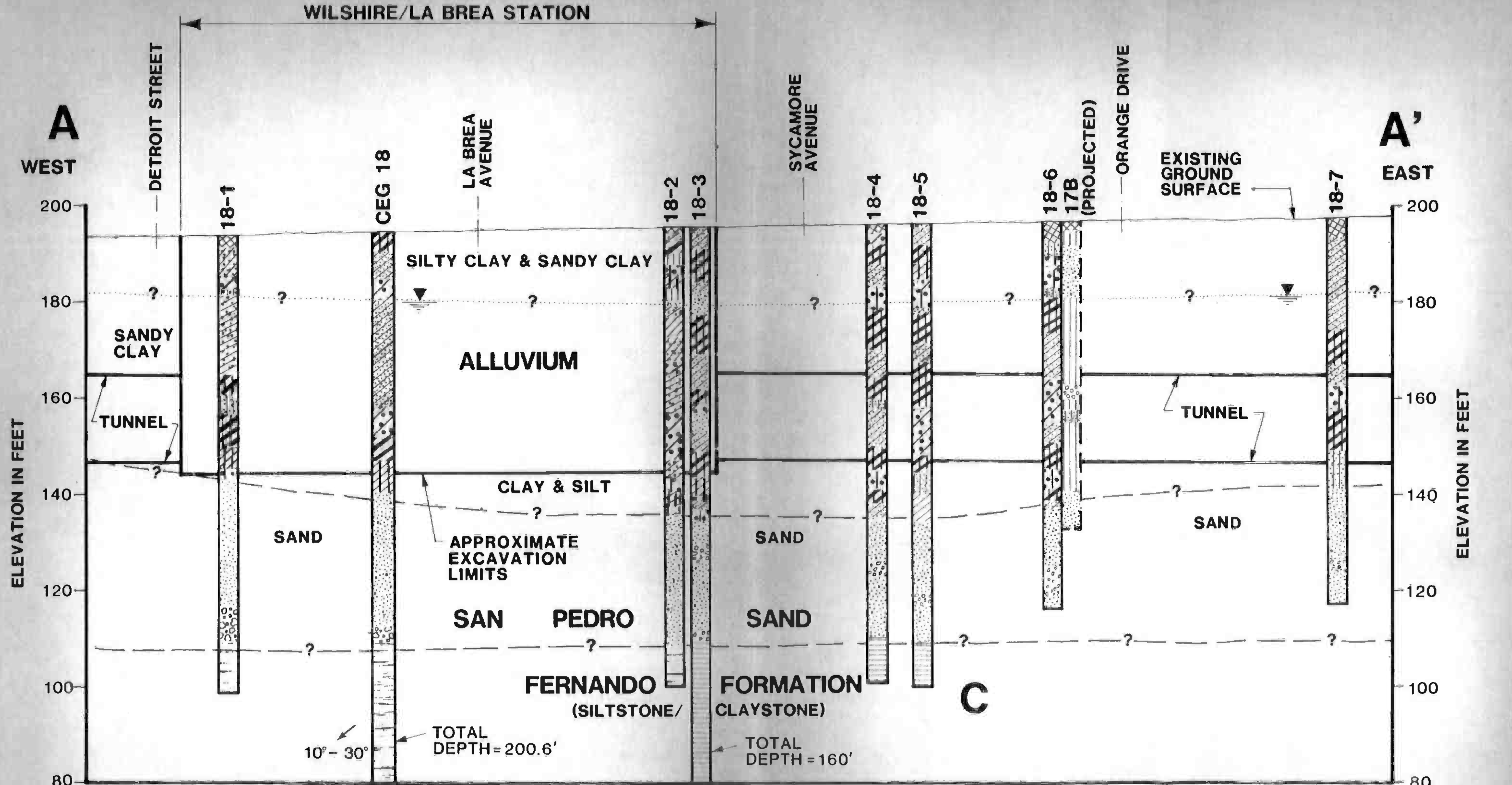
NOTES: 1.) FOR GEOLOGIC SECTION A-A' SEE DRAWING NO. 4
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

LOCATION OF BORINGS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	As Shown	Project No	83-1140
Date	Mar., 84	Drawing No	3
Prepared by	APT/CSJ		
Checked by	RG		
Approved By	JAD		



SCALE IN FEET

SCALE: HOR. 100 0 100 200

VER. 20 0 20 40

NOTES: 1.) FOR SECTION LOCATION SEE DRAWING NO. 3

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

SUBSURFACE SECTION A-A'

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	As Shown
Date	MAR., 84
Prepared by	CSJ
Checked by	RG
Approved By	JAD
Project No	83-1140
Drawing No	4

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

Period	Epoch	Unit	Description
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.
QUATERNARY	HOLOCENE	A_2	YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.
QUATERNARY	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.
QUATERNARY	PLEISTOCENE	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.
QUATERNARY	PLEISTOCENE	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)*

Period	Epoch	Unit	Description
TERTIARY	MIOCENE	3	Terzaghi Rock Condition Number
			Approximate boundary between Terzaghi numbers
TERTIARY	MIOCENE	2-5	TOPANGA FORMATION: Conglomerate, sandstone, and siltstone; thickly bedded; primarily hard and strong (Geologic symbol Tt).
TERTIARY	MIOCENE	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 A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	N/A
Date	Mar., 84
Prepared by	RG
Checked by	JAD
Approved By	HAS
Project No	83-1140
Drawing No	5

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A245 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, and 1983 borings drilled for this investigation. The specific boring logs included are numbered CEG-18, 18-1 through 18-7, and 17B.

Locations of the borings are shown on Drawing 2. Ground water observation wells (piezometers) were installed in borings listed in Section 5.4 (Table 5-1). Geophysical downhole and crosshole surveys were made for the 1981 investigation at Boring CEG-18 (see Appendix B).

The borings were drilled to depths generally ranging from 64 to 200 feet, and penetrated through the alluvium into the underlying San Pedro sand or bedrock. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was generally good in both the siltstone and claystone bedrock and the alluvium.

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

A.2 FIELD STAFF AND EQUIPMENT

A.2.1 Technical Staff

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

A.2.2 Drilling Contractor and Equipment

The roatry wash drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 1500 rotary wash rigs, each operated by a two-man crew. Man-sized auger borings were drilled with bucket auger equipment by A&W Drilling Company of Brea, California.

A.3 SAMPLING AND LOGGING PROCEDURES

Logging and sampling were performed in the field by the geologist. 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 450-pound slip-jar hammer. The Converse sampler was followed with the standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the Fernando Formation was encountered, the borings were sampled using a Pitcher Barrel and Converse ring sampler at 20-foot intervals.

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

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

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

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

A.3.2 Field Classification of Soils

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

Table A-1 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-1 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-2.

---

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

TABLE A-2 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.

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings 17B, CEG-18, 18-1, 18-3, and 18-7. Procedures for piezometer installation were as follows:

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

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 17B

Proj: DESIGN UNIT A-220 Date Drilled 10-26-83 Ground Elev. 198'

Drill Rig MAN-SIZE AUGER Logged By J. STELLAR Total Depth 64'

Hole Diameter 33" Hammer Weight & Fall N.A.

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		A/C PAVEMENT 0.0-0.6				Hole stands well 0'-48', continuous caving 48"-64'
	ML	FILL 0.6-2.0 CLAYEY SILT: dark brown, moist firm				
2	ML	ALLUVIUM 2.0-5.0 SILT: alternating light and dark brown, moist, stiff				
4						
	SP	5.0-16.0 SAND: light brown, moist, medium dense, with layers and streaks of silty and clayey sand and calcareous streaks and blebs				
6						
8						
10		becomes light green				
12						
14						
16	ML	16.0-34.0 SILT: light greenish brown, very moist, firm, with layers of clayey silt and numerous calcareous streaks and nodules				
18						bag sample at 18', water level at 18' after 2 hours
20		becomes wet				

Sheet 1 of 3

Project DESIGN UNIT A-220 Date Drilled 10-26-83 Hole No. 17B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	ML	16.0-34.0 SILT: (continued)				
22						water level at 23' after 1 hour
24						
26						
28		moderate H2O seepage				seepage 1± gpm
		gravelly layer, gravel to 1"				
30						
32						water level at 32' during drilling operation
34	ML/GM	34.0-37.0 GRAVELLY SILT: orange brown, wet, stiff, gravel to 1"				slow drilling @ 34'
36						
	ML	37.0-39.0 SILT: blue, wet, stiff, layers of sandy silt				
38						
	SM	39.0-41.0 SILTY SAND: orange brown, wet dense, with layers of sandy silt and sand				
40						
	ML	41.0-56.0 SILT: blue, wet, stiff, with layers of sandy silt and sand				
42						
44						

Project DESIGN UNIT A-220 Date Drilled 10-26-83 Hole No. 17B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	ML	41.0-56.0 SILT: (continued)				
46						bag sample at 46'
52		52'-54': sand layer, wet				
56	SP	SAN PEDRO FORMATION 56.0-64.0 SAND: blue, wet, dense, medium grained, slight sulfur odor				sand continuously caving, only small amounts of material remain in bucket bag sample @ 58' hole caved back to 48' after 2 hours
64		B.H. 64.0' Hole terminated due to running ground below 56'. No gas detected by meter. Downhole Observers: J. Stellar				

Depth scale: 44, 46, 48, 50, 52, 54, 56, 58, 60, 62, 64, 66, 68

Sheet 3 of 3

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 18

Proj: DESIGN UNIT A245 **Date Drilled** 1-19-27-81 **Ground Elev.** 194'
Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 200.6'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	3" CONCRETE FILL			AD	Auger to 3', set casing to 4', 1' stick-up
	CL	0.2-4.5 SILTY CLAY: olive black; mostly fines, trace of sand; medium stiff; moist				
2						
					RD	
4						
	CL	ALLUVIUM 4.5-11.0 SANDY CLAY: Moderate yellowish brown to pale yellowish brown; mostly clays and fine sand; very stiff; moist	J-1	5 6 9	SS	1.3/1.5 recovery
6					RD	
8		grading coarser with depth				
10			J-2	5 10 12	SS	1.2/1.5 recovery
	SC	11.0-15.0 CLAYEY SAND: pale yellowish brown; mostly fine to medium angular sand with occasional gravel and little fines; medium dense; moist			RD	
12						
14		gravelly lens				
	CL	15.0-37.5 SANDY CLAY: pale yellowish green mottled with light greenish grey; mostly clay and fine to medium sand; stiff; moist				
16			J-3	4 6 8	SS	1.5/1.5 recovery
					RD	
18						
			C-1	8 11	DR	
20						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	15.0-37.5 SANDY CLAY: continued	J-4	9 12 15	SS	1.5/1.5 recovery
22					RD	
24		color change to dusky yellow mottled with pale greenish yellow				
26			J-5	5 10 17	SS	1.5/1.5 recovery
28					RD	
30		sandy clay lens; medium bluish grey	J-6	10 17 17	SS	1.5/1.5 recovery
32					RD	
34		medium sand lens grading sandier with depth				
36			J-7	11 11 16	SS	1.5/1.5 recovery
					RD	
38	SC	37.5-44.0 CLAYEY SAND: dusky yellow; mostly fine to medium subangular sand and clay, interbedded with sandy clay; dense to very dense; moist				
40			C-2	16 18	DR	1.0/1.0 recovery
			J-8	17 24 30	SS	1.5/1.5 recovery
42					RD	
44		becoming more clayey				

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	44.0-48.5 SILTY CLAY: dark greenish grey; mostly fines, trace of sand; occasional concretions; very stiff; moist			RD	
			J-9	8	SS	1.5/1.5 recovery
46				18		
				20		1/19/81
					RD	1/20/81 water at 15' in am
		grading coarer				
48						
	ML	48.5-55.4 CLAYEY SILT: dark greenish grey; very stiff; moist				
50			J-10	18	SS	1.5/1.5 recovery
				20		
				20		
					RD	
52						
54						
			Box 1		PB	begin continuous pitcher samples 2.2/2.5 recovery
56	SP	55.4-84.8 SAND: greyish green; mostly fines; granular to subangular sand, trace silt; moist to wet; dense; sulfur odor; occasional gravel				
58						2.0/2.5 recovery
60						pocket penetrometer 0.5 2/9/81
					RD	
62						
64			C-3	32	DR	
				44		1.0/1.0 recovery
			J-11	12	SS	15/.5, 50/.4
				50		
66					RD	
68						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	55.4-84.8 SAND: continued			RD	
		gravel and coarse sand lens				
70			J-12	28 33 45	SS	1.3/1.5 recovery
					RD	
72		little subrounded medium gravel				
74						checked gas: 21% O_2 0% combustibles
			S-1		PB	1.5/2.4 recovery
76		silty claystone				
						chatter
			Box 1 (cont.)			
78						1.7/2.6 recovery
80		coarse sand lens				
						1.4/2.5 recovery
82		shells and angular to round sand and gravel				intense rig chatter
84						0/2.5 recovery
		FERNANDO FORMATION 84.8-200.6 SILTY CLAYSTONE: olive grey; moist; interbedded zones of banded colors, little compositional change, dips 10-30° Physical Condition: moderately fractured to massive; friable to weak strength; little weathered				pocket penetrometer 1.0 (broke apart) 2/9/81 0.3/2.5 recovery
86						
88						0/2.5 recovery
90					RD	added polydrill
			Box 1 (cont.)		PB	
92						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		84.0-200.6 <u>SILTY CLAYSTONE</u>:	Box 1 (cont.)		PB	1.8/2.5 recovery samples disturbed
94						1.5/2.5 recovery
96					RD	sample disturbed drilled out to try to recover rock
98			Box 2		PB	
100						2.5/2.5 recovery pocket penetrometer 3.0 2/9/81 picked up rock in tube
102			S-2			2.1/2.5 recovery
		silt lens, dry				
104		interbedded lenses of silty claystone and clayey siltstone	Box 2 (cont)			
						1.8/2.5 recovery
106						chatter
		thin cemented lens				1.3/2.8 recovery
108						
110						1.5/2.8 recovery
112						1.2/2.8 recovery
114						pocket penetrometer >4.5 2/9/81
			S-3			
116						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		84.8-200.6 SILTY CLAYSTONE: cont. Physical Condition: moderately fractured to massive; friable to low hardness; friable to weak strength; little weathered			PB	1.8/2.8 recovery
			Box 2 (cont)			
118						chatter 2.4/2.8 recovery
120			Box 3			
						1.9/2.8 recovery
122						
124						1.5/2.8 recovery
126						1.4/2.8 recovery
128						
130			S-4			2.8/2.8 recovery pocket penetrometer >4.5 2/9/81
			Box 3 (cont)			
132						2.3/2.3 recovery
134						1.2/2.8 recovery
136						1/20/81
						1/21/81 water at 15' in am
138		bedding dips 20° from horizontal				2.8/2.8 recovery
140						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		84.8-200.6 SILTY CLAYSTONE: continued Physical Condition: moderately fractured to massive; friable to low hardness; friable to weak strength; little weathered	Box 3 (cont)		PB	1.5/2.8 recovery
142		142.0-150.0 interbedded silty claystone with siltstone and fine sandstone; 30° dip of bedding; very thinly bedded; olive grey w/ dusky yellow green interbeds; 144.0', very thin layer of white ash with 10° dip	Box 4			
144						2.8/2.8 recovery pocket penetrometer >4.5 2/9/81
146			S-5			2.1/2.8 recovery
148			Box 4 (cont)			
150						2.8/2.8 recovery
152						2.3/2.8 recovery
154		minor cross bedding present				1.7/2.8 recovery
156						
158						
160		160', light bluish grey, thin, fine sandstone lens	Box 5			pocket penetrometer >4.5 2/9/81 1.5/2.8 recovery
162			S-6			2.0/2.8 recovery
164						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164		84.8-200.6 SILTY CLAYSTONE: continued Physical Condition: moderately fractured to massive; friable to low hardness; friable to weak strength; little weathered	S-6 Box 5		PB	2.2/2.8 recovery
166		166.0 thin silty fine sandstone lens, light bluish grey				gas: 0% combustibles 21% O_2
168						pocket penetrometer >4.5 1.4/2.8 recovery
170		170.8 very thin silty fine sandstone lens 171.2 very thin silty fine sandstone				1.3/2.8 recovery
172						
174		174.4 very thin fine sandstone lens bedding dip change to 10°, most fractures along sandstone and siltstone.				pocket penetrometer >4.5 2/9/81 2.4/2.8 recovery
176		177.5 thin claystone lens, soft				1.7/2.2 recovery
178		178.0-179.2 well cemented siltstone lens, closely fractured				intense chatter 0.2/2.8 recovery drilling smoothed out
180						
182			S-7			2.3/2.8 recovery
184			Box 6			2.2/2.8 recovery
186		187.0 very thin fine sandstone lens				1.9/2.8 recovery
188						

Project DESIGN UNIT A-245 Date Drilled 1-19-27-81 Hole No. 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		84.8-200.6 SILTY CLAYSTONE: continued Physical Condition: moderately fractured to massive; friable to low hardness; friable to weak strength; little weathered 190.0 very thin sandstone lens	Box 6 (cont)		PB	1.1/2.8 recovery pocket penetrometer >4.5
190						
192						1.6/2.8 recovery
194						
196						1.0/2.8 recovery
198						disturbed sample 0/2.8 recovery
200						
202		B.H. 200.6 - Terminated hole at 2:30 1/21/81, E-logged 1/21/81, down hole geophysics 1/21/81, water level noted on Sheet 1 following stabilization for 4 days prior to pressure test. Water pressure test attemped 1/26/81 problems with minor pack leakage and problems seating lower packer. Water loss was probably in fractured cemented zone at 178'. Hole reamed 1/27/81 to 6", 4" casing installed to 100'.				
204						
206						
208						
210						
212						

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

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

BORING LOG 18-1

Proj: DESIGN UNIT A-245 **Date Drilled** 10-8-9-83 **Ground Elev.** 192.5'
Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 94.7'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.7 CONCRETE			GB	start drilling 10:00
	GP	0.7-2.2 Base Rock - sandy gravel			AD	water immediately below concrete
2	CL	FILL 2.2-8.8 SANDY CLAY: brownish black; mostly fines with trace of fine sand; stiff; moist to wet; petroleum odor		7	DR	0.9/1.0
			C-1	10		
4					RD	set tub and cased to 4.5'
		little sand	J-1	6 8 11	SS	1.3/1.5
6					RD	
8	CH	increase to some sand; becoming stiff	C-2	4 7	DR	1.0/1.0
	SC	8.8-11.0 CLAYEY SAND: greenish black; mostly fine sand, with some fines; medium dense; wet; strong petroleum odor			RD	
10			J-2	5 5 6	SS	1.3/1.5
	ML	11.0-12.5 SANDY SILT: greenish black; mostly fines and fine sand; stiff wet; strong petroleum odor			RD	
12	CL	OLD ALLUVIUM 12.5-17.0 SANDY CLAY: greyish green; mostly fines, with little sand; stiff; wet; weak petroleum odor; contains cemented nodules	C-3	3 4	DR	pocket pen 1.5 1.0/1.0
14					RD	add casing to 13.5' losing circulation
				3 5 7	SS	no recovery
16					RD	
	CL SC	17.0-28.2 SANDY CLAY: yellowish olive grey; mostly fines with little fine to medium sand; very stiff; moist; contains cemented nodules; ferrous staining; & clayey sand	C-4	5 15	DR	1.0/1.0 pocket pen 2.75
18					RD	
			J-3	6 9	SS	1.3/1.5
20						

Project DESIGN UNIT A-245 **Date Drilled** 10-8-9-83 **Hole No.** 18-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	17.0-28.2 SANDY CLAY/CLAYEY SAND: cont.		11	SS	
					RD	
22		some sand with some clayey sand lenses, ferrous staining, decrease in cemented nodules		14	DR	0.9/1.0
	SC/ CL		C-5	24		
					RD	
24			J-4	5 8 9	SS	1.4/1.5
					RD	
26						
				36	DR	1.0/1.0
		heavily ferrous stained thin gravel lens	C-6	48		
28					RD	
	CL	28.2-32.8 SILTY CLAY: dark greenish grey; hard; moist; contains cemented nodules	J-5	7 19 20	SS	1.5/1.5
30						
					RD	
		silt becoming clayey				
32				33	DR	1.0/1.0
			C-7	46		
	ML	32.8-38.8 SANDY SILT: dark greenish grey; mostly fines with some fine sand; hard; moist; contains few cemented nodules; strong sulfur odor			RD	
34			J-6	6 19 30	SS	1.0/1.5
36					RD	
		thin silty sand lens		27	DR	1.0/1.0
			C-8	41		
38	SM	38.8-42.0 SILTY SAND: dark greenish grey; moist; weakly cemented; dense; strong sulfur odor			RD	
			J-7	6 18 25	SS	1.5/1.5
40						
		sand content decreases			RD	
42	CL	42.0-44.0 SILTY CLAY: moist; stiff		12	DR	1.0/1.0
			C-9	24		
44					RD	

Project DESIGN UNIT A-245 Date Drilled 10-8-9-83 Hole No. 18-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	44.0-47.0 CLAYEY SILT: dark greenish grey; very stiff; moist	J-8	7 13 18	SS	1.1/1.5
					RD	
46						
	SM	47.0-51.0 SILTY SAND: dark greenish grey; mostly fine sand, with little fines; dense; moist; contains silt lenses	C-10	17 50-5"	DR	0.9/0.9
48					RD	
			J-9	5 35 50-4.5"	SS	1.4/1.4
50					RD	
	SP	SAN PEDRO FORMATION 51.0-77.0 SAND: dark greenish grey; mostly fine sand, trace of silt; very dense; wet				
52			C-11	45 50-3"	DR	0.6/0.7
					RD	
54			J-10	32 50-5"	SS	0.7/0.9
					RD	
56						
				53 50-3"	DR	no recovery
58					RD	
			J-11	26 52	SS	0.7/1.0
60					RD	
62			C-12	37 50-4.5"	DR	0.7/0.9
					RD	
		— and silty sand				
64			J-12	31 39 46	SS	1.2/1.5
					RD	
66						
			C-13	35 50-4.5"	DR	0.7/0.9
68						

Project: DESIGN UNIT A-245 **Date Drilled:** 10-8-9-83 **Hole No.:** 18-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	51.0-77.0 SAND: continued			RD	
			J-13	23 50-	SS 5.5"	0.6/1.0
70					RD	
72		occastional gravelly lenses	C-14	68 50-3	DR "	0.7/0.7
					RD	
74			J-14	36 50	SS	0.7/1.0
					RD	
76						
	SM	77.0-84.0 GRAVELLY/SILTY SAND: grey and dark greenish grey; interbedded; very dense; wet; strong sulfur odor; increased gravel	C-15	86 50-2	DR "	0.6/0.9 6:00 10/8/83 7:00 am
78					RD	
			J-15	53	SS	0.4/0.5
					RD	
80		numerous shells				rig chatter
82			C-16	54 60-3	DR 5"	0.8/0.8
					RD	
84		FERNANDO FORMATION 84.0-94.7 CLAYSTONE: olive grey and dark greenish grey; irregular color variations; not bedded; sulfur odor; no cementation	C-17	25 50-5	DR "	0.8/0.9
					RD	
86						
88		Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh	C-18	48 50-4	DR "	0.8/0.8
90					RD	
92						

Sheet 4 of 5

Project DESIGN UNIT A-245 Date Drilled 10-8-9-83 Hole No. 18-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		84.0-94.7 CLAYSTONE: continued			RD	
94			C-19	47 50-3"	DR	0.7/0.7
96		B.H. 94.7 Terminated hole, installed piezometer to bottom, 75'-95' slotted, pea gravel backfill to surface				complete drilling and flusing 9:15 am
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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 18-2

Proj: DESIGN UNIT A-245 Date Drilled 10-4-5-83 Ground Elev. 195.5'
Drill Rig Failing 1500 Logged By L. Schoeberlein Total Depth 94.7'
Hole Diameter 4 7/8" Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.7 CONCRETE			GB	start drilling 3:30
	CL	FILL 0.7-2.8 SANDY CLAY: olive black; mostly clay with a trace of fine sand; very stiff; moist	C-1	6 8	DR	1.0/1.0
	CL	OLD ALLUVIUM 2.8-5.0 SILTY CLAY: moderate brown; stiff; moist			AD	
	ML	5.0-8.0 SILT: dusky yellow; hard; moist	J-1	14 26 21	SS AD	0.8/1.5
8	SM/ML	8.0-10.0 SANDY SILT: dusky yellow; mostly fine sand with some fines; dense; moist	C-2	17 27	DR AD	1.0/1.0
10	ML	10.0-12.5 CLAYEY SILT: dusky yellow; hard; moist	J-2	12 19 32	SS RD	1.5/1.5 set tub & case to 13' 4:30 10/4/83 7:00 10/3/83
	ML	12.5-14.5 SILT: dusky yellow; hard; moist; with cemented nodules	C-3	7 22	DR RD	1.0/1.0
	SP	14.5-16.0 SAND: brown; mostly fine sand, trace of silt; dense; moist	J-3	1 23 25	SS RD	1.5/1.5
16	CL	16.0-17.5 SANDY CLAY: dusky yellow; mostly fines with a trace of sand and gravel; hard; moist				
	CL	17.5-22.5 CLAY: dusky yellow; hard; moist; with cemented zones & nodules	C-4	12 22	DR RD	1.0/1.0 pocket pen> 4.5
20						

Project DESIGN UNIT A-245 **Date Drilled** 10-4-5-83 **Hole No.** 18-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	17.5-22.5 CLAY: continued becoming weakly cemented	J-4	12 22 30	SS	1.5/1.5
					RD	
22						
	CL	22.5-27.5 SANDY CLAY: dusky yellow with whitish nodules; mostly fines with trace of fine sand; hard; moist	C-5	11 15	DR	1.0/1.0
24					RD	
			J-5	21 22 26	SS	1.5/1.5
26					RD	
28	SC	27.5-29.5 CLAYEY SAND: moderately yellowish brown; mostly fine sand with some fines; dense moist	C-6	12 17	DR	1.0/1.0
					RD	
30	CL	29.5-38.5 SANDY CLAY: dusky yellow; mostly fines with trace of fine sand; very stiff; moist	J-6	5 11 14	SS	1.5/1.5
32					RD	
34	SC	some ferrous staining, becoming hard, and silty sand	C-7	17 37	DR	1.0/1.0
					RD	
			J-7	11 25 39	SS	1.5/1.5
36					RD	
38			C-8	27 53	DR	0.9/1.0
	SC	38.5-48.0 CLAYEY SAND: dusky yellow; mostly fine sand with some fines; very dense; moist; contains thin sand and sandy clay lenses			RD	
40			J-8	9 14 41	SS	1.5/1.5
42					RD	
44	CL	and sandy clay	C-9	48 55	DR	1.0/1.0

Project DESIGN UNIT A-245 Date Drilled 10-4-5-83 Hole No. 18-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	38.5-48.0 CLAYEY SAND: continued			RD	
		dense	J-9	10 21 22	SS	1.5/1.5
46					RD	
48	CL	48.0-51.0 SANDY CLAY: dark greenish grey; mostly fines with a trace of fine sand; hard; moist; weakly cemented in places; occasional very thin clayey silt lenses	C-10	33 41	DR	1.0/1.0
					RD	
50			J-10	7 11 22	SS	1.5/1.5
	SM	51.0-54.5 SILTY SAND: dark greenish grey; mostly fine sand with little fines; dense; moist to wet			RD	
52			C-11	14 34	DR	1.0/1.0
54	ML				RD	
	CL	54.5-57.5 SILTY CLAY: dark greenish grey mottled with browns; mostly fines, trace of fine sand; hard; moist	J-11	9 18 27	SS	1.5/1.5
56					RD	
58	SM	57.5-59.0 SILTY SAND: dark greenish grey; mostly fine sand with little fines; very dense; moist to wet	C-12	27 50-5	DR "	0.9/0.9
	SP	SAN PEDRO FORMATION 59.0-85.5 SAND: dark greenish grey; mostly fine sand, rounded; trace of silt; very dense; wet; sulfur odor			RD	
60				23 51	SS	no recovery
					RD	
62			C-13	65 50-2	DR "	0.7/0.7
64					RD	
			J-12	30 50-	SS 5"	0.7/0.9
66					RD	
68						

Sheet 3 of 5

Project DESIGN UNIT A-245 Date Drilled 10-4-5-83 Hole No. 18-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	59.0-85.5 SAND: continued	C-14	87	DR	0.6/0.7
				60-2	"	
					RD	
70			J-13	40	SS	0.5/0.9
				50-4	.5"	
					RD	
	SW	71.5 GRAVELLY SAND lenses with a little gravel				rig chatter
72						
			C-15	87	DR	disturbed
					RD	
74						
			J-14	40	SS	0.4/0.7
				50-2	.5"	
76					RD	
78			C-16	90	DR	0.3/0.5
		with trace of gravel			RD	disturbed
80			J-15	37	SS	0.7/0.9
				50-5	.5"	
					RD	
82						
			C-17	83	DR	0.7/0.7
				50-3	"	
84					RD	
			J-16	26	SS	1.5/1.5
				14		
86		FERNANDO FORMATION 85.5-94.7 CLAYSTONE: olive grey; massive bedding; contains mica; slight petroleum odor.		27		
					RD	
88		Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh;@88',6" well cemented hard zone				
			C-18	33	DR	0.8/0.8
				50-4	"	
90					RD	
92						

Project DESIGN UNIT A-245 Date Drilled 10-4-5-83 Hole No. 18-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		85.5-94.7 CLAYSTONE: continued			RD	
94			C-19	48 50-2.5"	DR	0.7/0.7
		B.H. 94.7 Terminated hole. Tremied grout to surface.				completed drilling 2:45
96						
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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 18-3

Proj: DESIGN UNIT A-245 **Date Drilled** 10-5-6-83 **Ground Elev.** 195.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 160.8'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.2 ASPHALT			GB	Start drilling 5:15
	CONC	0.2-1.0 CONCRETE				
	CL	OLD ALLUVIUM 1.0-4.6 SANDY CLAY: moderate brown; mostly fines with a little fine sand and gravel; firm to stiff; moist to wet color change @ 2' to moderate brown			AD	
2				4	DR	0.5/1.0
			C-1	3		
					AD	
4				7	SS	1.0/1.5
	CL	4.6-11.5 SILTY CLAY: moderate yellowish brown; hard; moist; contains numerous cemented nodules; weakly cemented throughout	J-1	19		set tub & case to 4.5
				33		6:00 10/5/83
6					RD	7:00 10/6/83
	ML/MH			8	DR	1.0/1.0
8			C-2	19		
					RD	
		trace of sand		12	SS	1.2/1.5
10			J-2	20		
				72		
					RD	
12	CL	11.5-13.5 SANDY CLAY: moderate yellowish brown; mostly fines with a little fine to medium sand; very stiff; moist		12	DR	0.9/1.0
			C-3	23		
					RD	
14	SP	13.5-18.5 SAND: brown; mostly fine sand, trace of silt; medium dense; moist		7	SS	1.5/1.5
			J-3	14		
				16		
					RD	
16		sandy clay lens				
				18	DR	0.8/1.0
18			C-4	36		
					RD	
	CL	18.5-26.5 SILTY CLAY: yellowish grey		9	SS	1.2/1.5
20			J-4	14		

Project DESIGN UNIT A-245 Date Drilled 10-5-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	18.5-26.5 SILTY CLAY: cont.		17	SS	
		very stiff; moist; contains some cemented nodules with thin sandy clay lenses			RD	
22		some ferrous staining, nodules have Mn staining on fracture surface		23	DR	1.0/1.0
			C-5	33		
	CH				DR	
24			J-5	7	SS	1.5/1.5
				11		
		occasional sand		17		
26					RD	
	SC	26.5-33.5 CLAYEY SAND: moderate yellowish brown; medium dense; moist; cemented nodules		13	DR	1.0/1.0
			C-6	32		
28					RD	
			J-6	4	SS	1.5/1.5
30				9		
				12		
					RD	
32		well cemented zone, caliche	C-7	50-3"	DR	0.2/0.2
					RD	not in rings
34	CL	33.5-36.5 SILTY CLAY: yellowish grey; hard moist; contains cemented nodules and clayey sand lenses	J-7	13	SS	1.5/1.5
				27		
				28		
36					RD	
	SC	36.5-38.8 CLAYEY SAND: yellowish grey; mostly fine sand with a little fines; very dense; wet; contains cemented nodules	C-8	27	DR	0.7/0.8
				50-4"		
38					RD	
	SP	38.5-42.5 SAND: yellowish grey; mostly fine to medium sand with a trace of silt; very dense; wet		13		no recovery
40				25		
				45		
					RD	
42				54	DR	1.0/1.0
	CL	42.5-46.5 SANDY CLAY: yellowish grey; with a little fine sand; hard; moist	C-9	39		
44					RD	

Project DESIGN UNIT A-245 Date Drilled 10-5-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	42.5-46.5 SANDY CLAY: cont.	J-8	10 22 26	SS	1.0/1.5
46					RD	
	CL	46.5-51.5 SILTY CLAY: dark bluish grey; hard; moist; contains brownish cemented zones at top; small cemented nodules throughout	C-10	76 54	DR	1.0/1.0
48					RD	
			J-9	7 19 28	SS	1.5/1.5
50					RD	
52	ML	51.5-56.0 CLAYEY SILT: dark bluish grey; hard; moist; occasional cemented nodules	C-11	29 50-5"	DR	1.0/1.0
					RD	
54			J-10	5 14 24	SS	1.5/1.5
					RD	
56	CL	56.0-57.5 SILTY CLAY: dark greenish grey				
			C-12	14 29	DR	1.0/1.0
58	CL	57.5-58.5 SANDY CLAY: dark greenish grey; mostly fines with little fine sand; hard; moist			RD	
	SM	58.5-61.0 SILTY SAND: dark greenish grey; mostly fine sand with a little fines; very dense; moist to wet	J-11	14 33 50	SS	1.0/1.5
60					RD	
	SP	SAN PEDRO FORMATION 61.0-86.0 SAND: dark greenish grey; mostly fine sand with a trace of silt; very dense; wet				
62			C-13	44 50-4.5"	DR	0.8/0.9
					RD	
64				30 50-4.5"	SS	no recovery
					RD	
66						
			C-14	113	DR	0.5/0.5
68					RD	

Sheet 3 of 7

Project DESIGN UNIT A-245 Date Drilled 10-5-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	61.0-86.0 SAND: cont.			RD	
		occasional gravel	J-12	46 50-5	SS "	0.5/0.9
70					RD	
72			C-15	106	DR	0.3/0.5
					RD	partial
74		several very thin clayey lenses	J-13	41 50-	SS 5"	0.6/0.9
					RD	
76						
			C-16	51 60-4.	DR 5"	0.7/0.9
78					RD	
			J-14	33 53	SS	0.5/1.D
80					RD	
82		with little silt	C-17	59 60-	DR 3"	0.7/0.7
					RD	
84			J-15	44 62	SS	0.5/1.D
					RD	rig chatter
		basal gravel				
86		FERNANDO FORMATION 86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; with fine sand partings; contains mica thinly bedded to massive bedding; sulfur odor				
			C-18	52 52-	DR 3"	0.7/0.7
88					RD	
		Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh				
90						
			C-19	50 50-	DR 3.5"	0.5-0.8
92						

Project DESIGN UNIT A-245 Date Drilled 10-5-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
		contains irregular angular inclusions of different color siltstone	C-20	63	DR	0.7/0.7
				63-3	"	
96					RD	
			C-21	48	DR	0.7/0.7
116				65-	3" RD	

Depth scale: 92, 94, 96, 98, 100, 102, 104, 106, 108, 110, 112, 114, 116

Project DESIGN UNIT A-245 Date Drilled 10-5-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
135			C-22	56	DR	0.7/0.7
				50-3	"	
136					RD	
140						

Sheet 6 of 7

Project DESIGN UNIT A-245 Date Drilled 10-6-83 Hole No. 18-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
160			C-23	34 60-4	DR "	0.8/0.8
		B.H. 160.8 Terminated hole at extended depth to get groundwater data within bedrock. Installed piezometer to bottom, slotted interval 140-160' backfilled w/pea gravel to 120', tremied grout seal 120' - 70', some cave overnight and backfilled top w/ pea gravel				Completed drilling & flushing hole 7:15

Depth scale: 140, 142, 144, 146, 148, 150, 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 18-4

Proj: DESIGN UNIT A-245 | Date Drilled 10-10-83 | Ground Elev. 196.5'

Drill Rig Failing 1500 | Logged By L. Schoeberlein | Total Depth 94.8'

Hole Diameter 4 7/8" | Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.7 CONCRETE			GB	start drilling 7:30 groundwater immediately below concrete
	CL	FILL 0.7-1.5 SANDY CLAY: brownish black; stiff			AD	
2	CL	OLD ALLUVIUM 1.5-3.5 SANDY CLAY: moderate brown; mostly fine to medium sand with some fines; dense; moist	C-1	4 19	DR	0.8/1.0
					AD	
4	CL	3.5-8.5 SILTY CLAY: moderate brown; hard; moist; contains cemented nodules; weakly cemented throughout	J-1	7 15 20	SS	1.5/1.5
6					RD	set tub & cased to ~5'
			C-2	9 19	DR	1.0/1.0 pocket pen > 4.5
8					RD	
	CL	8.5-13.2 SANDY CLAY: dark yellowish brown; mostly fines with a little fine sand; hard; moist; occasional cemented nodules	J-2	19 35 51	SS	1.3/1.5
10					RD	
12			C-3	14 20	DR	1.0/1.0 pocket pen > 4.5
					RD	
14	SM/SP	13.2-17.8 SILTY SAND: dark yellowish brown; mostly fine sand with a little fines; medium dense; moist; occasional clayey silt lenses	J-3	5 11 18	SS	1.2/1.5
16					RD	
		wet	C-4	12 15	DR	1.0/1.0 disturbed
18	CL	17.8-26.5 SILTY CLAY: yellowish grey; hard moist; contains cemented zones and nodules			RD	
			J-4	5 15	SS	1.5/1.5
20						

Project DESIGN UNIT A-245 Date Drilled 10-10-83 Hole No. 18-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	17.8-26.5 <u>SILTY CLAY</u>: cont.		17	SS	
					RD	
22		increased cementation		8	DR	1.0/1.0
			C-5	29		
					RD	
24			J-5	8	SS	1.5/1.5
				24		
				27		
26					RD	
	CL	26.5-32.0 <u>SANDY CLAY</u>: moderate yellowish brown; mostly fines with a little fine sand; stiff; moist; contains occasional cemented nodules; sand content increases with depth; ferrous staining		18	DR	1.0/1.0
			C-6	26		pocket pen 1.75
28					RD	
			J-6	3	SS	1.5/1.5
				9		
30				12		
					RD	begin circulation loss
32	CL	32.0-36.5 <u>SILTY CLAY</u>: moderate yellowish brown; hard; moist; occasional cemented nodules; some ferrous staining		10	DR	1.0/1.0
			C-7	28		pocket pen 4.25
					RD	
34						10:30
			J-7	38-5.5"	SS	hammer broke down
					RD	1.5 hrs; 0.5/0.5 mixed 1 sack mud
36						
	ML	36.5-38.5 <u>SANDY SILT</u>: moderate yellowish brown; mostly fines with some fine sand; hard; moist; contains lenses of silty sand and silty clay		49	DR	1.0/1.0
38			C-8	50		
					RD	
	CL	38.5-46.0 <u>SANDY CLAY</u>: moderate yellowish brown; mostly fines with some fine sand; hard; moist	J-8	7	SS	1.5/1.5
				17		
40				21		
					RD	
42		coarse sand		26	DR	1.0/1.0
			C-9	42		
44					RD	

Project DESIGN UNIT A-245 Date Drilled 10-10-83 Hole No. 18-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	38.5-46.0 SANDY CLAY: cont. clayey sand, silty sand and sandy silt lenses	J-9	7 30 50-4"	SS	1.2/1.2
					RD	
46	CL	46.0-51.5 SILTY CLAY: dark greenish grey; sandy clay lens at top; hard; moist; minor cementation in places				
			C-10	19 50-4.5"	DR	0.6/0.8 pocket pen 4.5
48					RD	
			J-10	9 16 23	SS	1.5/1.5
50					RD	
52	ML	51.5-55.0 SANDY SILT: dark greenish grey; fine sand; very stiff; moist; contains occasional cemented nodules	C-11	38 50-5"	DR	1.0/1.0
					RD	
54			J-11	7 18 33	SS	1.5/1.5
	CL	55.0-57.8 SILTY CLAY: dark greenish grey; weakly cemented; hard; moist; contains occasional cemented nodules			RD	
56						
			C-12	21 28	DR	pocket pen > 4.5
58	CL	57.8-61.0 SANDY CLAY: dark greenish grey; mostly fines with some fine sand; hard; moist; grades to sandy silt and silty sand			RD	
				16 40 46	SS	no recovery
60					RD	
	SP	SAN PEDRO FORMATION 61.0-86.0 SAND: dark greenish grey; mostly fine sand with trace of silt; very dense; wet				
62				47 50-3.5"	DR	sample fell out
					RD	jetting out
64						
			J-12	24 50-5"	SS	0.8/0.4
					RD	
66		sulfur odor				
			J-13	75 60-2.5"	DR RD	sample disturbed
68						

Project DESIGN UNIT A-245 Date Drilled 10-10-83 Hole No. 18-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	61.0-86.0 <u>SAND</u>:			RD	
			J-17	27 50-5	SS "	0.7/0.9
70					RD	
72	(SW)	mostly gravel	C-13	98	DR	0.5/0.5
					RD	disturbed continuing circulation loss , mixed mud
74		fine to medium sand	J-15	25 50	SS	0.4/1.0
					RD	
76						
		fine sand	C-14	47 50-4	DR .5"	5" sample recovered 0.4/0.8
78					RD	
		fine sand	J-16	53 50-5	SS "	0.6/0.9
80					RD	rig chatter
82			C-15	40 50-	DR 5"	0.9/0.9
					RD	
84			C-16	19 53-4	SS .5"	0.7/0.9
					RD	
86		FERNANDO FORMATION 86.0-94.8 <u>INTERBEDDED CLAYSTONE and SILTSTONE</u>: olive grey and dark greenish grey; thinly bedded to massive; contains mica <u>Physical Condition</u>: little fractured to massive; friable hardness and strength; little weathered to fresh				
88			C-17	37 50-5	DR "	0.9/0.9
					RD	
90						
92						

Project DESIGN UNIT A-245 Date Drilled 10-10-83 Hole No. 18-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		86.0-94.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94				41	DR	0.8/0.8
			C-18	50-3.5"		
		B.H. 94.8 Terminated hole; tremied grout to surface				Drilling complete 5:15
96						
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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 18-5

Proj: DESIGN UNIT A-245 **Date Drilled** 10-7-8-83 **Ground Elev.** 197'
Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 95.7
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.6 Concrete			GB	start drilling 8:30
	CL	OLD ALLUVIUM 0.6-3.5 SANDY CLAY: moderate brown; mostly fines with some fine sand; stiff moist			AD	
2			C-1	4 4	DR	0.8/1.0
					AD	
4	CL	3.5-8.5 SILTY CLAY: moderate brown; very stiff; moist; gasoline odor	J-1	4 14 18	SS	1.3/1.5
6					RD	set tub & cased to 4.5' mixed mud
	SC	mottled and layered with greyish green, and clayey sand	C-2	8 22	DR	1.0/1.0
8					RD	
	ML	8.5-11.5 CLAYEY SILT: greyish green; hard; moist	J-2	7 17 18	SS	1.0/1.5
10					RD	
12	SC	11.5-13.5 CLAYEY SAND: light olive grey; mostly fines with a little fine sand; very dense; moist; contains some cemented nodules	C-3	14 28	DR	1.0/1.0
					RD	
14	SM	13.5-17.5 SILTY SAND: light olive brown; mostly fine sand with a little fines; very dense; moist to wet	J-3	9 22 31	SS	1.0/1.5
16					RD	
			C-4	11 21	DR	1.0/1.0
18	CL	17.5-18.5 SANDY CLAY: moderate brown; mostly fines with little fine sand			RD	
	CL	18.5-25.0 SILTY CLAY: yellowish grey	J-4	4 6	SS	1.5/1.5
20						

Project DESIGN UNIT A-245 Date Drilled 10-7-8-83 Hole No. 18-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	18.5-25.0 SILTY CLAY: cont. stiff; moist; contains numerous cemented zones and nodules		8	SS	
					RD	
22				41	DR	0.9/0.9
		mostly cemented, caliche, becomes hard	C-5	50-	5"	
					RD	
24				6	SS	1.3/1.5
			J-5	11		
				27		
	CL	25.0-28.5 SANDY CLAY: yellowish grey; mostly fines, little fine sand; hard; moist; contains numerous cemented zones and nodules; ferrous staining				
26					RD	
				72	DR	0.7/0.7
			C-6	80-2"		
28					RD	
	CL	28.5-37.8 SILTY CLAY: moderate yellowish brown; very stiff; moist; relatively uncemented at top		4	SS	1.5/1.5
			J-6	8		
30				13		
					RD	
32				26	DR	1.0/1.0
		cemented nodules, becomes hard	C-7	28		
					RD	
34				6	SS	1.3/1.5
			J-7	14		
				25		
					RD	
36		increased cementation				
				25	DR	1.0/1.0
	SC	becoming sandy	C-8	45		
38	SC	37.8-47.8 CLAYEY SAND: dusky yellow; mostly fine sand with some fines; thinly interbedded with sand; silty sand and silty clay beds; hard to very dense; moist to wet			RD	
				18	SS	1.0/1.5
			J-8	25		
40				29		
					RD	
42				37	DR	1.0/1.0
			C-9	39		
					RD	
44		sand lens				

Sheet 2 of 5

Project DESIGN UNIT A-245 Date Drilled 10-7-8-83 Hole No. 18-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	37.8-47.8 CLAYEY SAND: cont.	J-9	12 18 23	SS	1.5/1.5
		ferrous staining			RD	
46		silty clay lens				
			C-10	40 45	DR	1.0/1.0
48	CL	47.8-51.5 SILTY CLAY: dark greenish grey; mostly fines, trace of fine sand; hard; moist; contains some cemented nodules			RD	
			J-10	12 19 25	SS	1.5/1.5
50					RD	
52	ML	51.5-56.0 SANDY SILT: dark greenish grey; mostly fines with some fine sand hard; moist; 4" cemented nodule	C-11	45 50-4.5"	DR	0.8/0.9
					RD	
54				16 24 37	SS	0.0/1.5
					RD	
56	CL	56.0-61.5 CLAY: greenish black; hard; moist				
			C-12	35 50	DR	1.0/1.0
58					RD	
				10 14 26	SS	0.2/1.5
60		grading to silty sand then sandy silt			RD	
62	SP	SAN PEDRO FORMATION 61.5-87.0 SAND: dark greenish grey; mostly fine sand with trace silt; very dense; wet	C-13	65 50-5"	DR	1.0/1.0
					RD	
64			J-11	41 50-4.5"	SS	0.5/0.9
					RD	
66						
		strong sulfur odor	C-14	100	DR	0.5/0.5 partial
68					RD	

Project DESIGN UNIT A-245 Date Drilled 10-7-8-83 Hole No. 18-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	61.5-87.0 SAND: cont.			RD	
			J-12	51	SS	0.5/0.5
70					RD	
72			C-13	97 52-	DR 2"	0.7/0.7
					RD	
74			J-13	34 50-4"	SS	0.7/0.8
					RD	
76		becoming coarser grained				
			C-16	54 70-	DR 3"	0.6/0.7
78		gravelly zone			RD	
			J-14	36 51-	SS 5.5"	0.7/1.0
80		beoming finer grained			RD	
82			C-17	84 84-	DR 2"	0.7/0.7
					RD	
84			J-15	42 50-	SS 5.5"	0.7/1.0 6 10/7/83
					RD	7 am 10/8/83
86						
88		FERNANDO FORMATION 87.0-95.7 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; contains mica; thinly bedded to massive; sulfur odor Physical Condition: little fractured to massive; friable hardness and strength; little weathered	C-18	33 50-	DR 3.5"	0.8/0.8
90					RD	
92			C-19	35 50-4"	DR RD	0.8/0.8

Project DESIGN UNIT A-245 Date Drilled 10-7-8-83 Hole No. 18-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		87.0-95.7 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
				52	DR	0.7/0.7
			C-20	75-3	"	
96		B.H. 95.7 Terminated hole, tremied grout to surface				Complete drilling 7:45

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 18-6

Proj: DESIGN UNIT A-245 **Date Drilled** 10-11-83 **Ground Elev.** 196.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 80.0

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.5 ASPHALT			GB	start drilling 9 am
	CL	FILL 0.5-4.4 SILTY CLAY: brownish black, mostly fines with a trace of fine sand; stiff; moist to wet			AD	
2				4	DR	.5/1.0
			C-1	12		
					AD	
4				2	SS	1.2/1.5
	ML	OLD ALLUVIUM 4.4-6.5 SANDY SILT: moderate yellowish brown; mostly fines with a trace of fine sand; very stiff; dry to moist	J-1	7		
				10		
6					RD	set tub & cased to ~5', mixed mud
	ML	6.5-8.5 SANDY SILT: dark greenish grey; mostly fines with some fine sand; very stiff; moist; contains roots		12	DR	1.0/1.0
			C-2	17		
8					RD	
	SM	8.5-9.8 SILTY SAND: dark greenish grey; mostly fine sand with some silt		7	SS	1.3/1.5
			J-2	10		
10	SC	9.8-13.5 CLAYEY SAND: dark greenish grey; mostly fine sand, little fines; medium dense; moist; contains cemented zones and nodules		13		
					RD	5' of casing added
12				7	DR	1.0/1.0
			C-3	15		
					RD	
14	SM/SP	13.5-15.5 SILTY SAND: dark greenish grey; mostly fine sand with a trace of fines; dense; wet		7	SS	1.0/1.5
			J-3	13		
				20		
16	CL	15.5-23.0 SILTY CLAY: light olive grey; mottled with yellowish grey and ferrous staining; numerous cemented zones and nodules; hard; moist			RD	
				14	DR	1.0/1.0
			C-4	20		
18					RD	
		well cemented zone, caliche		25	SS	1.0/1.5
20			J-4	40		

Project DESIGN UNIT A-245 Date Drilled 10-11-83 Hole No. 18-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	15.5-23.0 SILTY CLAY: cont.		23	SS	rig chatter
					RD	
22			C-5	17	DR	1.0/1.0
				20		
	CL	23.0-32.5 SANDY CLAY: light olive grey to yellowish grey; mostly fines with a trace of fine sand; very stiff; moist; well cemented			RD	
24			J-5	3	SS	1.5/1.5
				8		
				23		
26		26' end of cemented zone, occasional nodules remaining, color change to dusky yellow			RD	
			C-6	7	DR	1.0/1.0
				10		pocket pen 2.0
28					RD	
			J-6	5	SS	1.3/1.5
30				13		
				29		
		very well cemented zone 1' thick Fe and Mn staining			RD	
32			C-7	39	DR	0.9/1.0
				13		
	SC	32.5-41.5 CLAYEY SAND: moderate yellowish brown; mostly fine sand with some fines; very dense; moist			RD	
34		interbeds of silt sand, sand and cemented clay	J-7	5	SS	1.2/1.5
				36		
				44		
36					RD	
			C-8	37	DR	1.0/1.0
				50-5"		
38					RD	
		clayey silt and sand lenses	J-8	8	SS	1.5/1.5
40				32		
				39		
					RD	
42	CL	41.5-45.4 SANDY CLAY: moderate yellowish brown; mostly fines with some fine sand; hard; moist; contains occasional cemented nodules	C-9	21	DR	1.0/1.0
				35		
44					RD	

Sheet 2 of 4

Project DESIGN UNIT A-245 **Date Drilled** 10-11-83 **Hole No.** 18-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	41.5-45.4 SANDY CLAY: cont.	J-9	12 33 21	SS	0.5/1.0
46	CL	45.4-51.0 SILTY CLAY: dark greenish grey; hard; moist; contains cemented nodules			RD	
			C-10	36 50-5.5"	DR	1.0/1.0 pocket pen 4.0
48					RD	
			J-10	15 53	SS	1.0/1.0
50					RD	
52	SM/SP	51.0-54.8 SILTY SAND: dark greenish grey; mostly fine sand, trace of silt; very dense; moist				
			C-11	28 50-5"	DR	1.0/1.0
					RD	
54			J-11	15 24 40	SS	1.5/1.5
	CL	54.8-58.0 SILTY CLAY: dark greenish grey; hard; moist; occasional cemented nodules			RD	
56						
			C-12	18 23	DR	1/0/1.0
58	CL	58.0-59.0 SANDY CLAY: dark greenish grey; mostly fines with a little fine sand; hard; moist			RD	
	SP	SAN PEDRO FORMATION 59.0-80.0 SAND: dark greenish grey; mostly fine sand, trace of silt; very dense; wet	J-12	16 54	SS	1.0/1.0
60					RD	
62			C-13	58 50-2"	DR	0.6/0.6
					RD	
64			J-13	33 50-5.5"	SS	1.0/1.0
					RD	
66						
68			C-14	76 50-2.5"	DR RD	0.5/0.7 disturbed

Project DESIGN UNIT A-245 Date Drilled 10-11-83 Hole No. 18-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	59.0-80.0 SAND: cont.			RD	
			J-14	33 53	SS .	1.0/1.0
70					RD	rig chatter
		occasional sandy gravel lens				
72		silty sand	C-15	56 50-	DR 3"	0.5/0.7
	SM	grading coarser				
					RD	
74			J-15	35 37 50	SS	1.3/1.5
					RD	intense rig chatter
76		beoming gravelly				
			C-16	60 50-3	DR "	0.7/0.7
78					RD	
			J-16	27 54	SS	1.0/1.0
80		B.H. 80.0 Terminated hole, tremied grout to surface				complete drilling 2:30
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

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

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

BORING LOG 18-7

Proj: DESIGN UNIT A-245 Date Drilled 10-9-83 Ground Elev. 195.5'
Drill Rig Failing 1500 Logged By L. Schoeberlein Total Depth 79.7'
Hole Diameter 4 7/8" Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.5 ASPHALT			GB	start drilling 11:45
	CL	FILL 0.5-4.0 SILTY CLAY: dark greenish grey; mostly fines with a trace of fine sand; stiff; moist			AD	
2			C-1	5 14	DR	0.5/1.0 pocket pen 1.75
					AD	
4	CL	OLD ALLUVIUM 4.0-23.5 SANDY CLAY: dark yellowish brown; mostly fines, little fine sand; very stiff; moist; contains some minor wood fragments	J-1	5 12 13	SS	0.6/1.5
6					RD	set tub & cased to 4.5', rig chatter @ 5.2'
	SM	silty sand lenses	C-2	9 14	DR	1.0/1.0 pocket pen > 4.5
8						
			J-2	9 15 24	SS	1.5/1.5
10					RD	
12		sand content increases	C-3	19 22	DR	1.0/1.0
					RD	
14		contains some cemented nodules;	J-3	7 11 18	SS	1.5/1.5
16					RD	
	SC	clayey sand lens	C-4	7 17	DR	1.0/1.0
18					RD	
20		becoming yellowish grey, trace of sand	J-4	5 11	SS	1.5/1.5

Project DESIGN UNIT A-245 **Date Drilled** 10-9-83 **Hole No.** 18-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	4.0-23.5 SANDY CLAY: cont.		13	SS	
					RD	
22				14	DR	1.0/1.0
			C-5	21		pocket pen > 4.5
					RD	
24	CL	23.5-31.0 SILTY CLAY: light olive grey and yellowish grey; very stiff; moist; contains cemented nodules	J-5	4	SS	1.5/1.5
				10		
				14		
26					RD	
				13	DR	1.0/1.0
28			C-6	36		pocket pen > 4.5
					RD	
		8" zone with no cemented nodules	J-6	6	SS	1.5/1.5
30				13		
				19		
					RD	
32	CL	31.0-35.0 SANDY CLAY: light olive grey; mostly fines with a little fine sand; hard; moist		20	DR	1.0/1.0
			C-7	23		pocket pen 4.5
					RD	
34			J-7	5	SS	1.5/1.5
				17		
				35		
36	SM	35.0-40.0 SILTY SAND: light olive grey; mostly fine sand, some fines; very dense; moist; contains silty clay interbeds and occasional cemented nodules			RD	
				37	DR	1.0/1.0
38	SC	and clayey sand	C-8	52		
					RD	
			J-8	8	SS	1.5/1.5
40				20		
	CL	40.0-48.5 SILTY CLAY: greyish green; hard; moist		26		
					RD	
42				17	DR	1.0/1.0
			C-9	31		
44					RD	

Project DESIGN UNIT A-245 Date Drilled 10-9-83 Hole No. 18-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	40.0-48.5 SILTY CLAY: cont.	J-9	7 13 19	SS	1.5/1.5
46					RD	
				27	DR	1.0/1.0
48	ML	grading to sandy silt	C-10	47		pocket pen >4.5
					RD	
	ML	48.5-56.5 SANDY SILT: dark greenish grey; mostly fines, some fine sand; hard; moist	J-10	12 18 29	SS	1.5/1.5
50					RD	
52				19	DR	1.0/1.0
			C-11	47		pocket pen> 4.5
					RD	
54		contains some clayey silt/silty clay lenses, very stiff	J-11	6 11 15	SS	1.5/1.5
56					RD	
	SP	SAN PEDRO FORMATION 56.5-79.7 SAND: dark greenish grey; mostly fine sand, trace of silt, very dense; wet		41	DR	0.9/0.9
58			C-12	50-5	"	
					RD	
			J-12	31 52	SS	1.0/1.0
60					RD	
62				59	DR	0.7/0.7
			C-13	50-2	"	
					RD	
64			J-13	35 50-2.5"	SS	0.5/0.7
					RD	
66						
				113	DR	sample fell out
68					RD	

Sheet 3 of 4

Project DESIGN UNIT A-245 Date Drilled 10-9-83 Hole No. 18-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	56.5-79.7 SAND: cont.			RD	
			J-14	30	SS	0.5/0.7
70				50-3"		
		occasional coarse sand and fine gravel zones			RD	
72			C-14	65	DR	0.4/0.7
				65-3"		
					RD	
74			J-15	41	SS	0.2/0.9
				50-5"		
76					RD	
				63	DR	sample lost
78				65-4"		
					RD	
			C-15	77	DR	0.5/0.7
80		B.H. 79.7 Terminated hole, installed piezometer to bottom, 60-80' slotted, backfilled with pea gravel		50-6.5"		complete drilling and flushing 5:45
82						
84						
86						
88						
90						
92						

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-18 for Design Unit A245. Measurements were made at 5-foot intervals from the ground surface to depths of 130 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-18.

B.1.4 Discussion of Results

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

The error analysis performed for these surveys involved a least squares fit of these data by estimating the mean of the slope ($\bar{V}$) in Table B-1 and the standard deviation of this estimate of the slope. This estimate of the standard deviation was combined with an estimate of the overall accuracy to produce the best estimated velocity (V*). 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 was found to be approximately 1000 feet per second. To depths of about 190 feet, shear wave velocity estimates generally increased to 1200 feet per second. Exception to this trend occurred at Boring CEG-18 where the shear wave velocity decreased from 1100± feet per second between depths of 50 and 85 feet to 900 feet per second.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed also in Boring CEG-18. The crosshole technique for determining shear wave velocities of in-situ materials was utilized in a three-borehole array. The array consisted of boring CEG-18 and two additional holes drilled approximately 15 feet away. All boreholes were drilled to a depth of 100 feet. Compressional wave and shear wave velocities are presented in Table B-2.

B.2.2 Field Procedure

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

B.2.3 Data Analysis

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

B.2.4 Discussion of Results

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

TABLE B-1
DOWNHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
18	10- 80	6038	209	302	13	6040±510	1234	28	62	15	1230±90
	80-150	5176	307	259	16	5180±570	1326	32	66	15	1330±100
	150-192	6373	477	319	8	6370±800	1168	465	58	9	1170±520

$\bar{V}p$ = 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 comrpessional wave.

Vp* = overall accuracy of compressional wave velocity estimate.

Vs* = overall accuracy of shear wave velocity estimate.

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

TABLE B-2
CROSSHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE $\bar{V}p$	σp	Ep	Np	Vp*	SHEAR WAVE $\bar{V}s$	σs	Es	Ns	Vs*
18	10						687	14	34	14	690±50
	15						881	12	44	13	880±60
	20	6030	---	600	1	6030±600	1070	53	53	11	1070±110
	25	8351	1030	418	5	8350±1450	1107	24	55	11	1110±80
	30	7263	587	363	6	7260±950	1290	40	65	7	1290±100
	35	5423	1328	271	5	5420±1600	1246	103	62	5	1250±170
	40	6393	613	320	7	6390±930	1140	27	57	8	1140±80
	45	6957	187	298	6	6960±490	1190	33	60	8	1190±90
	50	6207	1083	310	4	6210±1390	1121	37	56	6	1120±90
	55	5768	670	288	6	5770±960	1045	34	52	8	1050±90
	60	5338	458	267	10	5340±460	958	33	48	12	960±80
	65	5549	490	277	8	5550±770	959	9	48	12	960±60
	70	5390	880	270	10	5390±1150	928	12	46	12	930±60
	75	6096	641	305	7	6100±950	908	6	45	8	910±50
	80	6390	1155	315	5	6310±1470	999	31	50	10	1000±80
	85	5403	---	540	1	5400±540	937	29	47	6	940±80
	90	4591	---	460	1	4590±460	1093	10	55	7	1090±70
	95	4970	---	500	1	4970±500	1212	48	61	8	1210±110
	97	4660	---	470	1	4660±470	1124	34	56	8	1120±90

$\bar{V}p$ = 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 comrpessional wave.
Vp* = overall accuracy of compressional wave velocity estimate.
Vs* = overall accuracy of shear wave velocity estimate.
Ns = number of points used for straight line fit of shear wave velocity data.

Approved for publication 4/84 by JAD

TRACE IDENTIFICATION

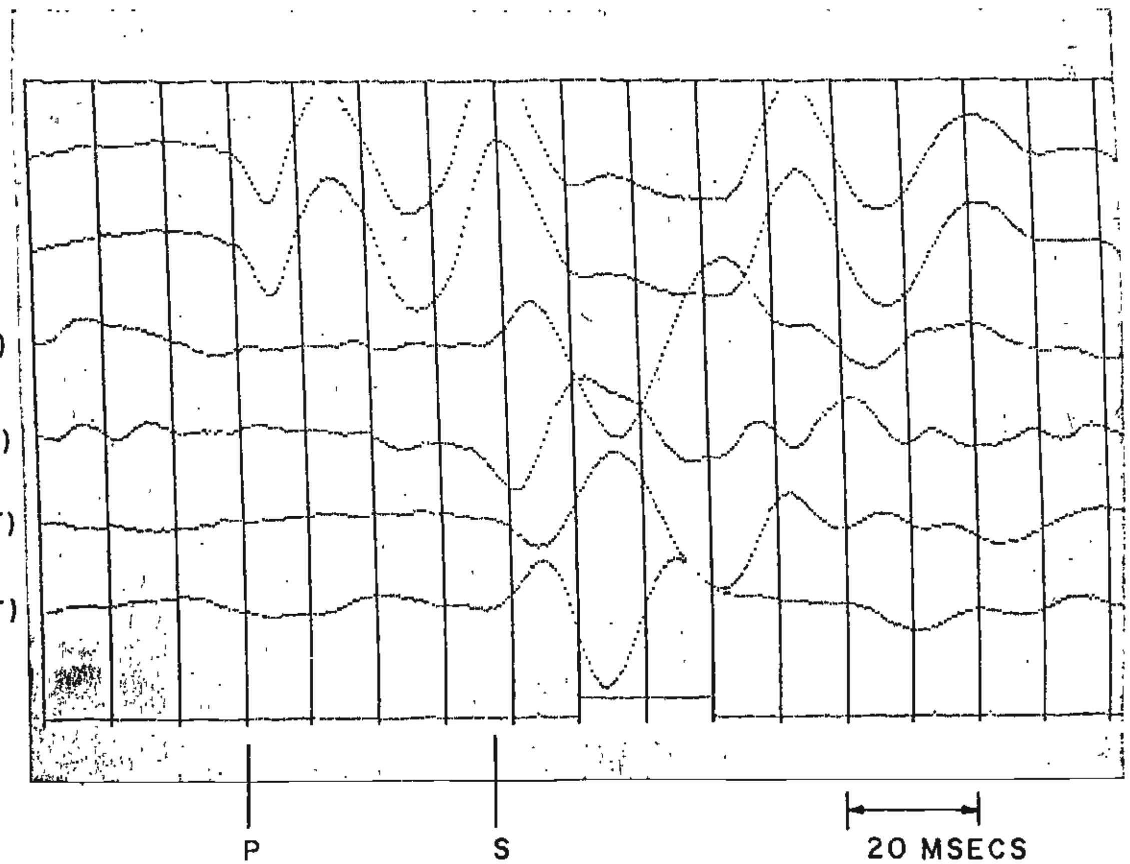
VERTICAL (DOWN)

HORIZONTAL 1 (WEST)

HORIZONTAL 1 (EAST)

HORIZONTAL 2 (WEST)

HORIZONTAL 2 (EAST)



BOREHOLE: 13

DEPTH: 70 FT

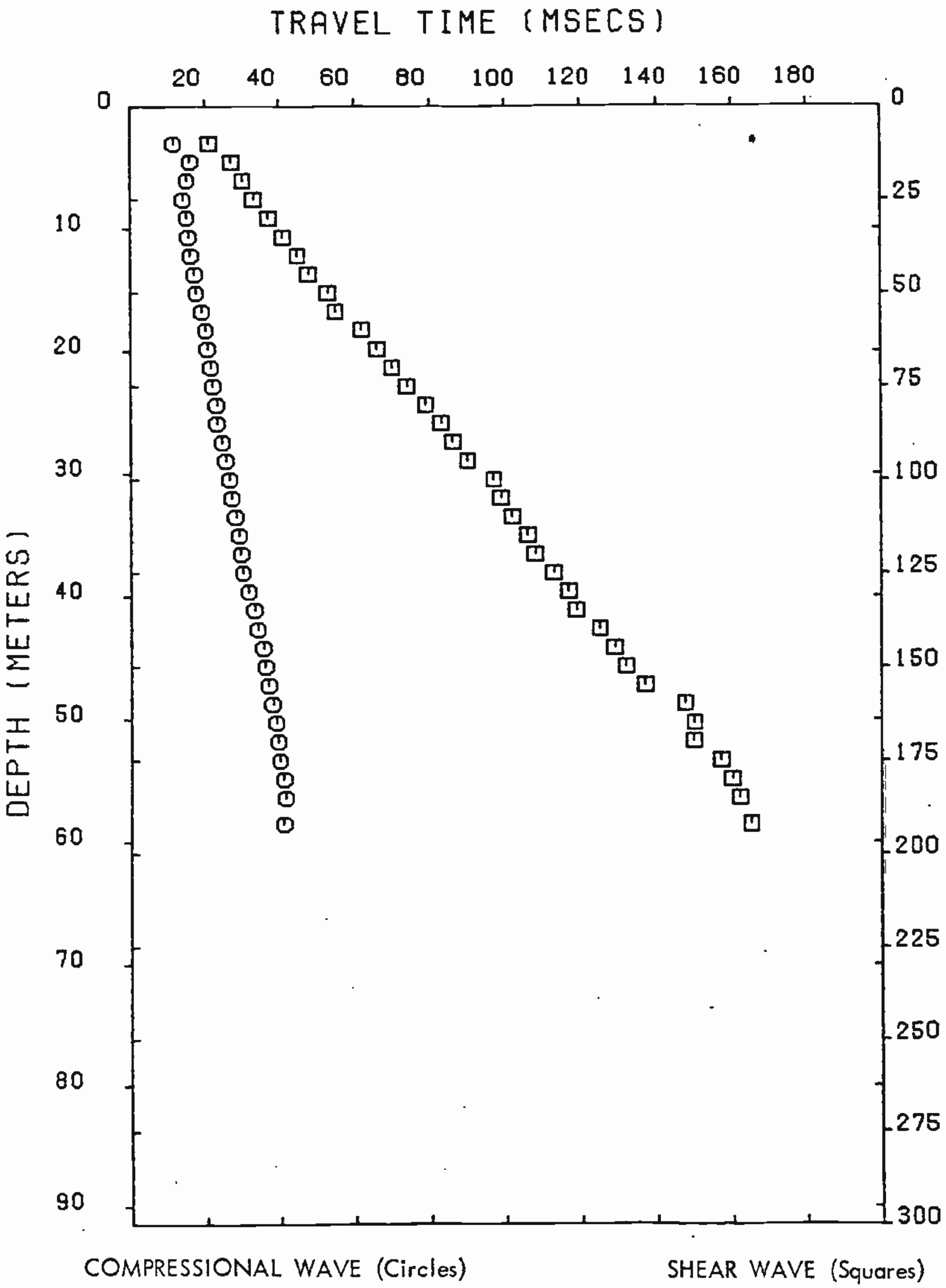
Converse Consultants
Geotechnical Engineering and Applied Sciences

DOWNHOLE SAMPLE RECORD

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. B-1



DOWNHOLE TRAVEL TIME PROFILE - BORING 18

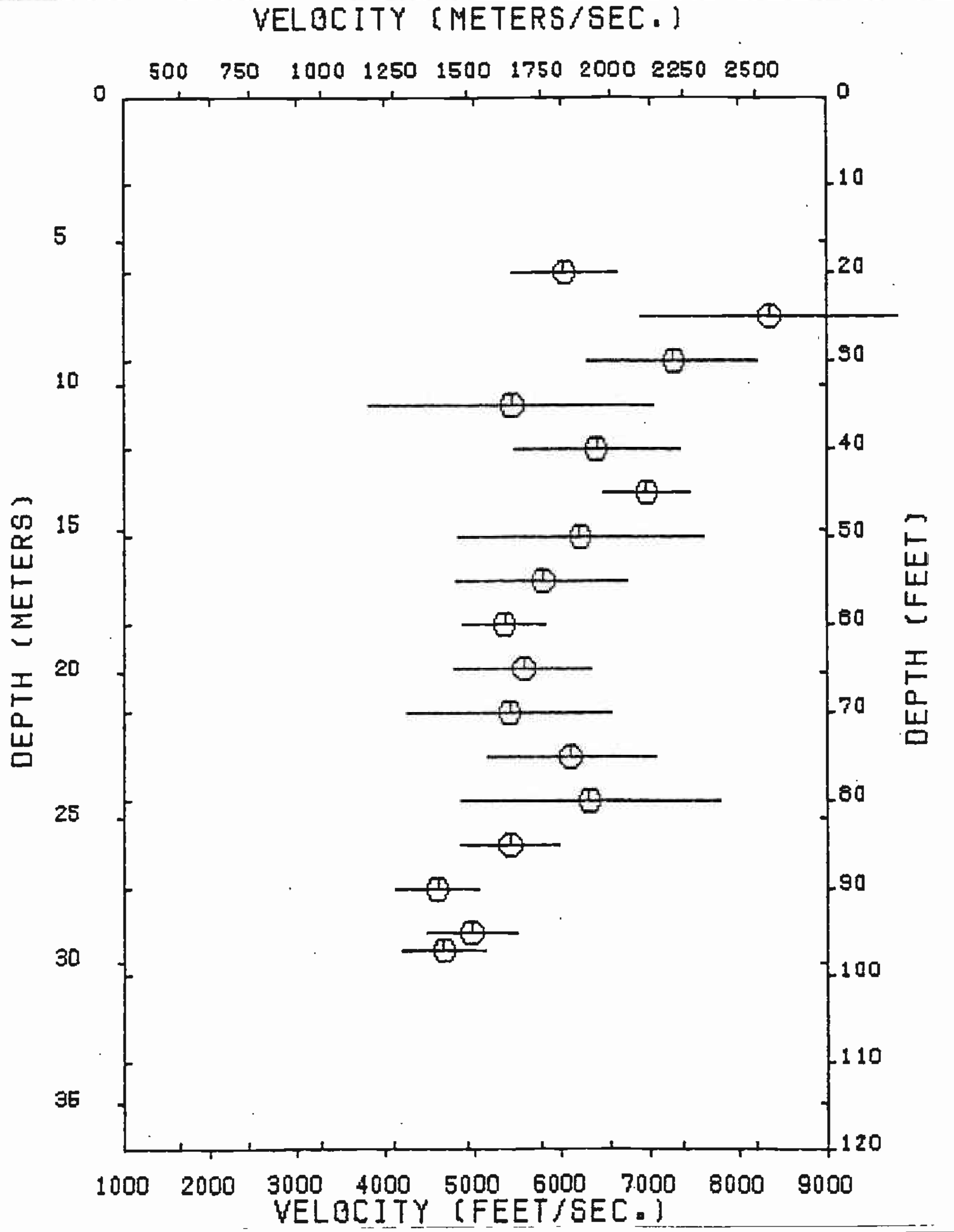
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Figure No.
B-2

Approved for publication 4/8/84 by [illegible]



Approved for publication 4/84 by

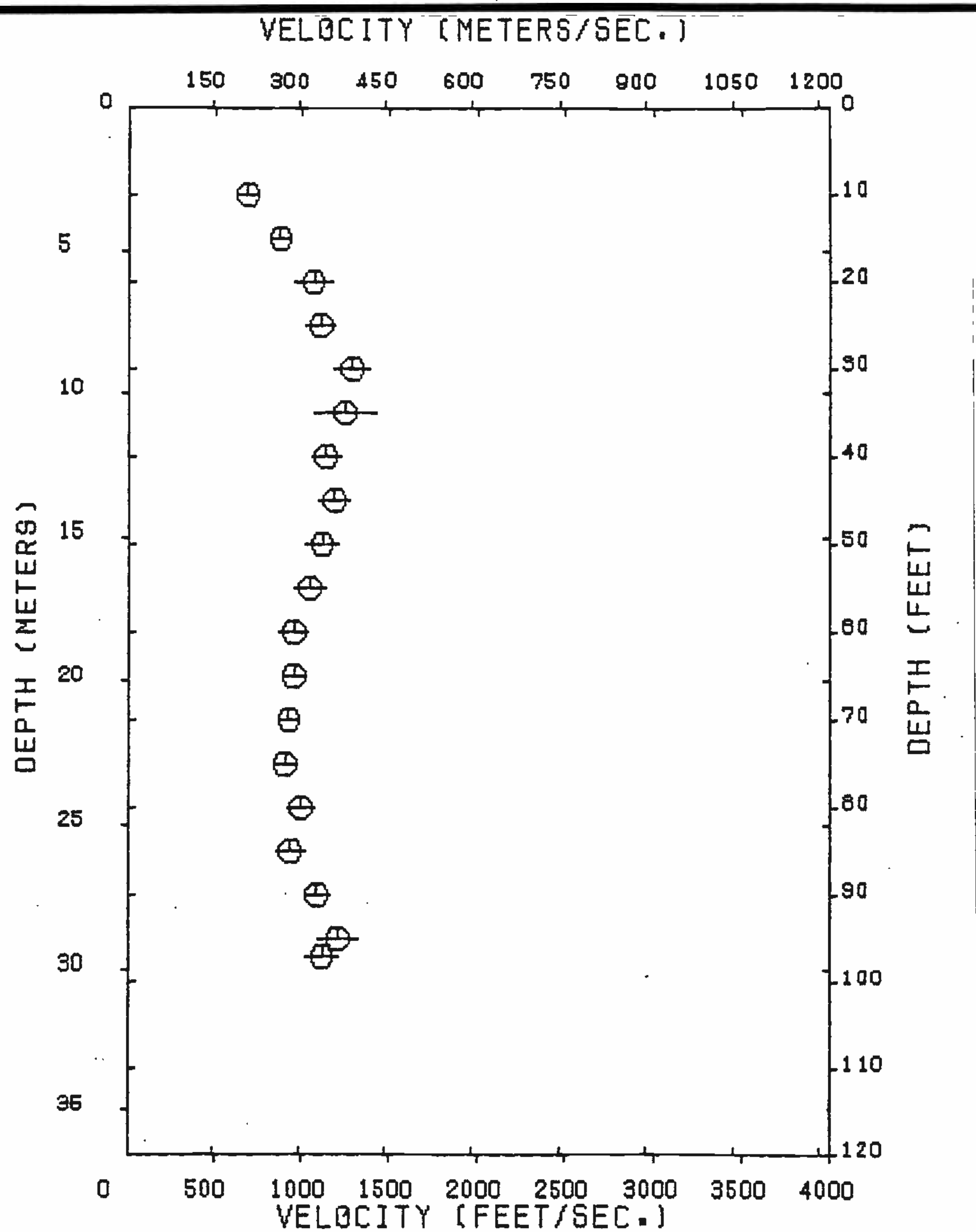
COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 18

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. B-3



SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 18

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B-4

Approved for publication ______ by ______

Appendix C

Geotechnical Laboratory Testing

APPENDIX C GEOTECHNICAL LABORATORY TESTING

C.1 INTRODUCTION

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

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

- Index or identification tests which included visual classification, grain-size distribution, Atterberg Limits, moisture content, and unit weight testing;

- Engineering properties testing which included unconfined compression, triaxial compression, direct shear, consolidation, permeability, and resonant column.

The laboratory test data from the present investigation are presented in Table C-1, while data from the 1981 geotechnical investigation are presented in Table C-2. The geologic units listed in these tables are described in Section 5.0 of the report. Figures C-1 through C-10 summarize strength and modulus data for fine-grained alluvium, San Pedro sand, and bedrock at this site and other nearby station sites.

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-11 through C-14.

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-15 and Tables C-2 and C-3.

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 Tables C-1 and C-2.

C.2.5 Unit Weight

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

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

C.2.6 Specific Gravity and Porosity

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

C.3 ENGINEERING PROPERTIES: STATIC

C.3.1 Unconfined Compression

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

C.3.2 Triaxial Compression

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

C.3.2.1 Consolidated Undrained (CU) Tests

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

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

Results of the triaxial compression tests are presented on Figures C-16 through C-20.

C.3.3 Direct Shear

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

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

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

C.3.4 Consolidation

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

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

Results of consolidation tests on the undisturbed samples are presented on Figures C-21 through C-26.

C.3.5 Permeability

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

C.4 ENGINEERING PROPERTIES: DYNAMIC

C.4.1 Resonant Column

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

C.4.1.1 Sample Preparation and Handling

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

C.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures, (σ_{3c}), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressures and progressing to the higher confining pressures. At each confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures C-27 and C-28.

C.4.1.3 Apparatus

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

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

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

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

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

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

C.4.1.4 Data Reduction

Data obtained from the resonant column tests were reduced in accordance with the ASTM "Suggested Methods of Test for Shear Modulus and Damping of Soils by the Resonant Column"* using a proprietary computer program developed by Converse Consultants, Inc. Graphs of the test results are presented on Figures C-27 and C-28.

*ASTM Special Technical Publication 479.

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	c, ksf	SPECIFIC GRAVITY	POROSITY	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-1	1	5	Clay	A_4	98	24				8.8								
	2	9	Sandy Clay	A_4	96	28	55	29		1.2								
	3	14	Clay	A_4	105	23				3.2								
	4	19	Sandy Clay	A_4	113	18				2.9			2.69	.327				
	5	24	Sandy Clay	A_4	116	17	28	11							X	X		X(2)
	6	28	Clayey Sand	A_3	118	12							2.71	.302				
	7	34	Silty Clay	A_4	111	17				1.5								
	8	38	Clayey Sand	A_3	111	18			1.8×10^{-4}						X			
	9	43	Silty Clay	A_4	87	33	70	39							X	X		X(2)
	10	49	Silty Sand	A_3	117	18			1.4×10^{-4}						X		X	
	11	53	Silty Sand	SP	108	18							2.71	.361			X	
	12	63	Sand	SP	101	23			1.3×10^{-3}						X			
	13	63	Sandy Silt	SP	109	19							2.65	.341			X	
	14	73	Silty Sand	SP	103	22					30	0.40						
	15	78	Silty Sand	SP	107	14												X(2)

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	SPECIFIC GRAVITY	POROSITY	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-1	16	83	Silty Sand	SP	102	22												
	17	86	Claystone	C	91	27							2.75	.470				
	18	90	Claystone	C	83	37											X	
	19	-	Claystone	C	-	-							2.54					
18-2	1	3	Silty Clay	A_4	106	21				4.6								
	2	9	Sandy Silt	A_4	109	20	28	7		4.3								
	3	14	Clayey Silt	A_4	96	28	38	12		1.7								
	4	19	Clay	A_4	102	18												
	5	24	Sandy Clay	A_4	91	32				1.7								
	6	29	Clayey Sand	A_3	105	21												
	7	34	Sandy Clay	A_3	107	22												
	8	39	Clayey Sand	A_3	101	24			1.2×10^{-5}		24	0.55			X			
	9	39	Sandy Clay	A_4	103	23	23	15							X	X		X(2)
	10	49	Sandy Clay	A_4	100	24											X	
	11	54	Sandy Silt	A_4	94	29			2.8×10^{-5}						X			

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-2	12	59	Sand	SP	112	18											X	
	13	64	Sand	SP	104	22											X	
	14	69	Sand	SP	102	21			2.5×10^{-3}						X			X(3)
	15	74	Disturbed	-	-	-												
	16	79	Sand	SP	109	15												
	17	94	Sand	SP	114	13												
	18	89	Silty Claystone	C	75	44												
	19	95	Silty Claystone	C	87	34												
18-3	1	4	Clay	A_4	107	16												
	2	8	Clayey Silt	A_4	97	26	50	19		6.1								
	3	13	Sandy Clay	A_4	111	19				4.7								
	4	18	Sand	A_3	107	19												
	5	23	Silty Clay	A_4	99	24	62	37							X			X(2)
	6	29	Sandy Clay	A_3	101	23			1.0×10^{-5}						X			
	7	33	Disturbed	-	-	-												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-3	8	38	Clayey Sand	A_3	106	19			1.0×10^{-4}						X			
	9	43	Sandy Clay	A_3	105	20				4.5								
	10	48	Silty Clay	A_4	92	31											X	
	11	53	Clayey Silt	A_4	99	27											X	
	12	58	Sandy Clay	A_4	98	27											X	
	13	63	Silty Sand	SP	108	19			1.7×10^{-4}		25	0.85			X			
	14	68	Sand	SP	102	19											X	
	15	73	Sand	SP	102	21												
	16	78	Sand	SP	104	19											X	
	17	83	Sand	SP	91	30												
	18	88	Clayey Siltstone	C	76	44												
	19	93	Clayey Siltstone	C	81	38												
18-4	1	3	Clayey Sand	A_3	110	15												
	2	8	Clay	A_4	97	24				3.8								
	3	13	Sandy Clay	A_4	110	20												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-4	5	23	Clay	A_4	103	23				1.7								
	7	33	Silty Clay	A_4	99	25				2.7								
	9	43	Sandy Clay	A_4	102	23												
	10	48	Silty Clay	A_4	100	22				2.5								
	11	53	Silty Sand	A_4	105	22												
	13	72	Gravelly Sand	SP	111	16												
	15	82	Sand	SP	106	20												
	17	88	Silty Claystone	C	90	32												
18-5	2	9	Clayey Silt	A_3	118	13				3.9								
	4	18	Sandy Clay	A_4	108	18				4.6								
	6	28	Sandy Clay	A_4	109	16				1.3								
	8	33	Silty Clay	A_4	101	26												
	10	48	Silty Clay	A_4	85	37				5.1								
	12	58	Silty Clay	A_4	88	34												
	14	68	Sand	SP	98	23												

TABLE C-1 LABORATORY TEST DATA

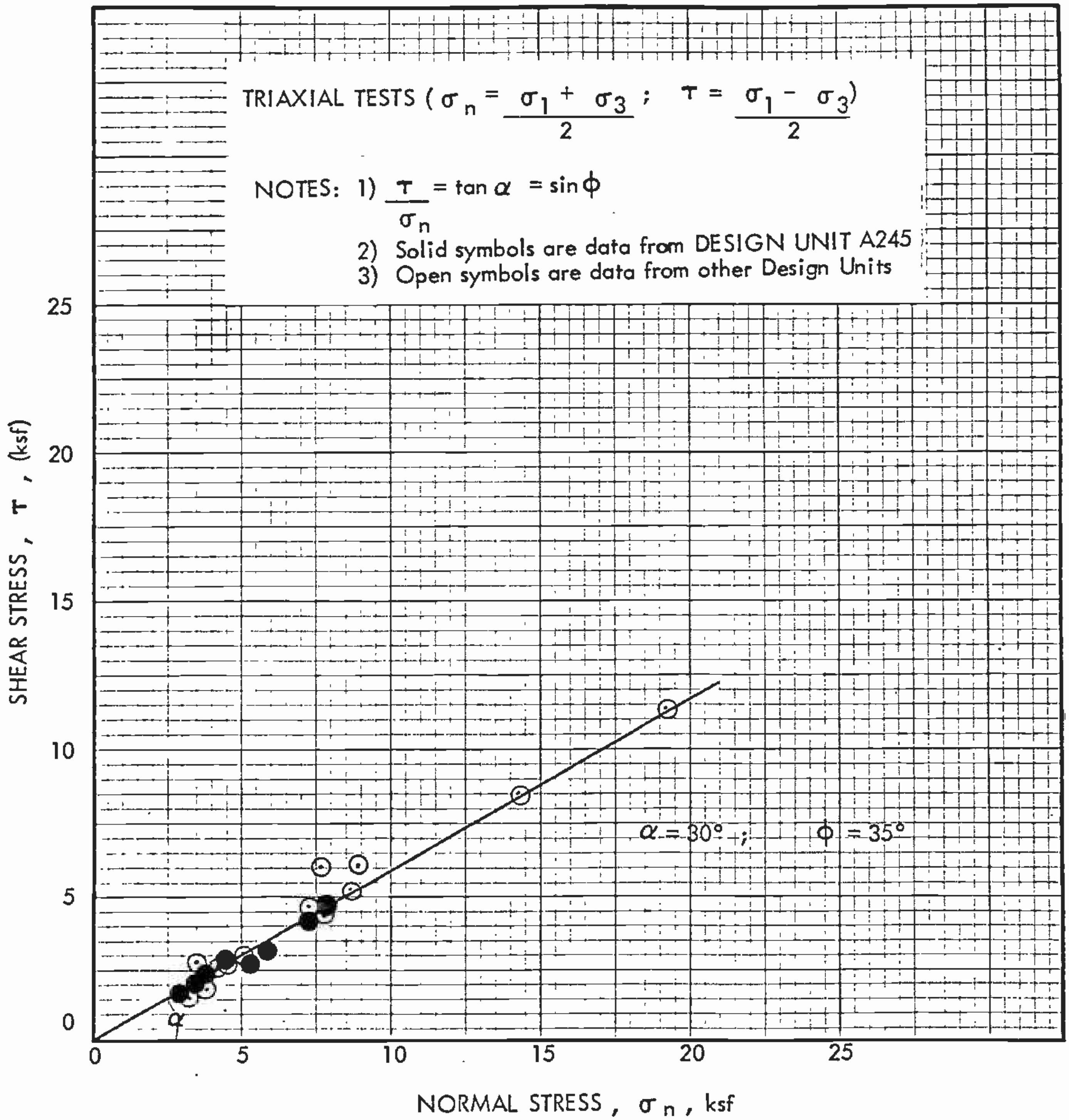
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-5	16	78	Sand	SP	114	15												
	18	88	Silty Claystone	C	71	48												
	20	95	Silty Claystone	C	86	35												
18-6	1	3	Silty Clay	A_4	102	22												
	3	13	Clayey Sand	A_3	111	18												
	4	18	Silty Clay	A_4	100	24				5.1								
	5	23	Sandy Clay	A_4	99	24												
	7	33	Clayey Sand	A_3	104	24												
	9	43	Sandy Clay	A_4	101	25												
	11	53	Silty Sand	SP	103	24												
	13	63	Sand	SP	105	21												
	15	73	Silty Sand	SP	113	16												
18-7	2	7	Sandy Clay	A_4	108	17												
	4	17	Clayey Sand	A_3	113	15												
	6	27	Silty Clay	A_4	106	22												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
18-7	8	37	Clayey Sand	A_3	102	22												
	10	48	Silty Clay	A_4	98	28												
	12	58	Sand	SP	109	19												
	14	67	Sand	SP	106	19												

TABLE C-2 Comprehensive List of Soils Engineering Properties from Laboratory Tests (continued)

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits LL	Atterberg Limits PI	Particle Size Cumulative % Passing Sieve No. 4	40	200	Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Coefficient of Consolidation Cc (in/in per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	c, ksf	One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
18	C1	20	Sandy Clay	A_4	97	26																X	
	C2	40	Clayey sand	A_4	110	19							4.1E-6		2.64	34.1	33	0.46					
	C2	41	Clayey sand	A_4	107	21																	
	C3	65	Gray sand	SP	100	21							1.4E-4		2.66	38.9	32	0.29					
	C3	65	Gray sand	SP	103	23																	
	S1	76	Silty claystone	C	97	26																	
	S2	100	Siltstone	C	92	30	50	22				97.7											
	S3	116	Silty claystone	C	73	45	75	15				115											
	S4	130	Siltstone & claystone, folded	C	79	40						109											
	S5	147	Clayey siltstone	C	83	37						91.7											
	S5	147	Clayey siltstone	C	81	38							6.1E-8			51.2							
	S7	184	Claystone, massive	C	89	32						80.4											



Approved for publication 4/84 by [signature]

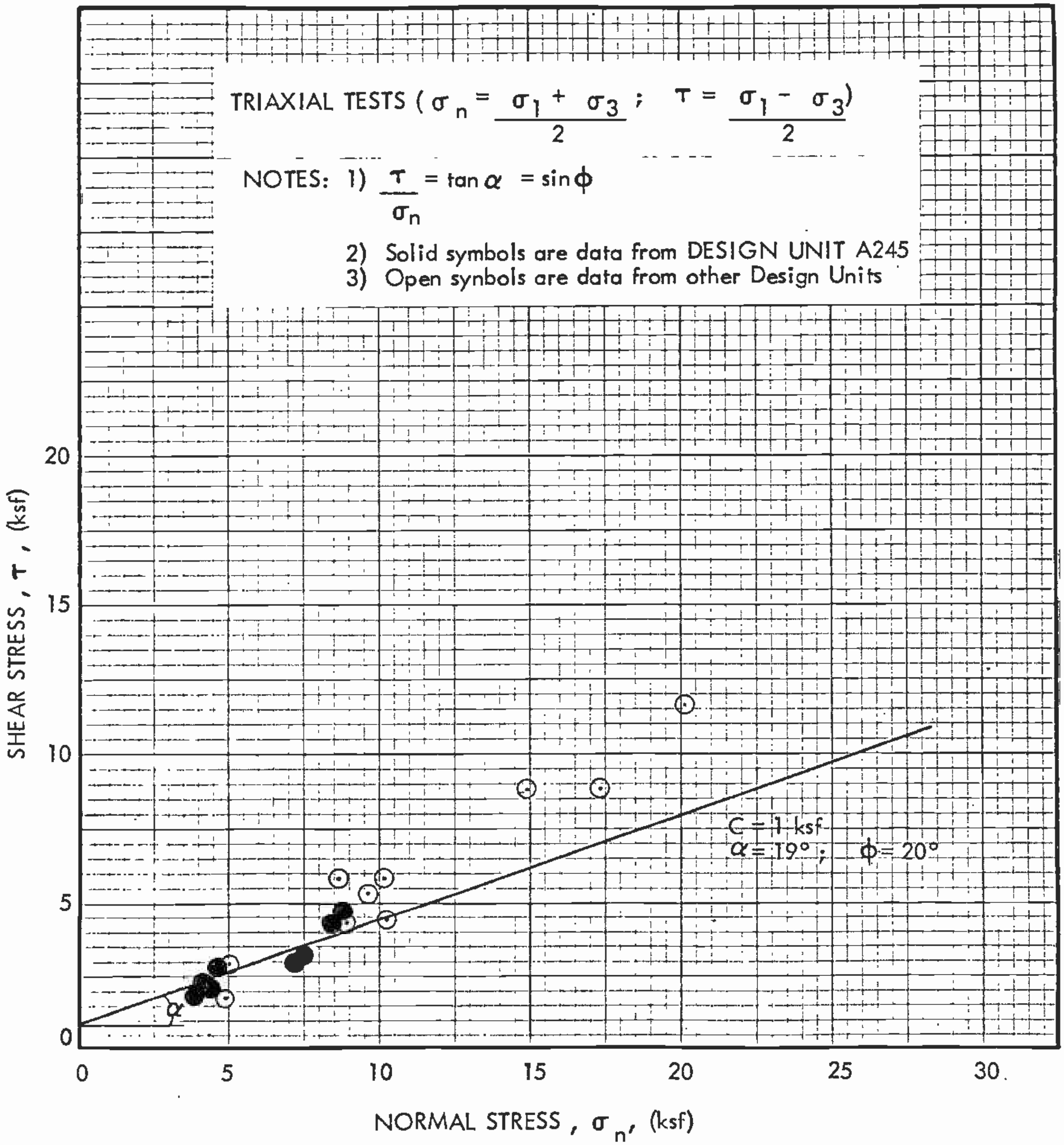
SUMMARY OF EFFECTIVE STRENGTH DATA - FINE-GRAINED ALLUVIUM

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-1



SUMMARY OF TOTAL STRENGTH DATA - FINE-GRAINED ALLUVIUM

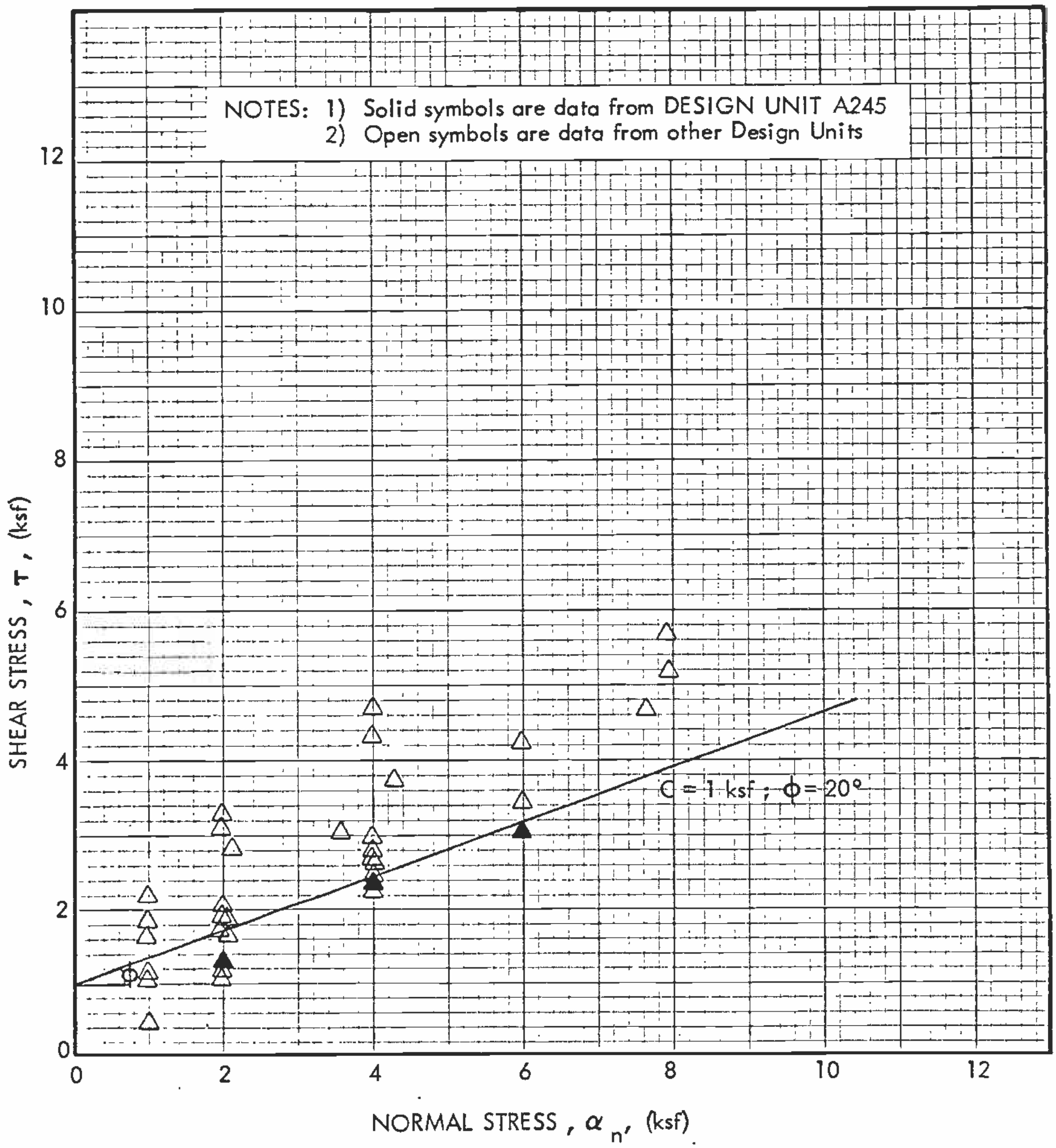
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No. C-2

Approved for publication 4/84 by [signature]



Approved for publication 4/84 by

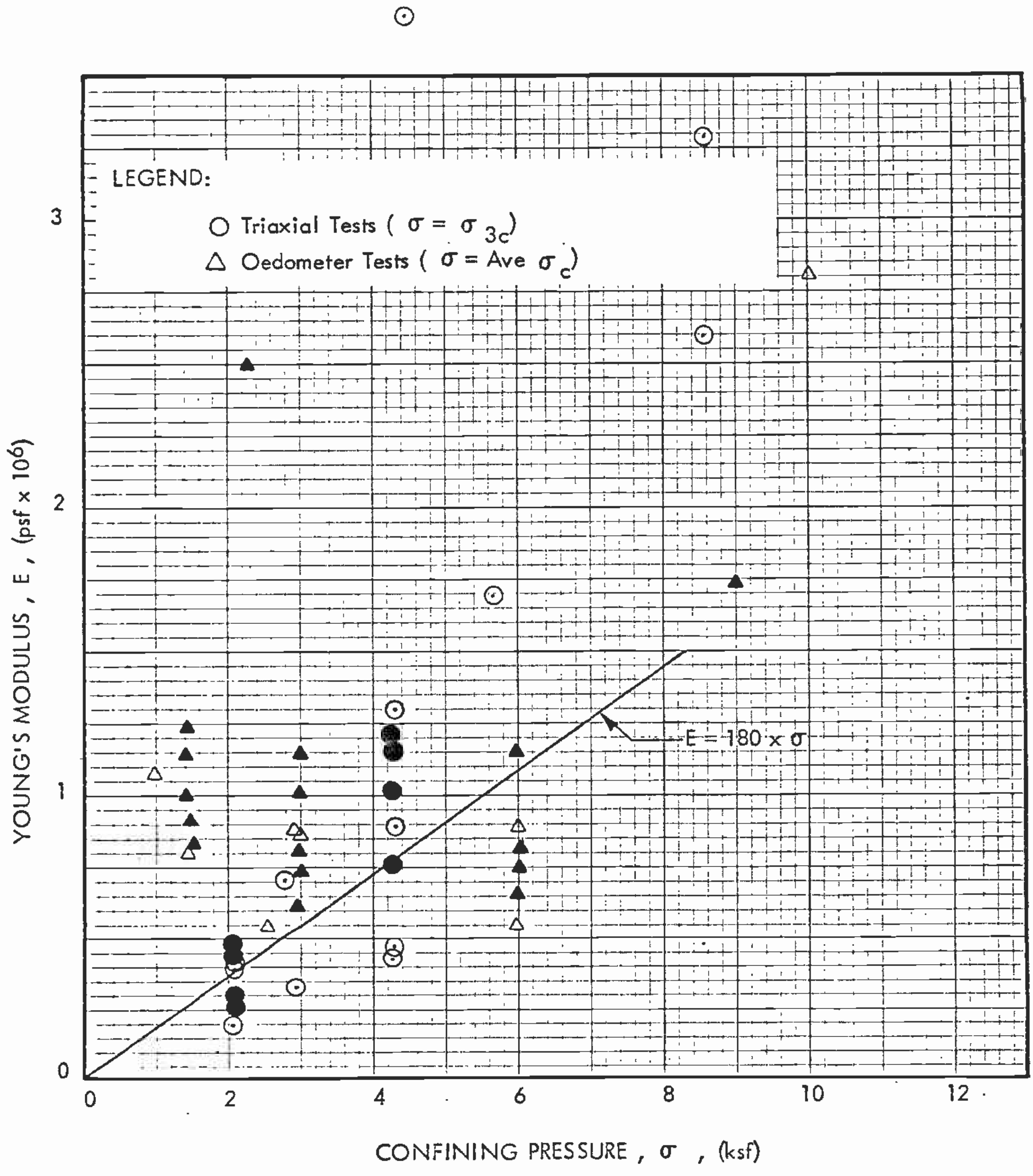
SUMMARY OF DIRECT SHEAR TEST RESULTS - FINE-GRAINED ALLUVIUM

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-3



SUMMARY OF MODULUS DATA - FINE-GRAINED ALLUVIUM

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

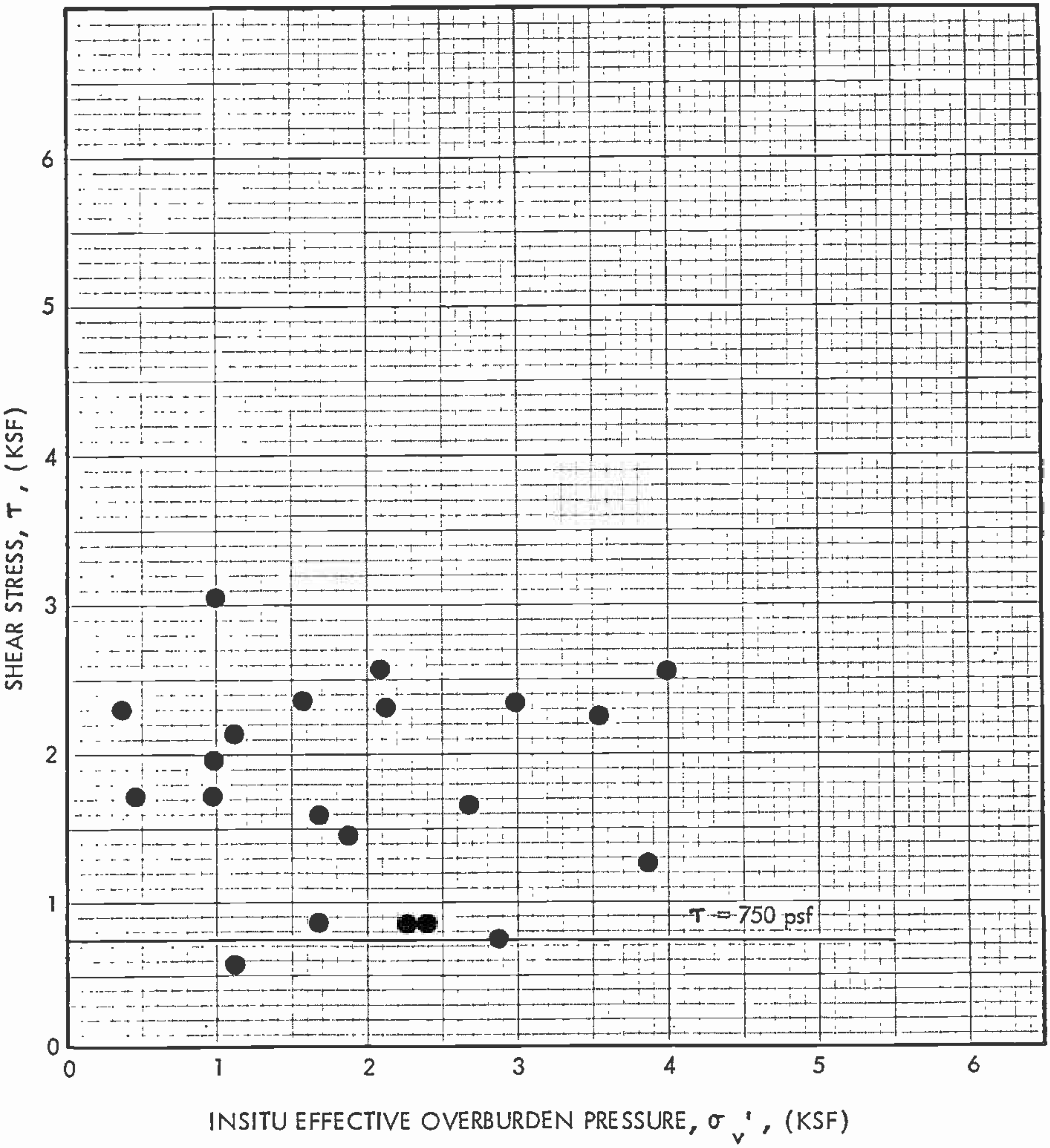
Project No.
83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No.
C-4

Approved for publication 4/84 by JHD

NOTE: $\tau = \dfrac{\text{Unconfined Compression Strength}}{2}$



Approved for publication 4/84 by [signature]

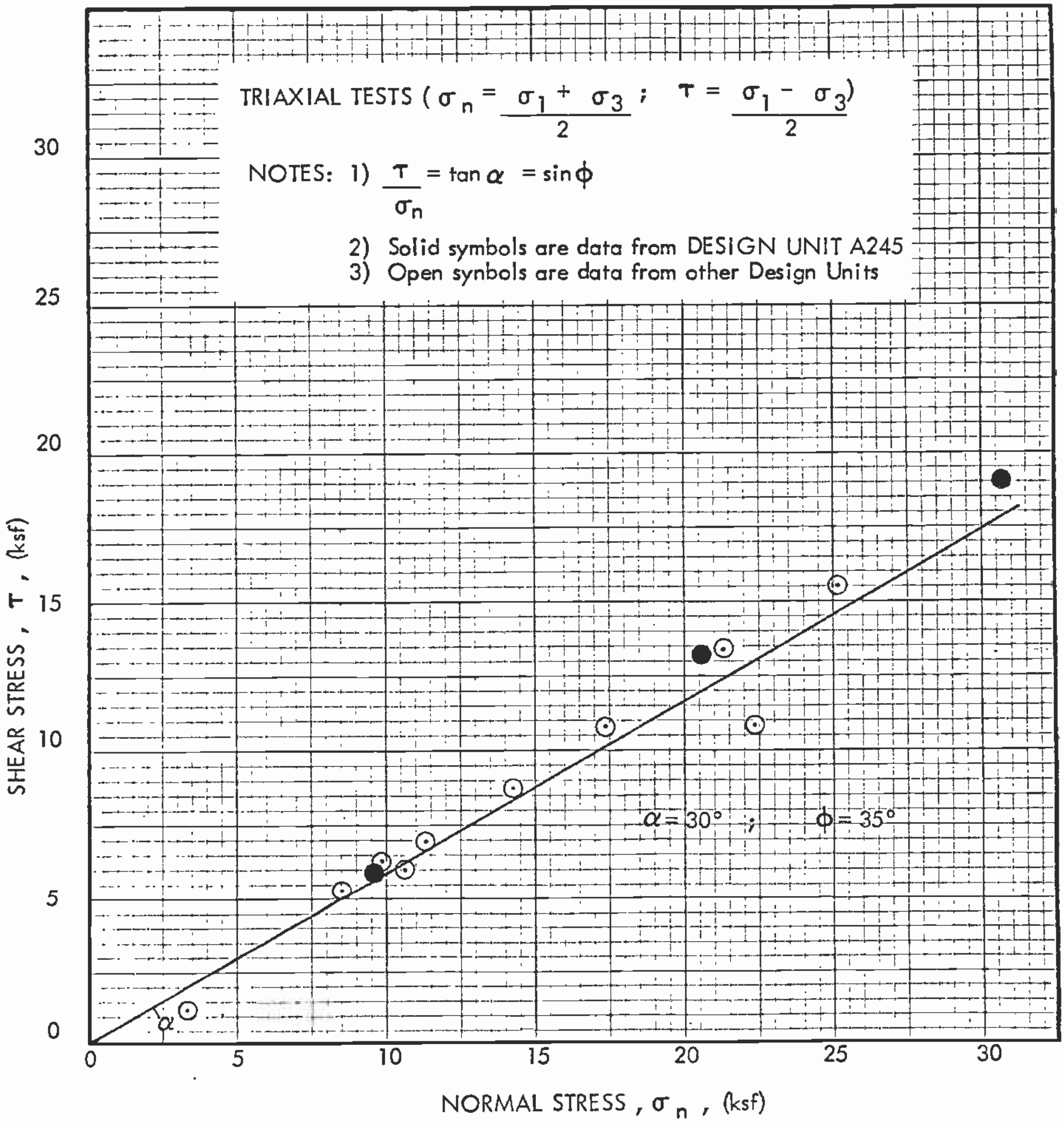
SUMMARY OF UNCONFINED COMPRESSION TESTS - ALLUVIUM

DESIGN UNIT A245
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 1/5/81 by [signature]

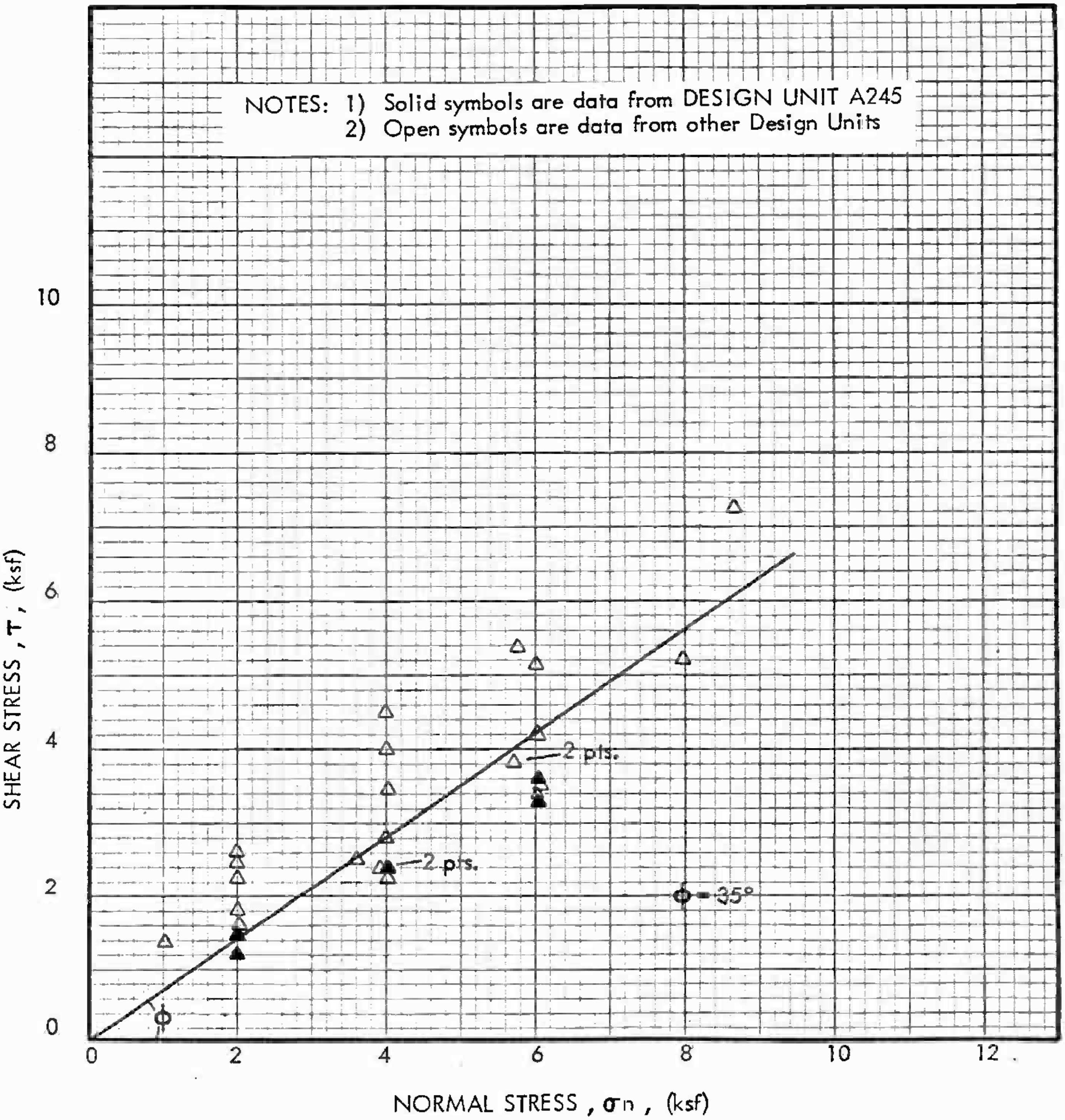
SUMMARY OF EFFECTIVE STRENGTH DATA - SAN PEDRO SAND

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-6



Approved for publication 4/84 by [signature]

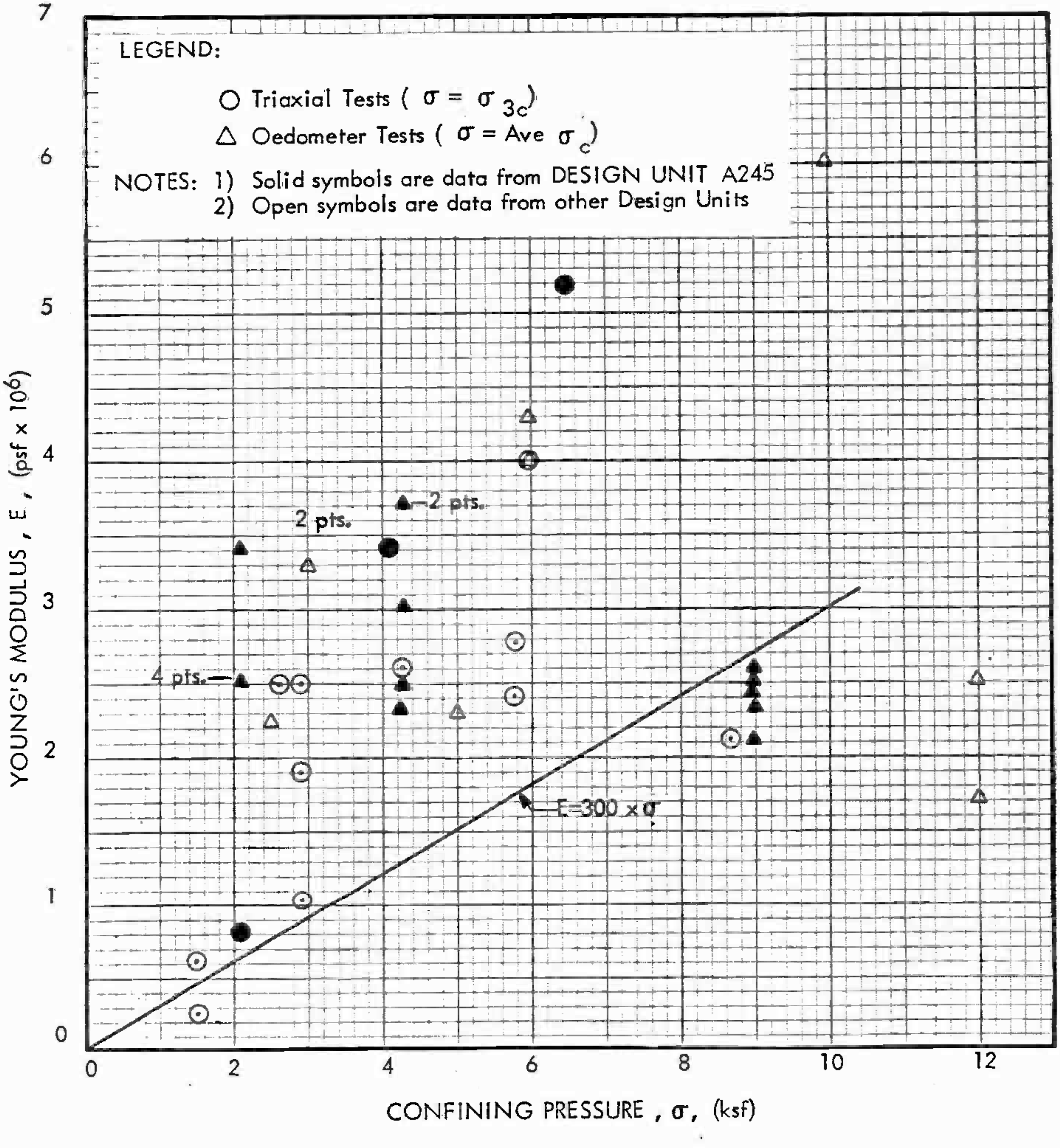
SUMMARY OF DIRECT SHEAR TEST RESULTS - SAN PEDRO SAND

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-7



SUMMARY OF MODULUS DATA - SAN PEDRO SAND

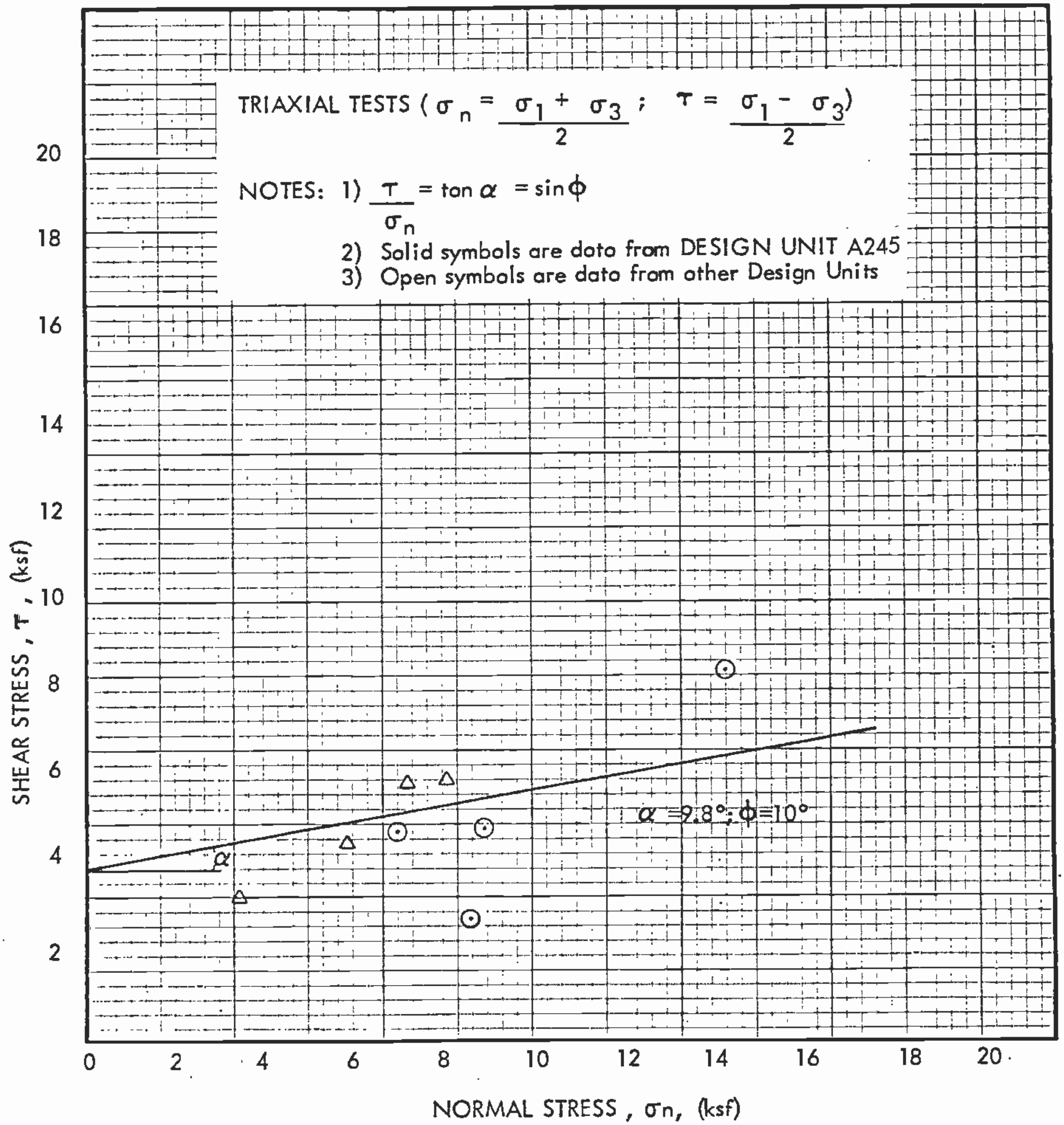
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. C-8

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 4/8/ by [signature]



Approved for publication 4/84 by [signature]

SUMMARY OF TOTAL STRENGTH DATA - BEDROCK

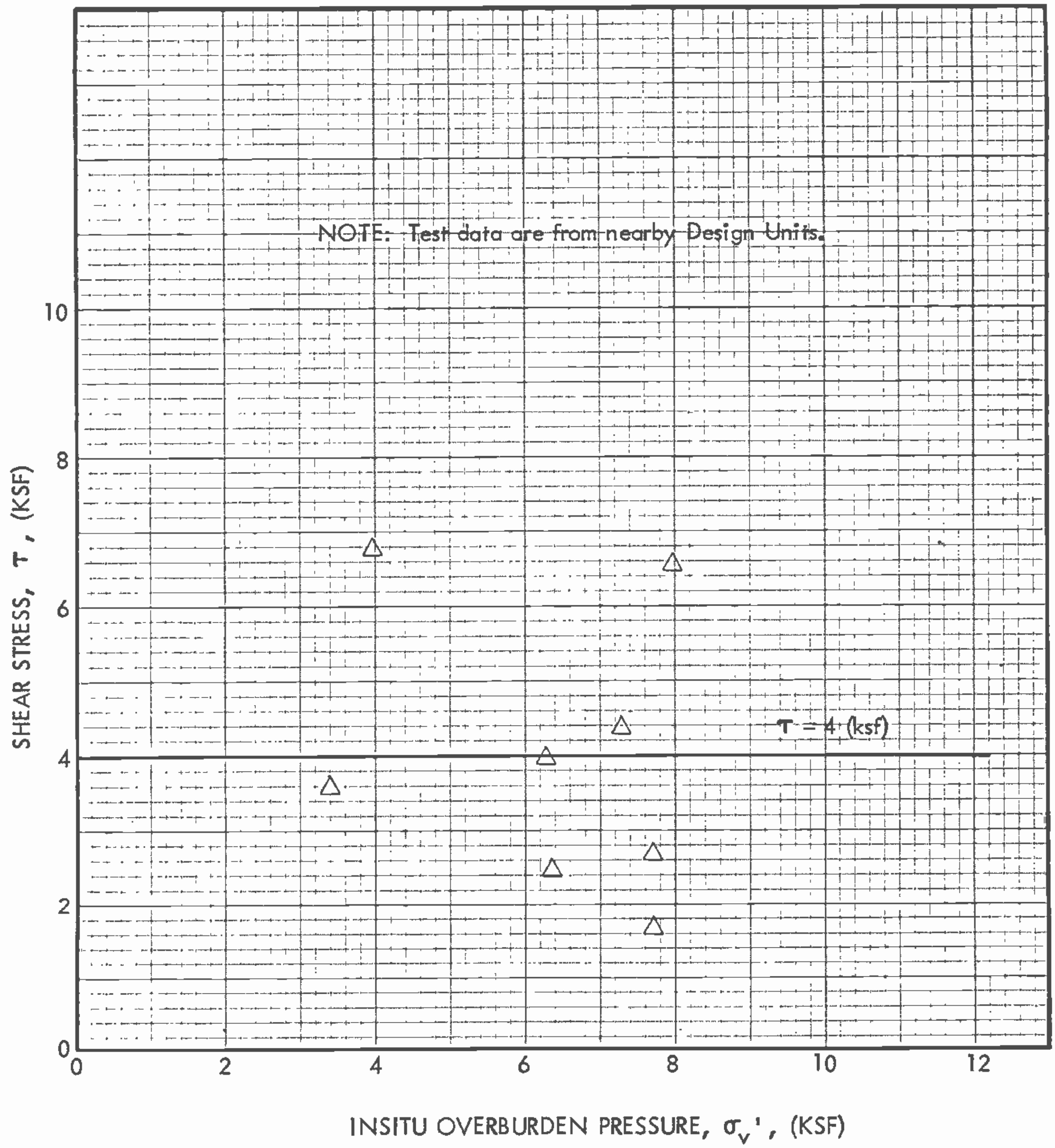
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No. C-9

NOTE: $\tau = \frac{\text{Unconfined compression strength}}{2}$



Approved for publication 4/84 by [signature]

SUMMARY OF UNCONFINED COMPRESSION TESTS - BEDROCK

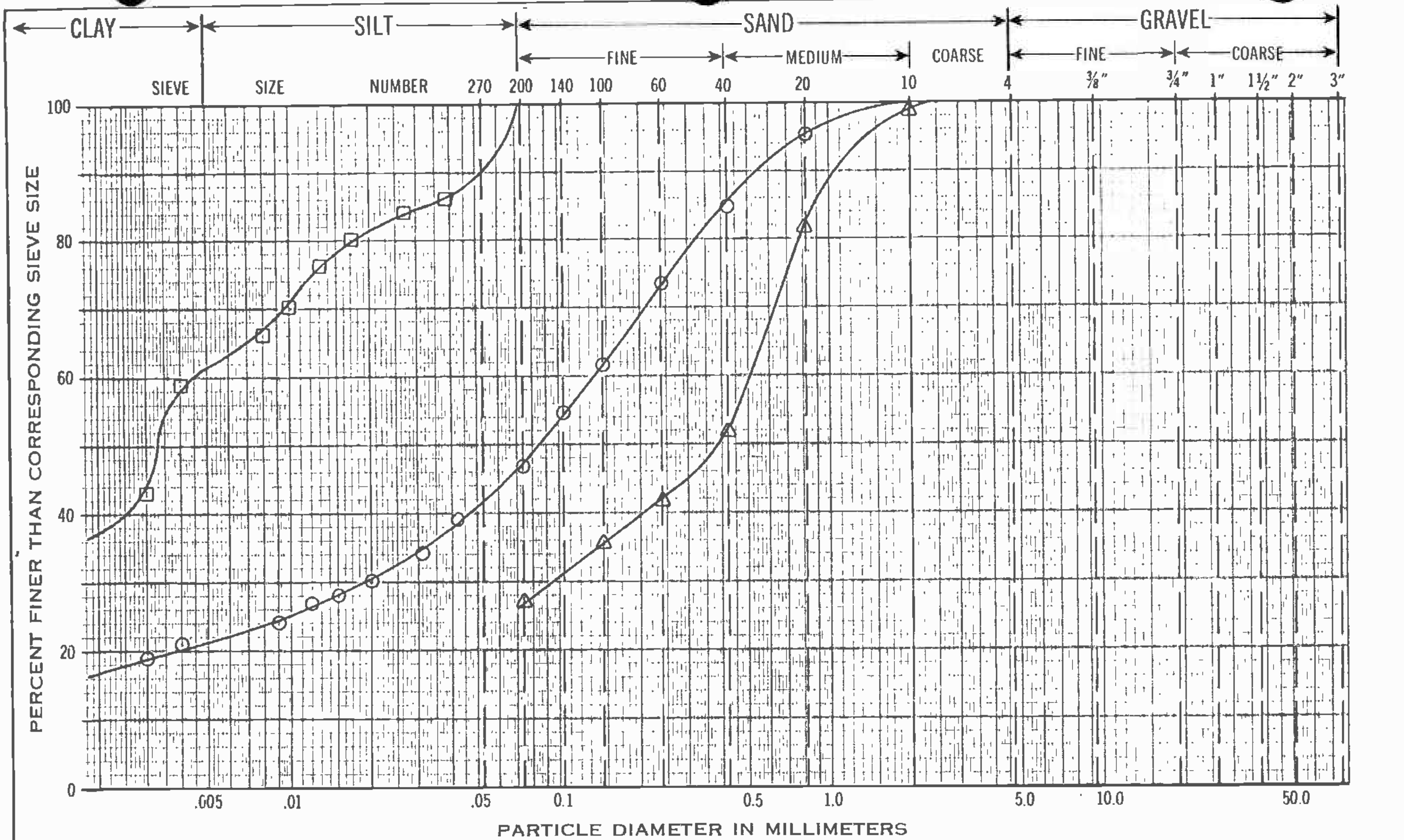
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-10

FORM NO. D-3-10 Approved for publication 4/84 by [illegible]



SYMBOL	BORING	SAMPLE	DEPTH
○	18/1	C-5	22'
△	18/1	C-8	37'
□	18/1	C-9	43'

GRAIN-SIZE DISTRIBUTION CHART

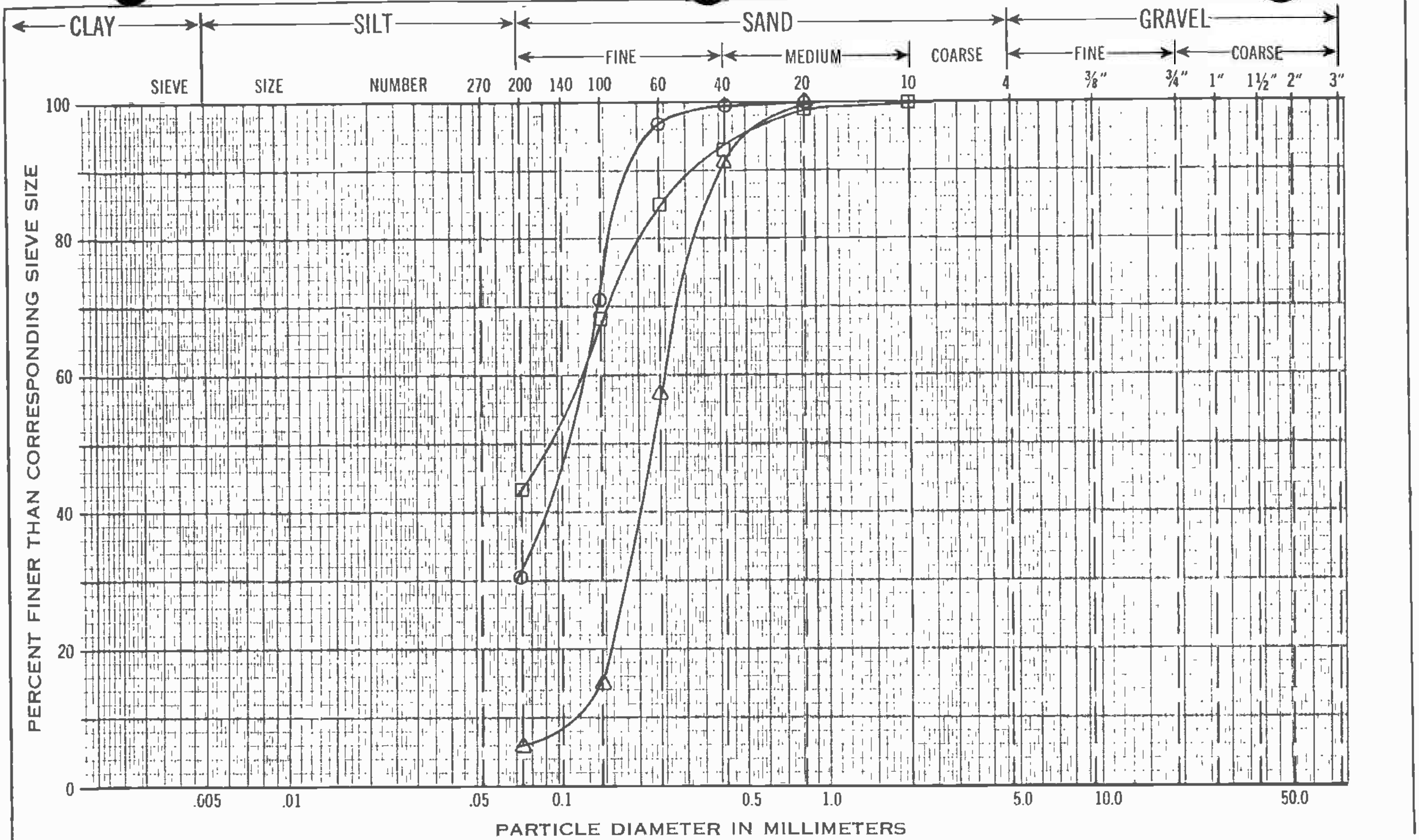
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-11

FORM NO. D-3-10 Approved for publication 4/84 by



SYMBOL	BORING	SAMPLE	DEPTH
○	18/1	C-10	47'
△	18/1	C-12	63'
□	18/2	C-8	38'

GRAIN-SIZE DISTRIBUTION CHART

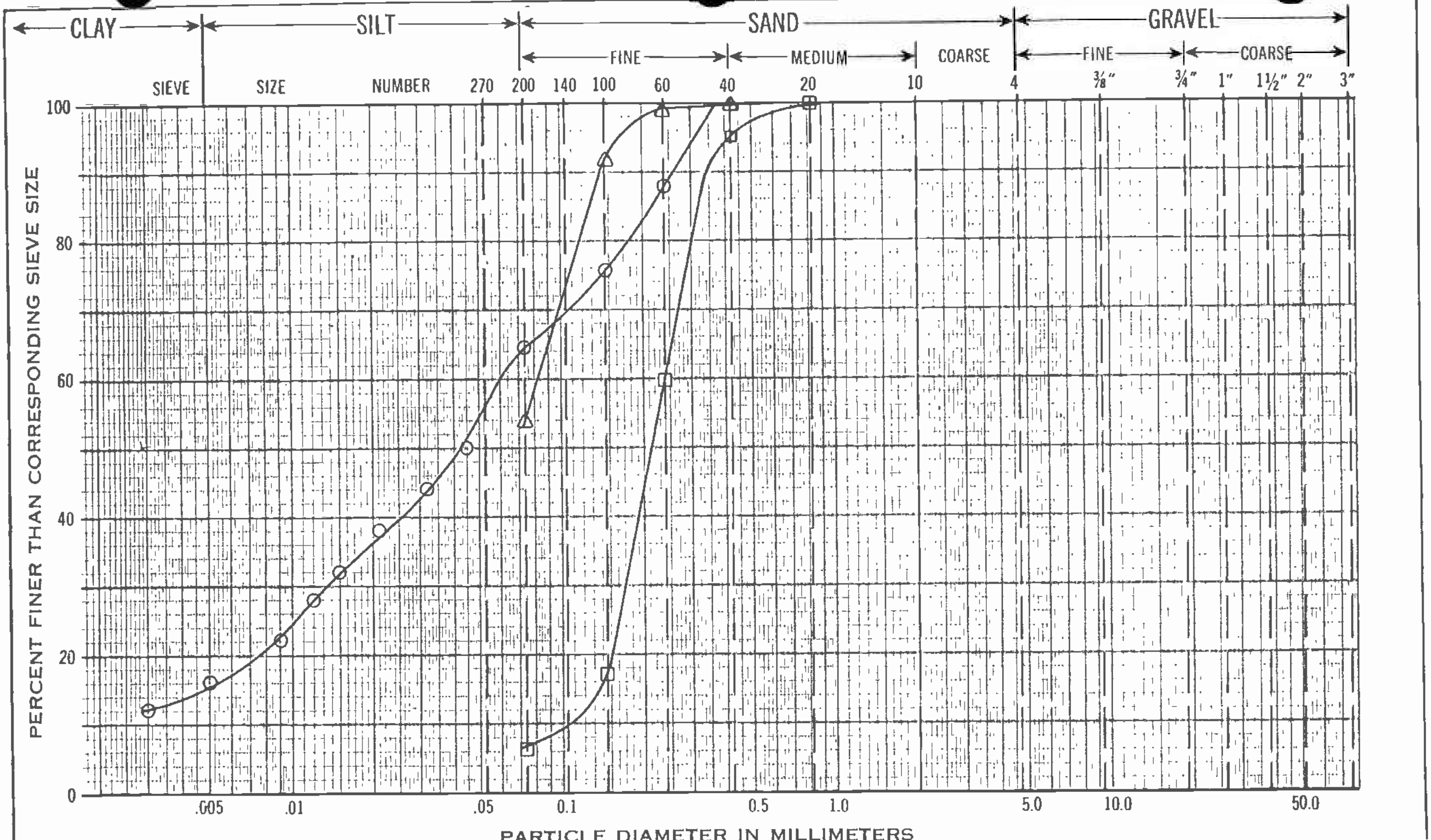
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Figure No.
C-12

FORM NO. D-3-10/8_ Approved for publication 4/84 by [signature]



SYMBOL	BORING	SAMPLE	DEPTH
○	18/2	C-9	44'
△	18/2	C-11	53'
□	18/2	C-14	68'

GRAIN-SIZE DISTRIBUTION CHART

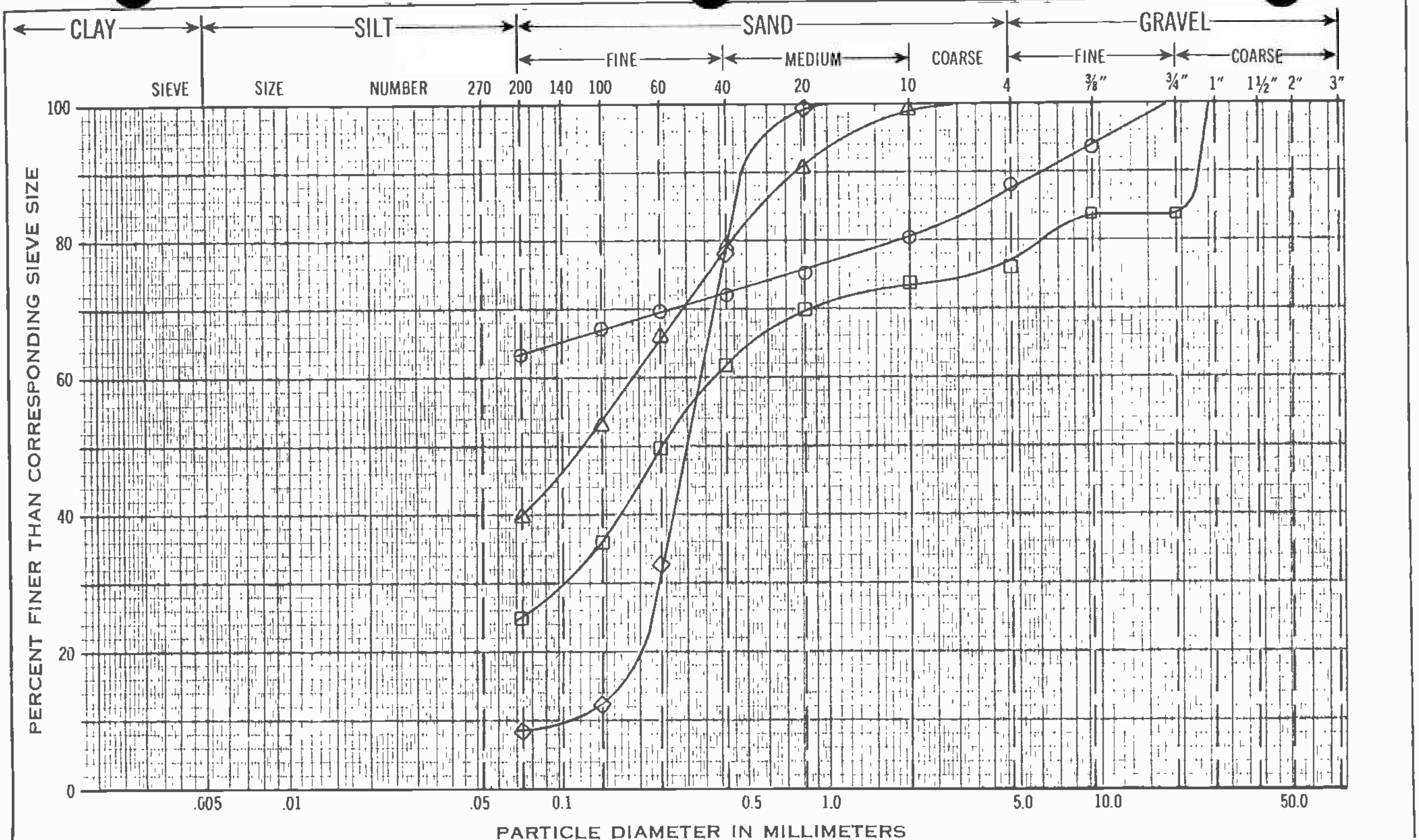
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No. C-13

FORM NO. D-3-10/ Approved for publication 4/84 by [signature]



SYMBOL	BORING	SAMPLE	DEPTH
○	18/3	C-5	23'
△	18/3	C-6	29'
□	18/3	C-8	38'
◇	18/3	C-13	63'

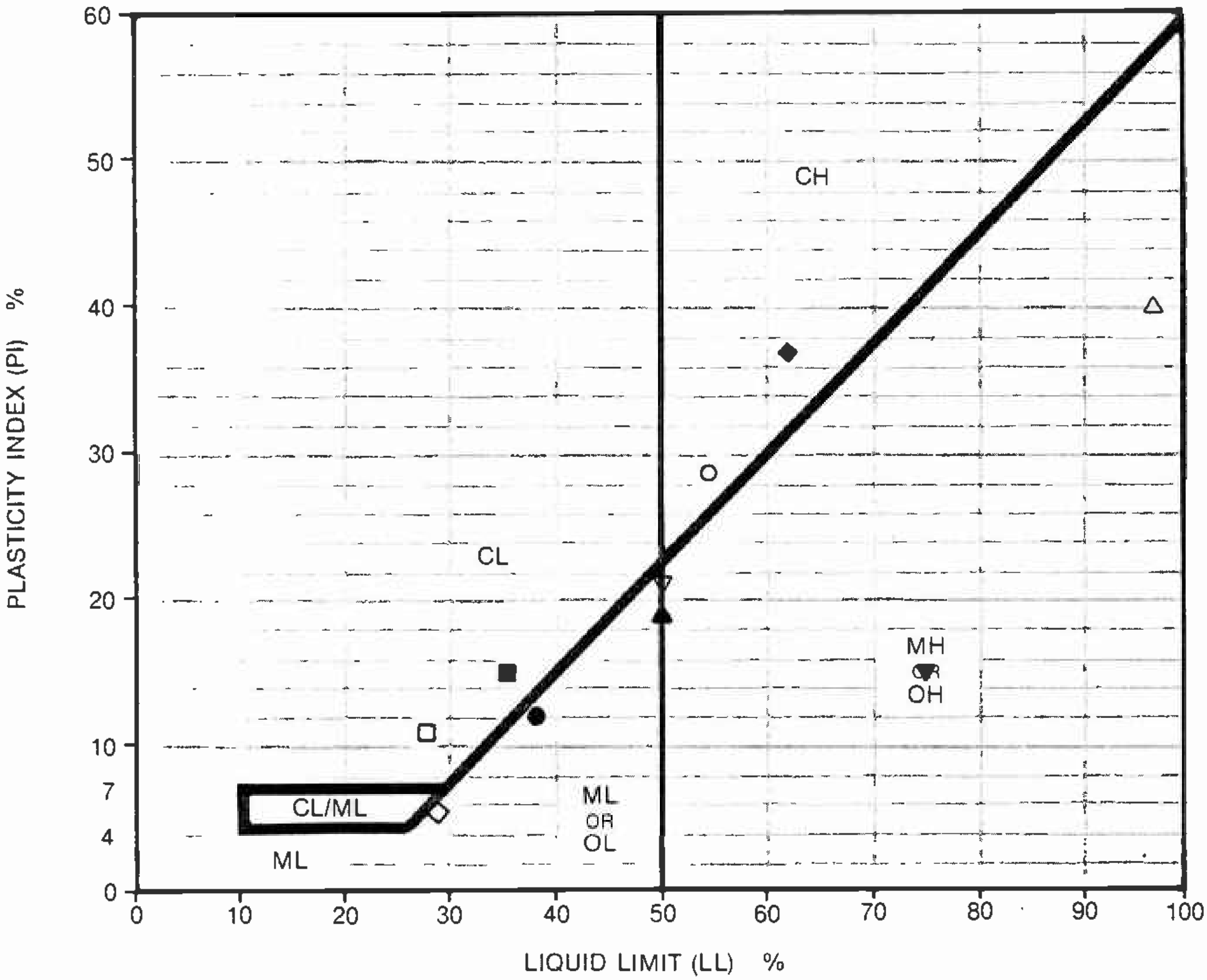
GRAIN-SIZE DISTRIBUTION CHART

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No.
C-14



Symbol	Classification and Source				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
○	B-18/1	C-2	8'-9'	CH	55	26	29	
□	B-18/1	C-5	23'-24'	CL	28	17	11	47%
△	B-18/1	C-9	42'-43'	MH/CL	96	40	56	100%
◇	B-18/2	C-2	8'-9'	ML	28	22	6	
●	B-18/2	C-3	13'-14'	ML	38	26	12	
■	B-18/2	C-9	43'-44'	CL	37	22	15	64%
▲	B-18/3	C-2	7'-8'	ML/MH	50	31	19	
◆	B-18/3	C-5	22'-23'	CH	62	25	37	63%
▽	B-18	S-2	100'	ML/MH	50	28	22	
▼	B-18	S-3	115'	MH	75	60	15	

PLASTICITY CHART

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

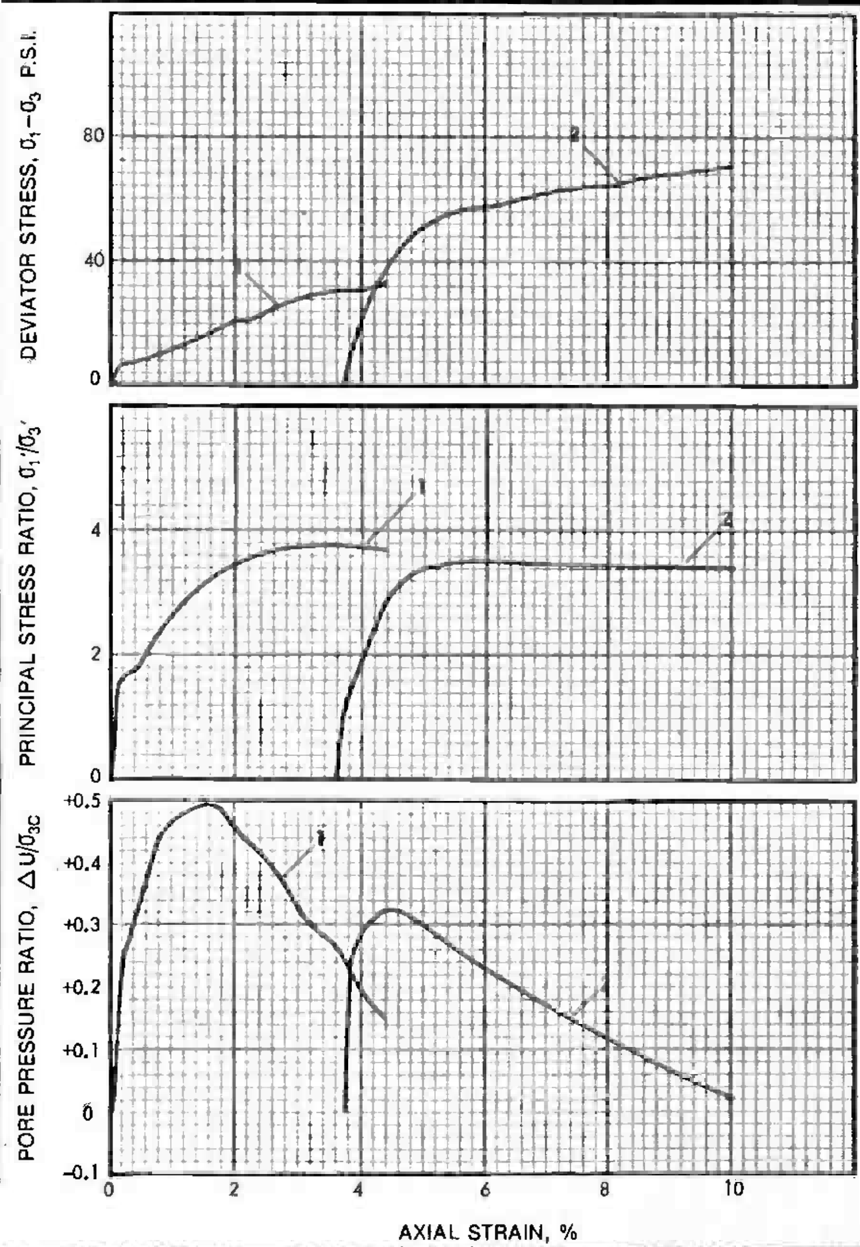
Project No. 83-1140

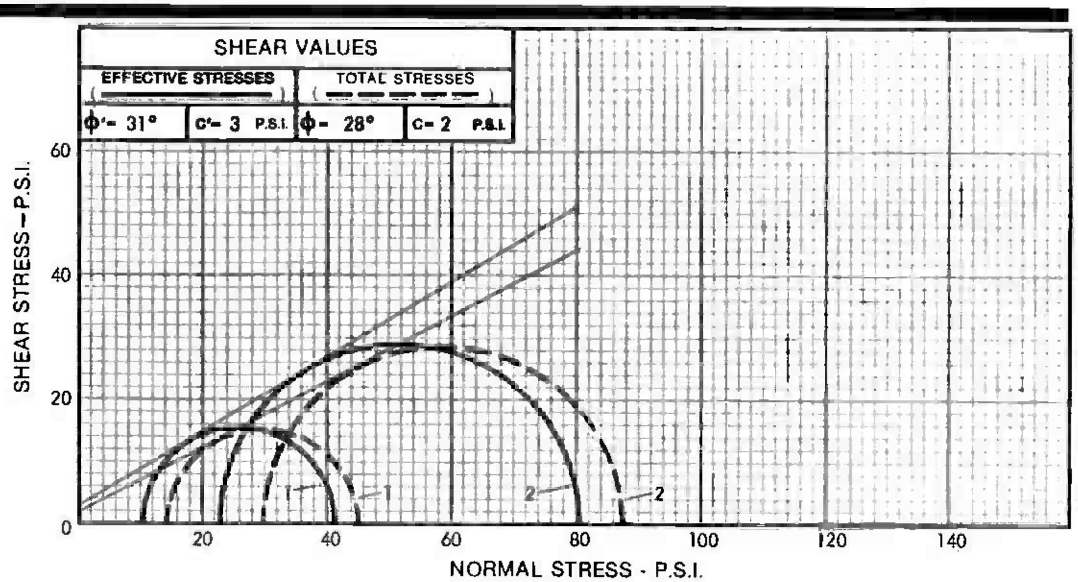
Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-15

Approved for publication 4/84 by [signature]

Approved for publication 4/8/1 by JAD





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)	
C-5	18/1	C-5	22.5-23.0	SC	5.0	2.42	115.5	16.5	5 Ring Converse

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (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.)	
C-5	1	15	30.6	4.0	11.0	41.6	Tx CUE Progressive
C-5	2	30	57.7	7.0	23.0	80.7	

TRIAXIAL COMPRESSION TESTS

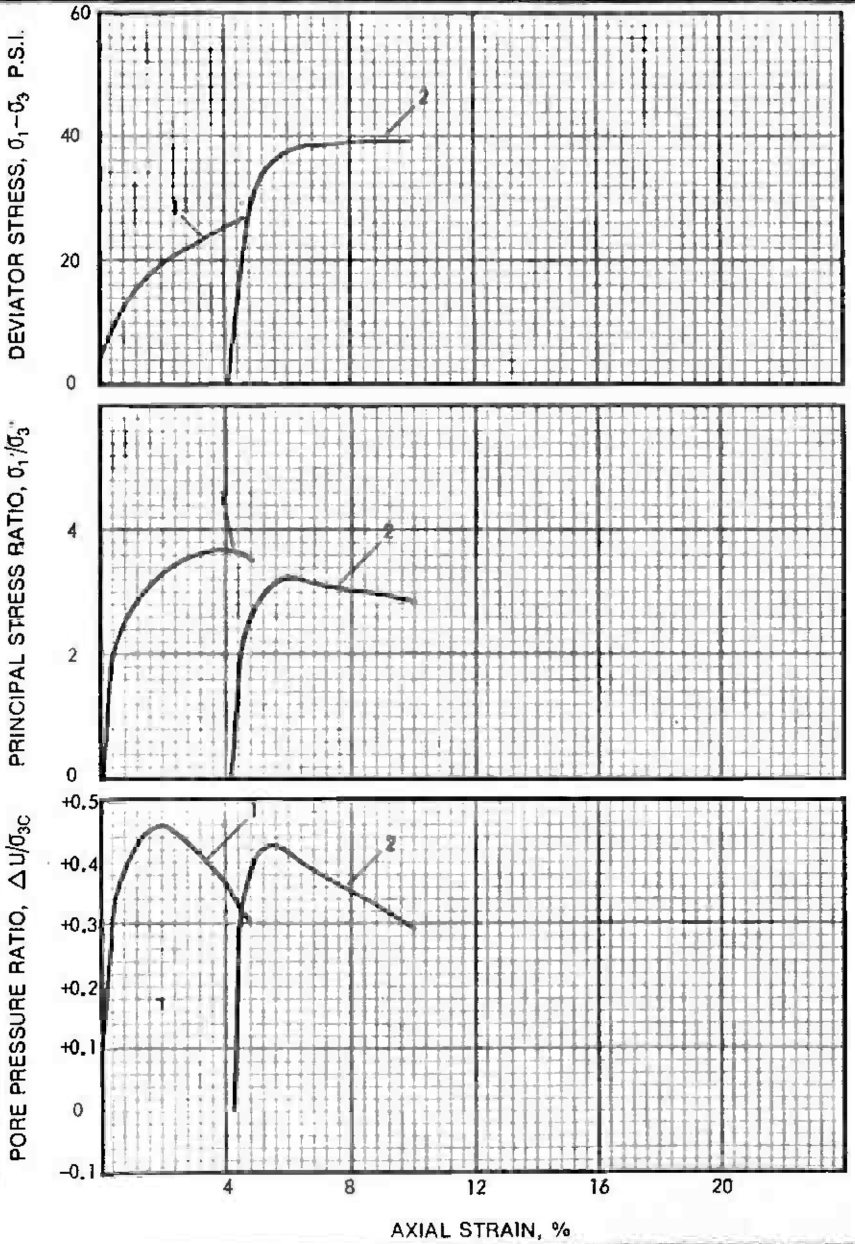
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

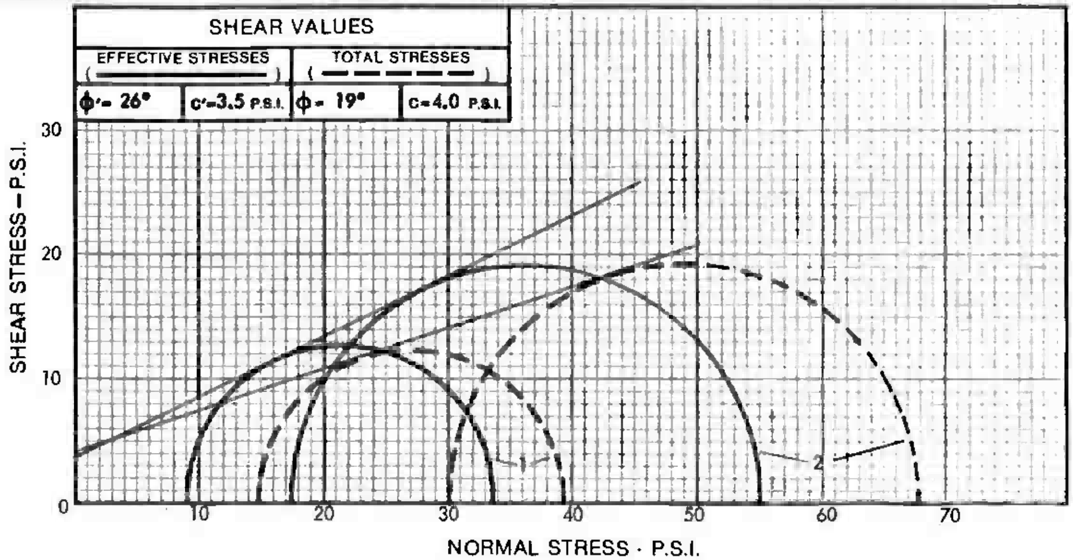
Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. C-16

Approved for publication 4/84 by JAD





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)	
C-9	18/1	C-9	42.5-43.0	ML	5.0	2.42	99.4	15.2	5 Ring Converse

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (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.)	
C-9	1	15	24.4	5.9	9.1	33.5	Tx CUE 2 Stage
C-9	2	30	37.7	12.4	17.6	55.3	Progressive

TRIAXIAL COMPRESSION TESTS

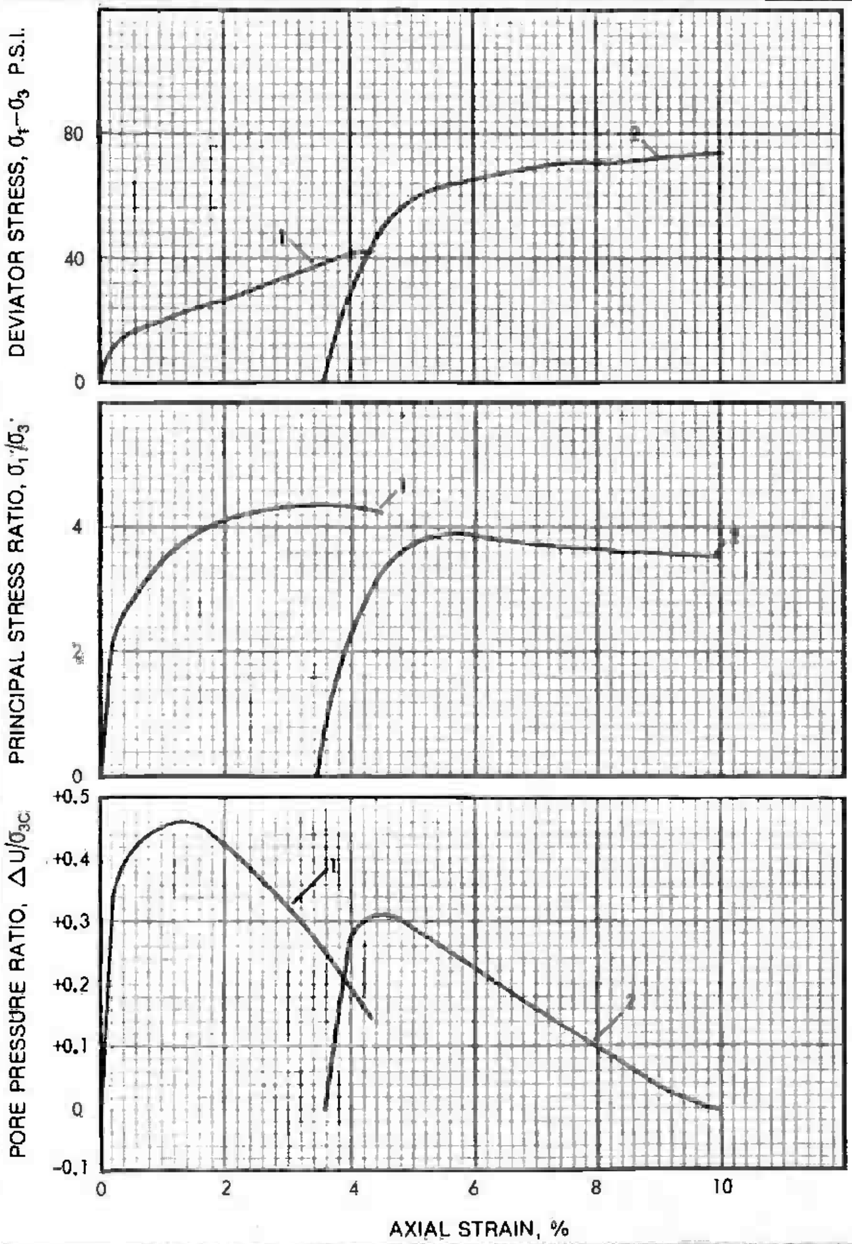
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

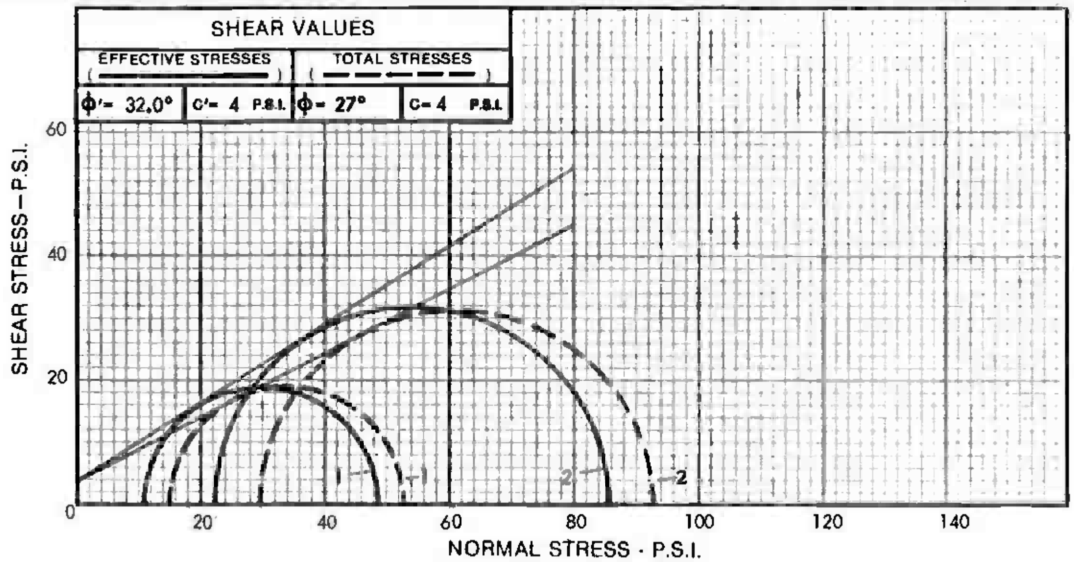
Project No
83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No
C-17

Approved for publication 1/1/81 by JAD





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)	
C-9	18/2	18/2	43.5-44.0	CL	5.0	2.42	106.2	20.7	5 Ring Converse

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (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.)	
C-9	1	15	38.0	3.7	11.3	49.3	Tx CUE Progressive
C-9	2	30	63.5	7.7	22.3	85.8	

TRIAXIAL COMPRESSION TESTS

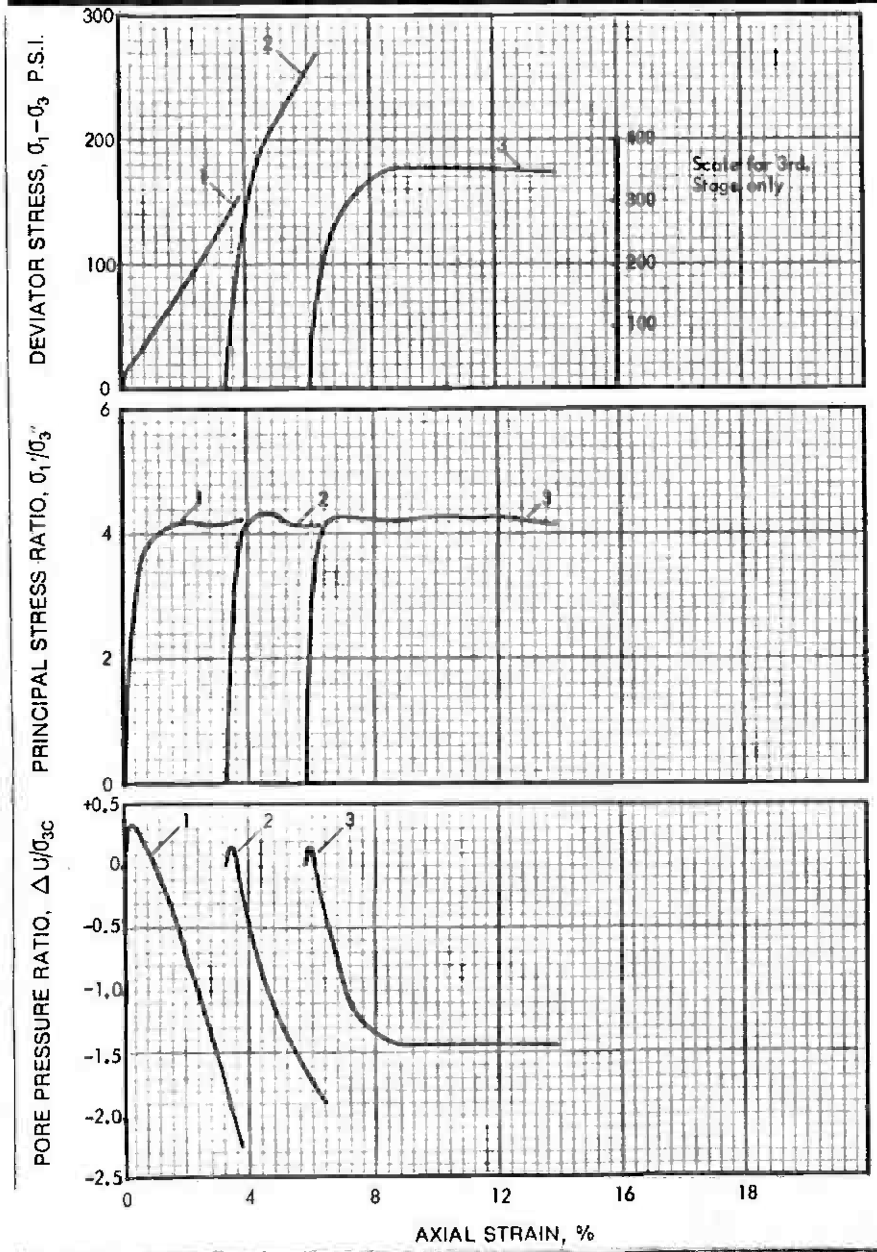
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

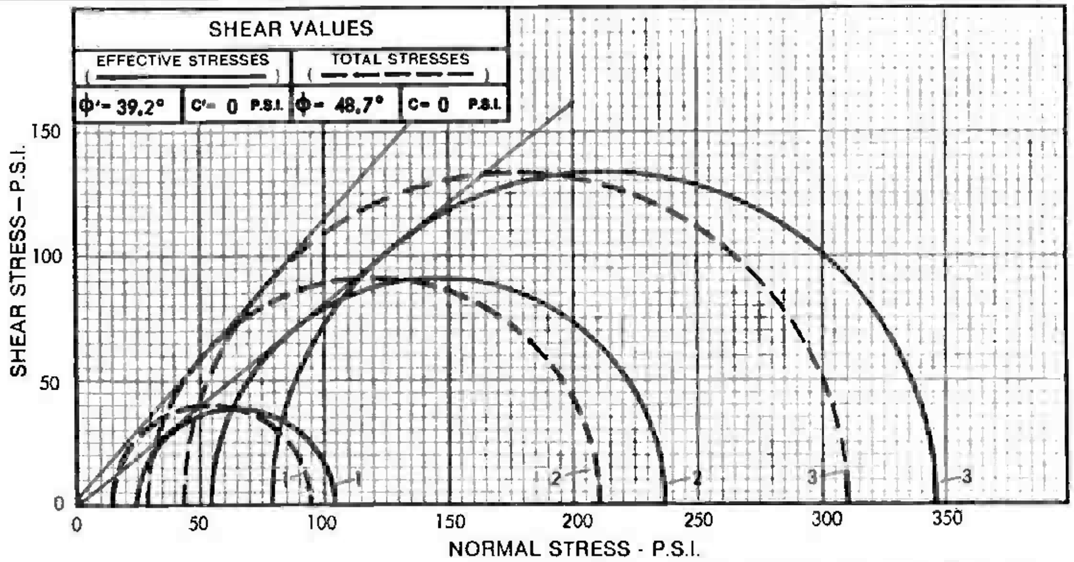
Project No 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No C-18

Approved for publication 4/84 by JAD





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)	
C-14	18/2	C-14	68.2-68.7	SP	5.0	2.42	101	23	5 Ring Converse

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (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.)	
C-14	1	15	81.0	-10.3	25.3	106.3	Tx CUE Progressive
	2	30	181.3	-25.2	55.2	236.5	
	3	45	265.4	-35.5	80.5	345.9	

TRIAXIAL COMPRESSION TESTS

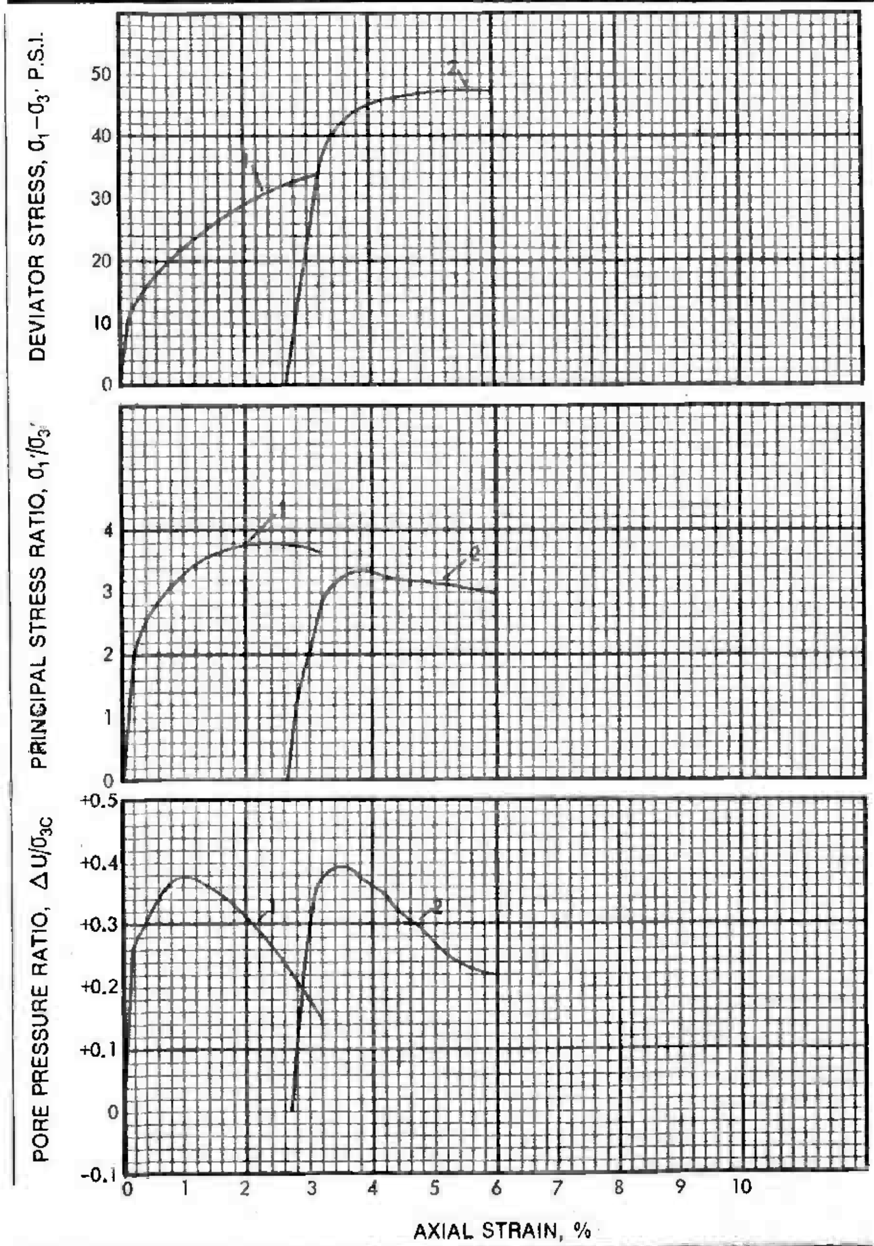
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

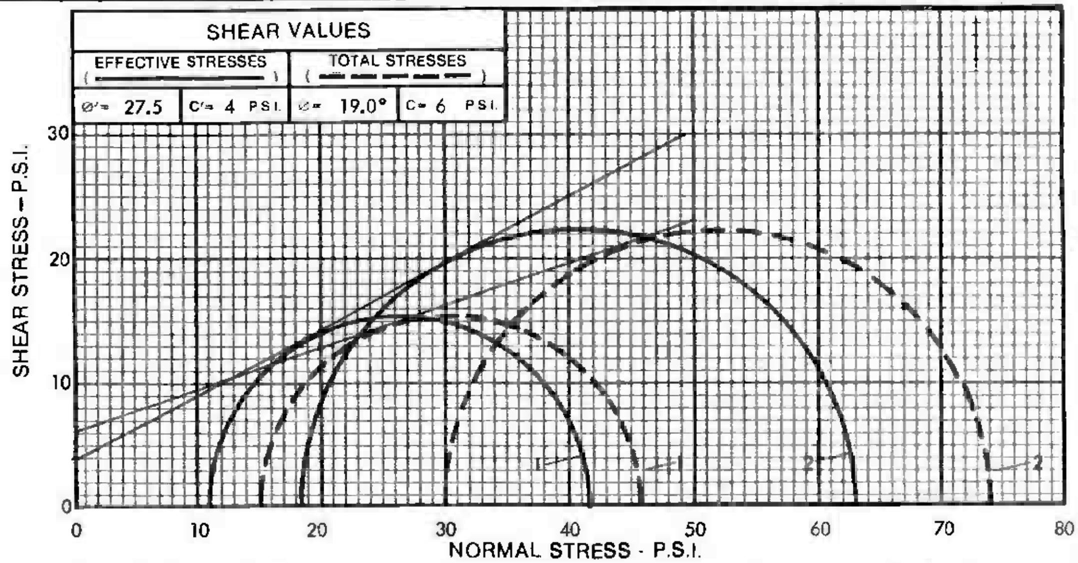
Project No
83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No
C-19

Approved for publication 4/84 by JAD





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)	
C-5	18/3	C-5	22.5-23.0	CL	5.0	2.42	98.9	25.5	5 Ring Converse

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (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.)	
C-5	1	15	30.9	4.0	11.0	41.9	Tx Progressive CUE
C-5	2	30	44.3	11.0	19.0	63.3	

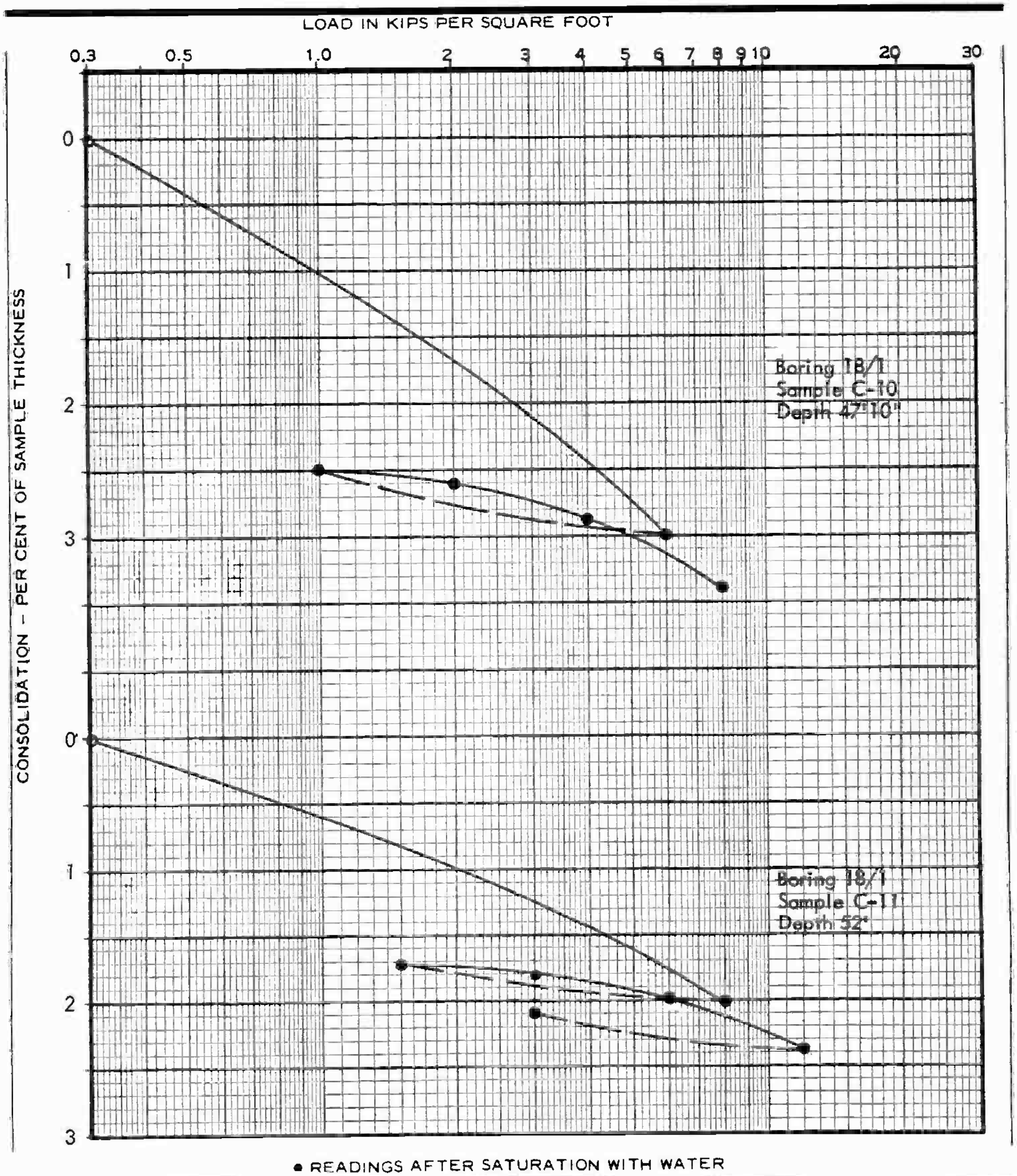
TRIAXIAL COMPRESSION TESTS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No. C-20



Approved for publication 4/84 by FAD

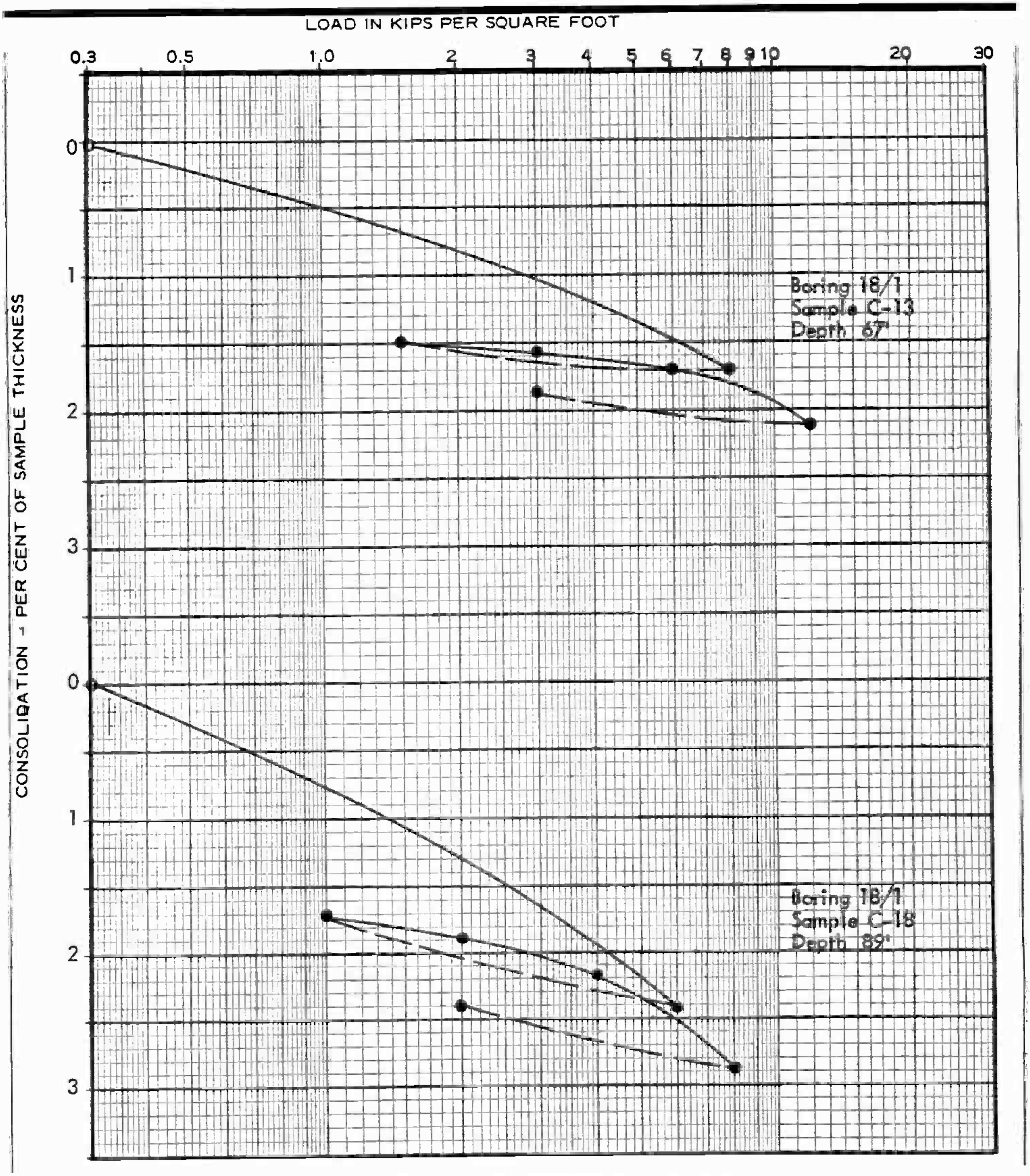
CONSOLIDATION TESTS

DESIGN UNIT A245
Southern California Rapid Transit District
Metro Rail Project

Project No. 83-1140

Figure No. C-21

Converse Consultants
Geotechnical Engineering and Applied Sciences



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

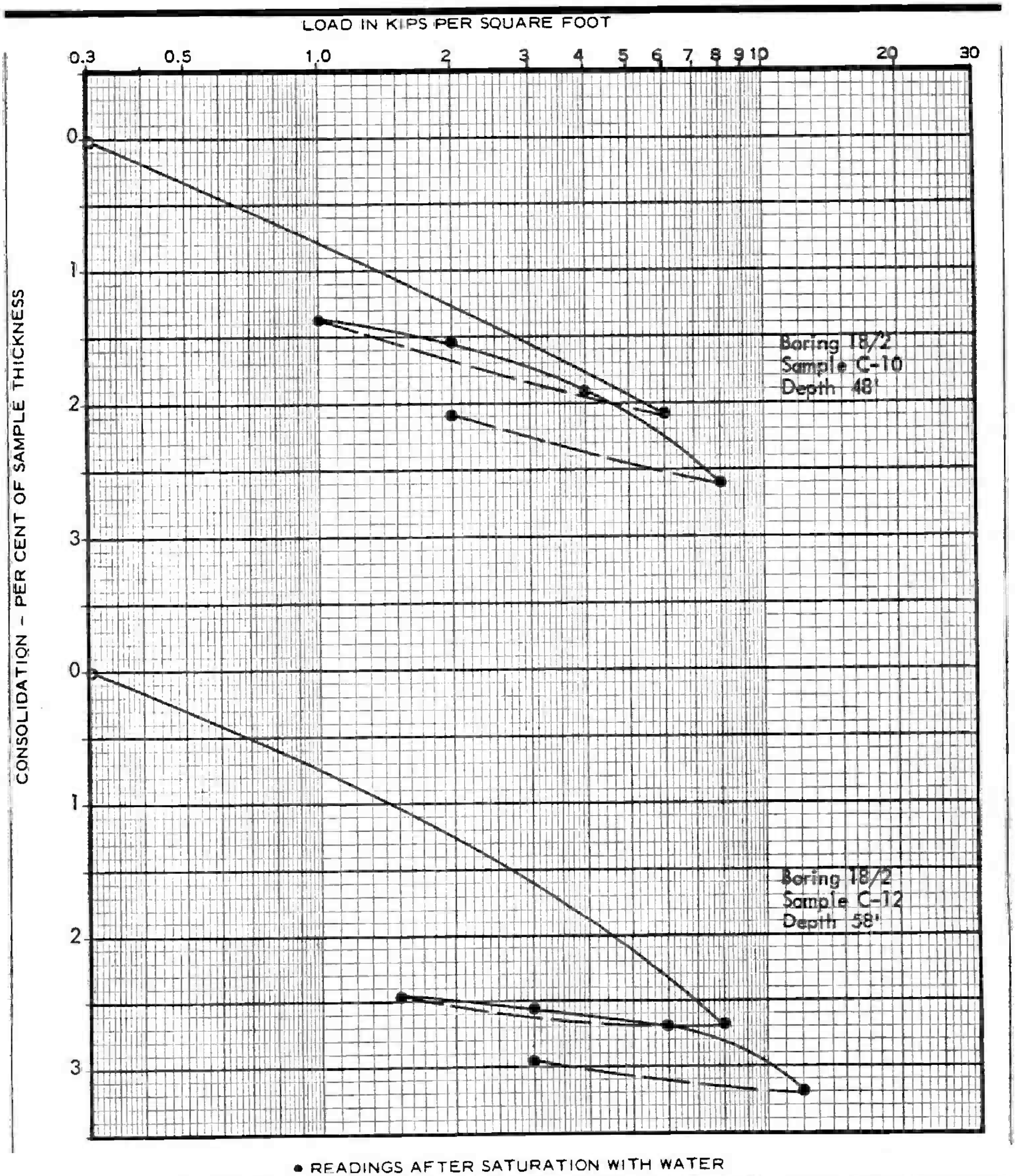
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RIL PROJECT

Project No
83-1140

Figure No.
C-22

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 4/84 by JAD



Approved for publication 4/84 by [signature]

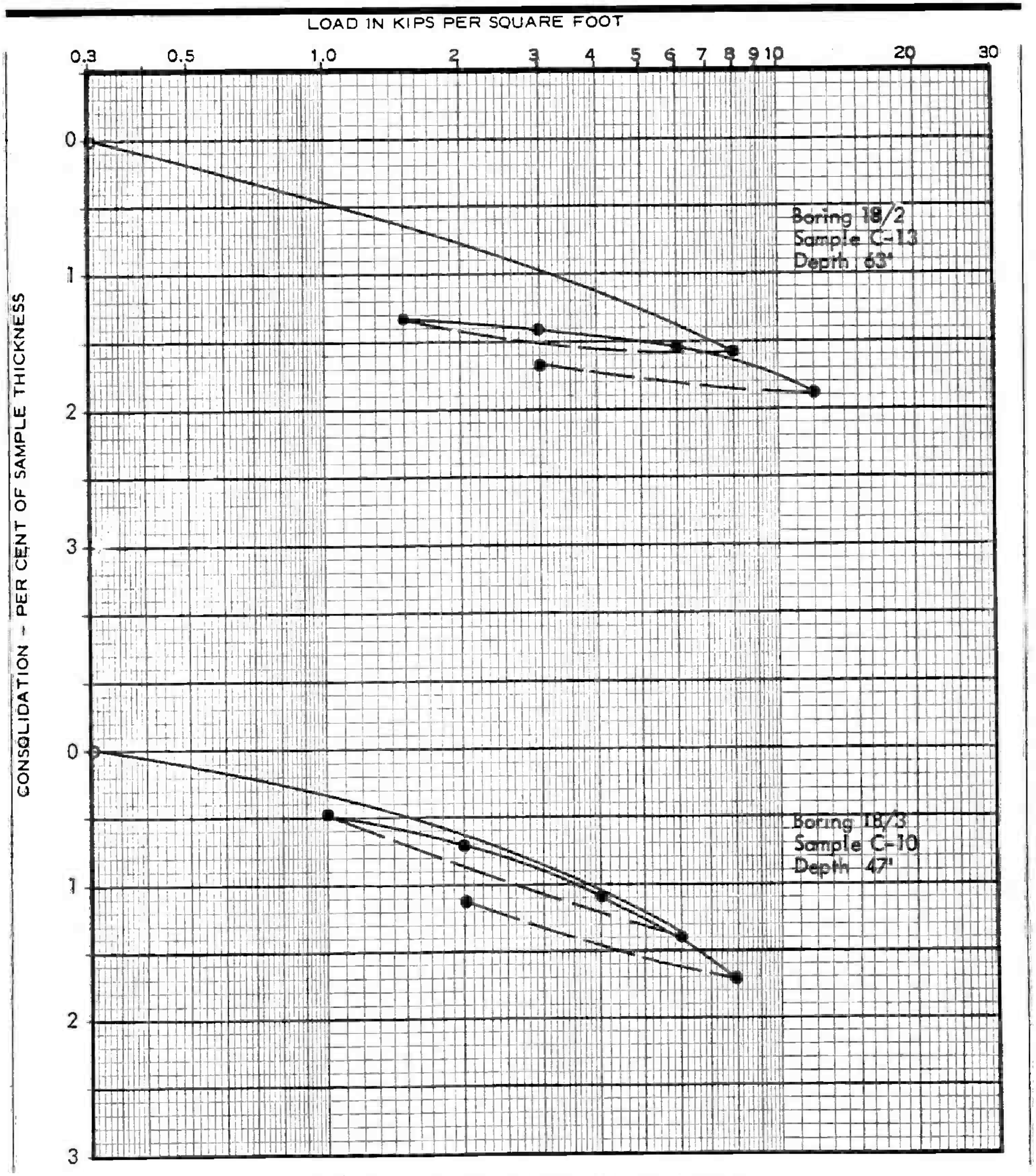
CONSOLIDATION TESTS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No. C-23



● READINGS AFTER SATURATION WITH WATER

Approved for publication 4/84 by [signature]

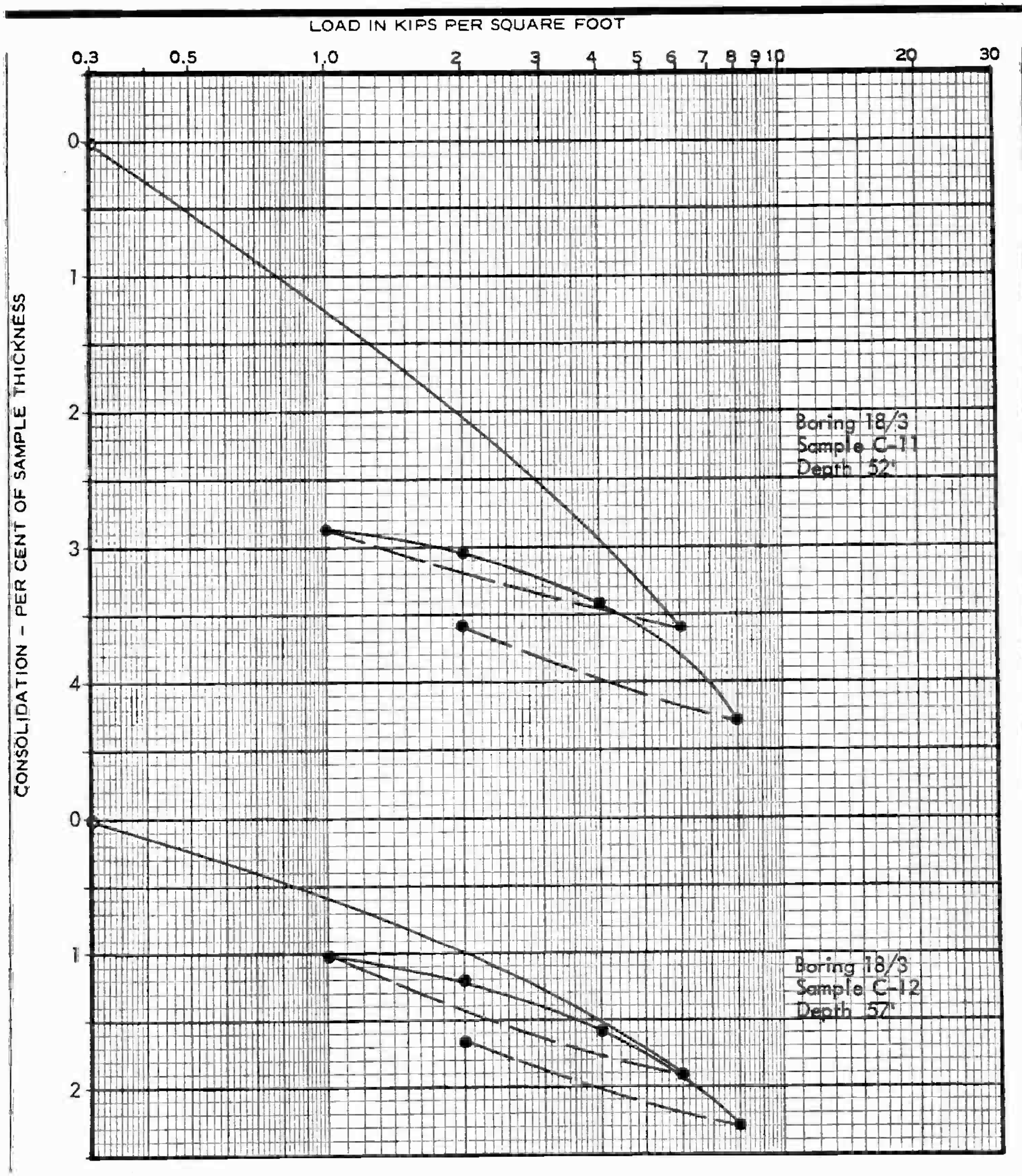
CONSOLIDATION TESTS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-24



• READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

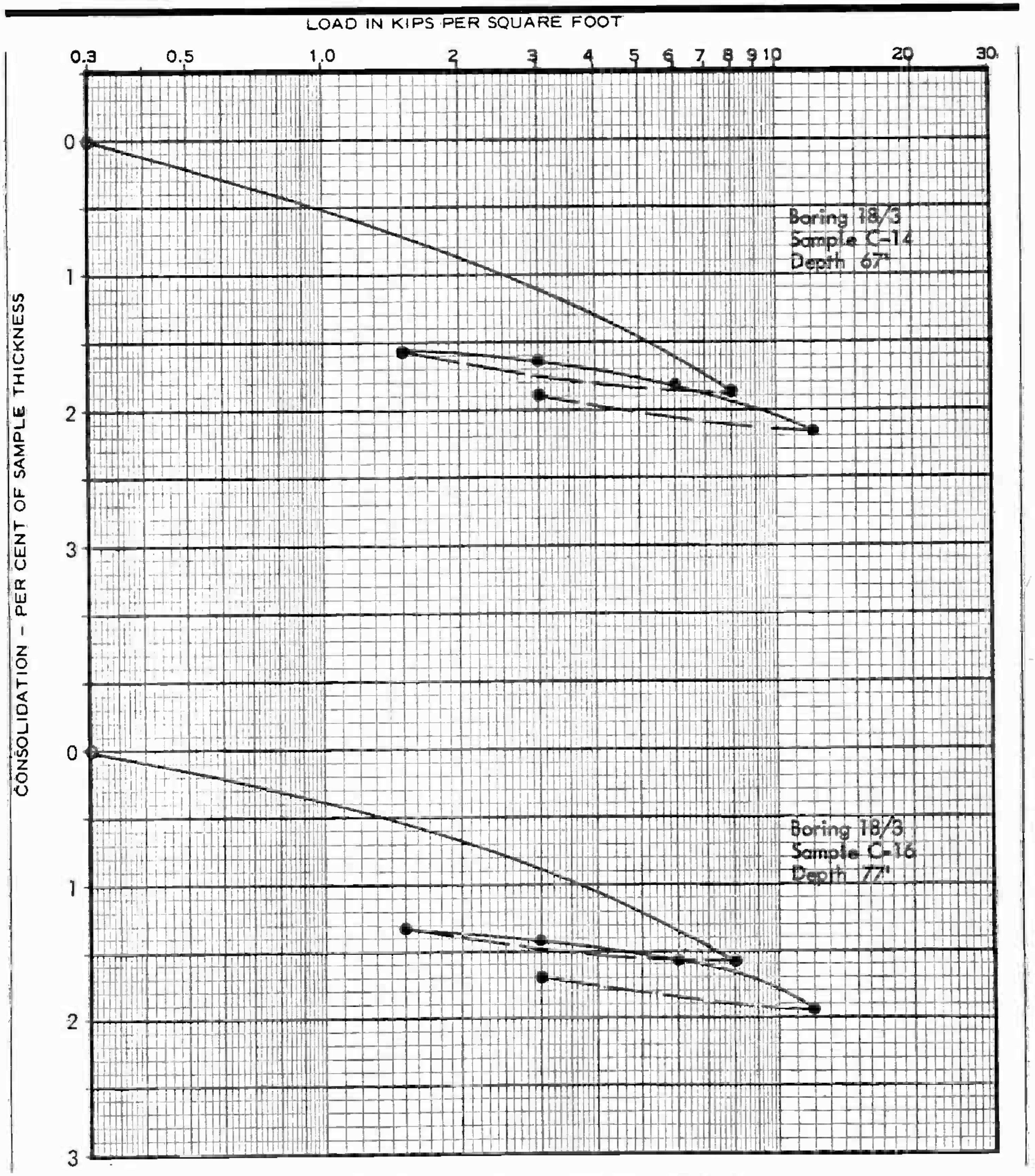
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-25

Approved for publication 9/84 by [signature]



• READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

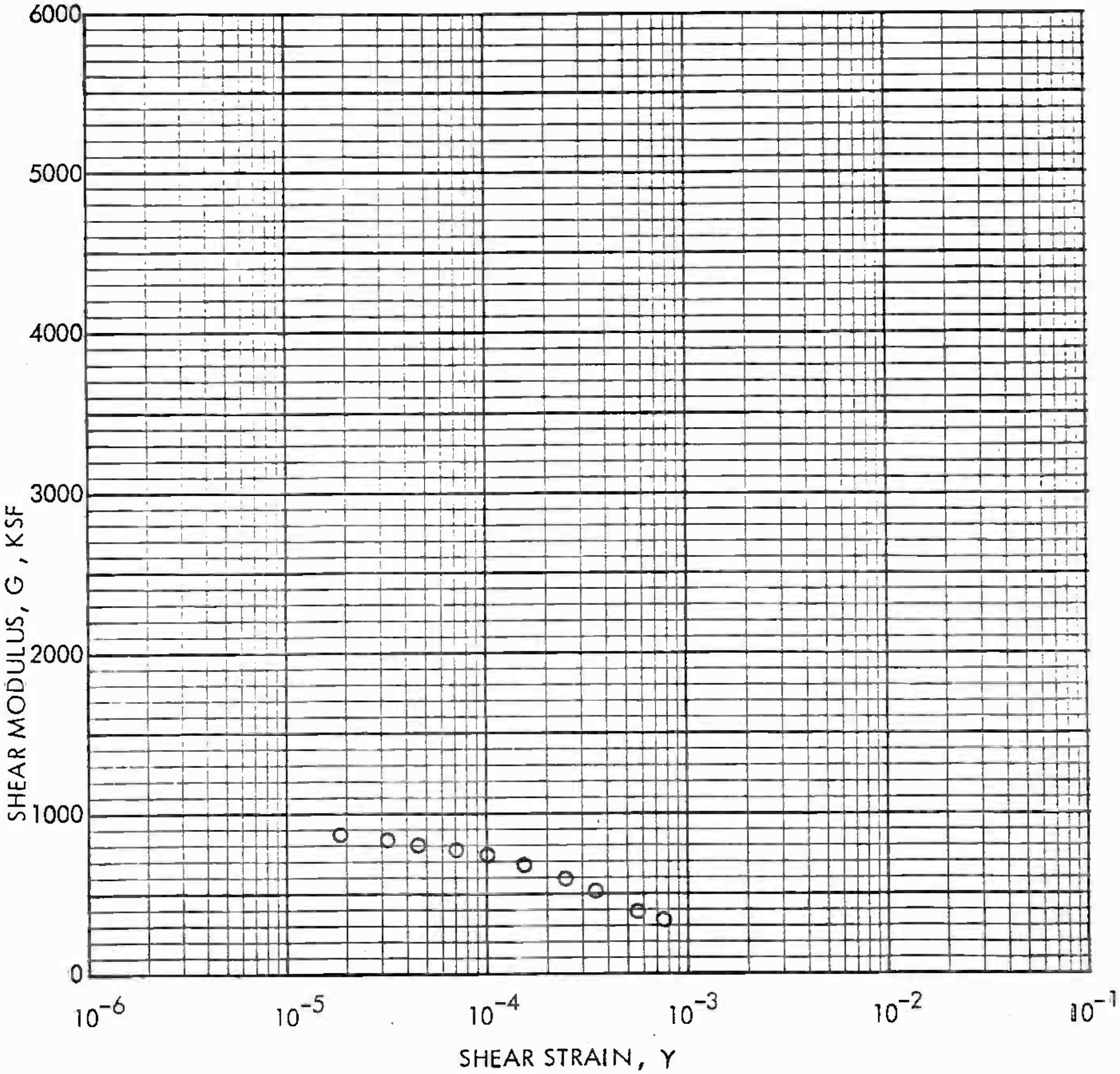
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-26

Approved for publication 4/84 by [signature]

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
18	C-1	19½	97	26	15

Sample Description: Gray Claystone with Calcareous Nodules throughout; moist

Approved for publication 4/5/84 by [signature]

RESONANT COLUMN TEST

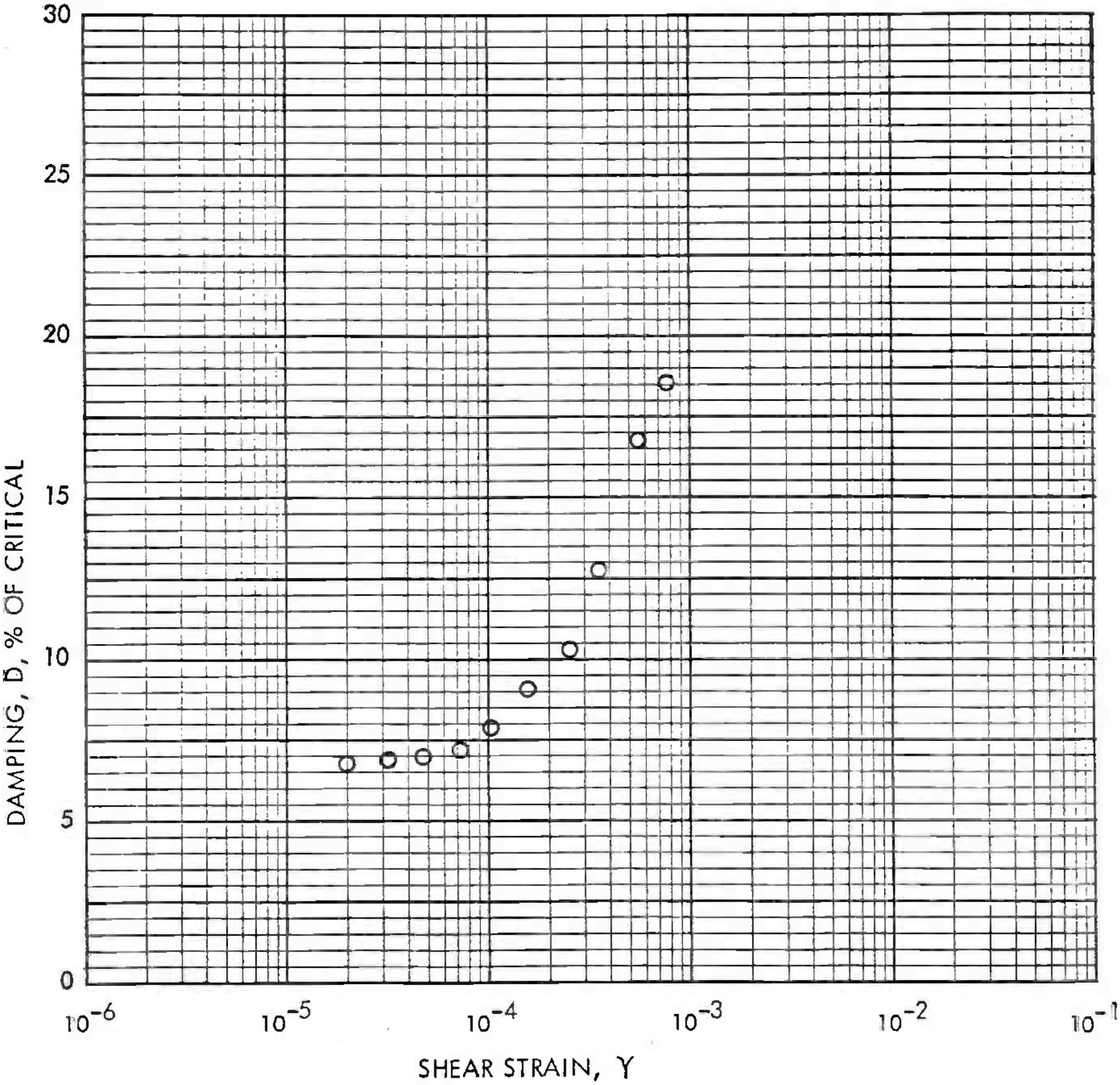
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-27

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
18	C-1	$19\frac{1}{2}$	97	26	15

Sample Description: Gray Claystone with Calcareous Nodules throughout; moist

Approved for publication 4/84 by JHS

RESONANT COLUMN TEST

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geofechnical Engineering and Applied Sciences

Figure No.
C-28

Appendix D

Water Quality Analysis

APPENDIX D WATER QUALITY ANALYSIS

D.1 RESULTS

Water samples were taken from Boring CEG-17B during the 1983 investigation. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for future tunneling activities. The chemical constituents tested are attached.

D.2 FIELD PROGRAM

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

GENERAL MINERAL ANALYSIS*

BROWN AND CALDWELL

CONSULTING ENGINEERS
ANALYTICAL SERVICES DIVISION
373 SOUTH FAIR OAKS AVE.
PASADENA, CA 91105
PHONE (213) 795-7553

RECEIVED
DEC 8 1983
CONVERSE CONSULTANTS

Log No. P83-11-056
Date Sampled 10-27-83
Date Received 11-04-83
Date Reported 12-07-83

Reported To: Converse Consultants

cc.

Laboratory Director

Sample Description 83-1140-71, BH 17B Wilshire / Orange Dr. A220

Anions	Miligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligr per li
Nitrate Nitrogen (as NO_3)	20	0.32	Hydroxide Alkalinity (as $CaCO_3$)	-0-		
Chloride	140	3.92	Carbonate Alkalinity (as $CaCO_3$)	-0-		
Sulfate (as SO_4)	70	1.47	Bicarbonate Alkalinity (as $CaCO_3$)	320		
Bicarbonate (as HCO_3)	400	6.40	Calcium Hardness (as $CaCO_3$)	230		
Carbonate (as CO_3)	-0-	-0-	Magnesium Hardness (as $CaCO_3$)	160		
Total Milliequivalents per Liter		12.11	Total Hardness (as $CaCO_3$)	390		
Cations	**Milligrams per liter**	**Milliequiv. per liter**	Iron	< 0.09		
Sodium	82	3.53	Manganese	< 0.04		
Potassium	0.8	0.02	Copper	< 0.07		
Calcium	91	4.55	Zinc	< 0.015		
Magnesium	38	3.12	Foaming Agents (MBAS)	< 0.1		
Total Milliequivalents per Liter		11.22	Dissolved Residue, Evaporated @ 180°C	670		
			Specific Conductance, micromhos @ 25°C	1200	pH	7

*Conforms to Title 22, California Administrative Code (California Domestic Water Quality and Monitoring Regulations)

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:

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

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

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.

In general, the SPT blow count measurements in the San Pedro Sands were greater than 100 blows per foot, indicating that these soils are generally very dense. These blow counts along with the relationship shown in Figure E-1 suggest that liquefaction of the San Pedro Sands would be very unlikely during ground shaking from the maximum design earthquake. Corrected SPT "N" values (normalized to 2 ksf overburden pressure for 12 SPT tests in saturated granular alluvium ranged from 29 to 60 with an average of about 45. Determination of dynamic strength was based on an M7.0 maximum design earthquake event. The liquefaction analysis based on Seed et al (1983) indicated the granular soils could withstand ground acceleration up to 0.6g before initial liquefaction. Therefore, the granular alluvium is considered to have a low liquefaction potential.

E.3.2 Shear Wave Velocity Measurements

Crosshole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project tunnel alignment were performed as part of the initial 1981 geotechnical investigation. One of the crosshole surveys was performed at Borings CEG-18 near the Wilshire/La Brea Station site. Shear wave velocities measured in the Alluvium (approximately the upper 30 feet of the borehole) range between 690±50 fps to 1200±100 fps for the crosshole measurements and 1230±90 fps for the downhole measurements.

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

E.3.3 Gradation/Plasticity Characteristics

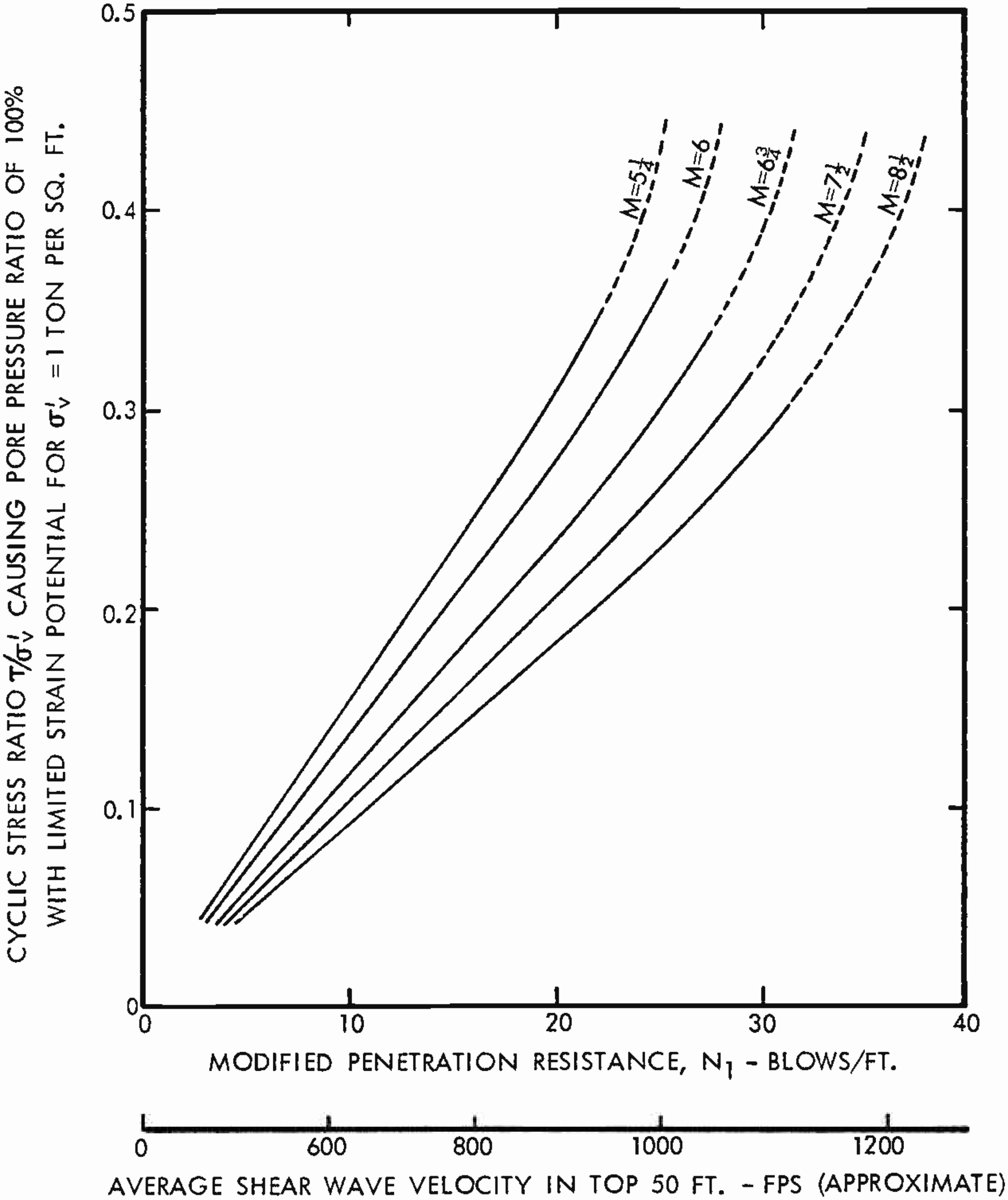
Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction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

The gradational characteristics of the various soils which comprise the onsite Alluvium were compiled from laboratory tests performed during this and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" soils shown in Figure E-2 are presented in Figure E-3. Figure E-3 indicates that several samples tested fall within the range of gradations of soils considered more "susceptible" to liquefaction. However, there are many factors other than gradational characteristics which affect the liquefaction potential of a particular soil, one of the most important being the soil density. The SPT blow counts discussed in E.3.1 indicate that alluvial soils are dense and, therefore, would have a low liquefaction potential.

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

E.3.4 Conclusions

Based on the above considerations and comparisons, it is our judgement that the 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 sufficiently high SPT blow counts of the granular soils and the clay content and clay characteristics of the fine-grained soils. The San Pedro Sands would have low liquefaction potential for similar ground shaking due to sufficiently high SPT blow counts.



(after Seed, 1983)

Approved for publication 4/84 by [signature]

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

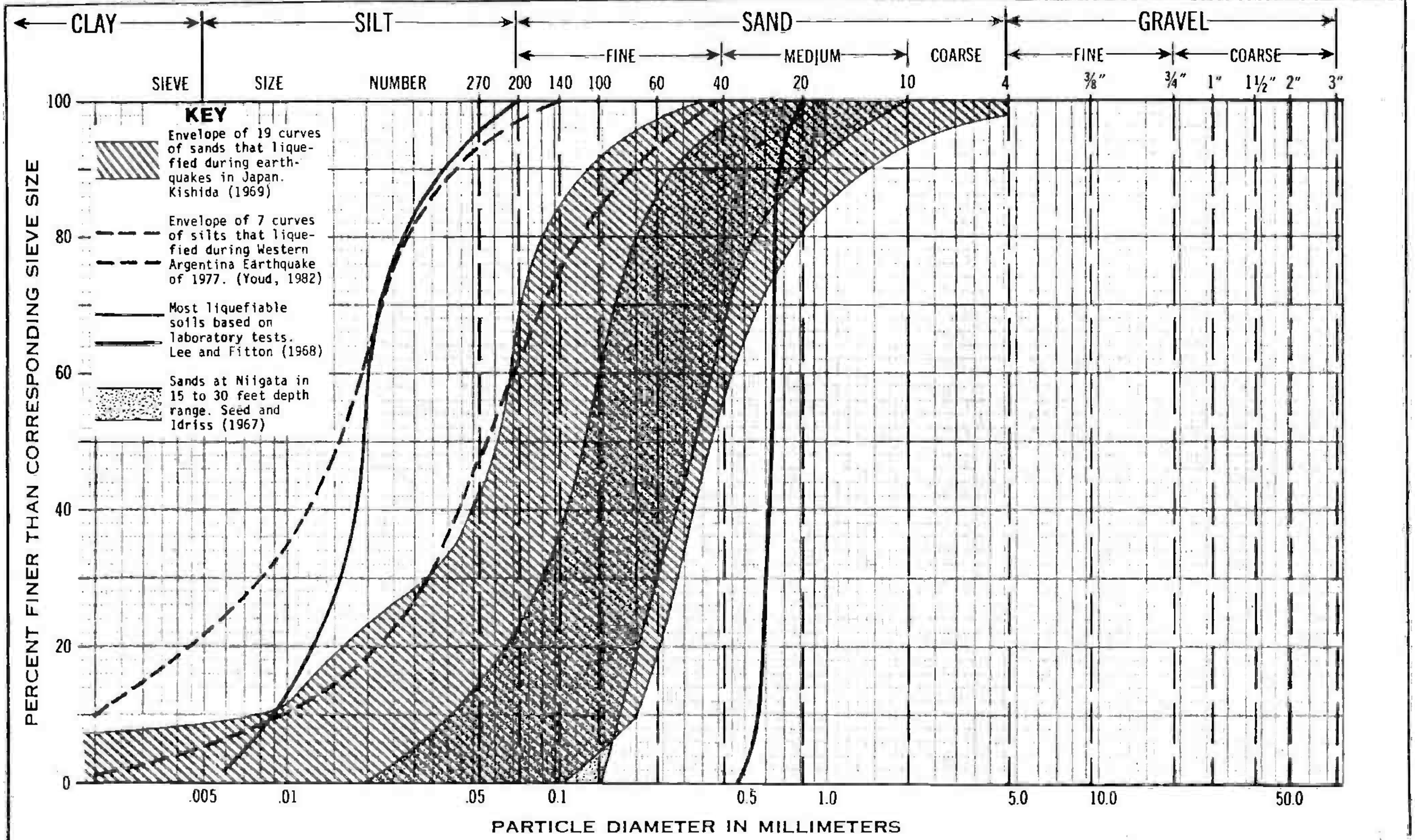
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
E-1

FORM NO. D-3-10 Approved for publication 4/84 by JAD



GRADATIONS OF SOILS SUSCEPTIBLE TO LIQUEFACTION

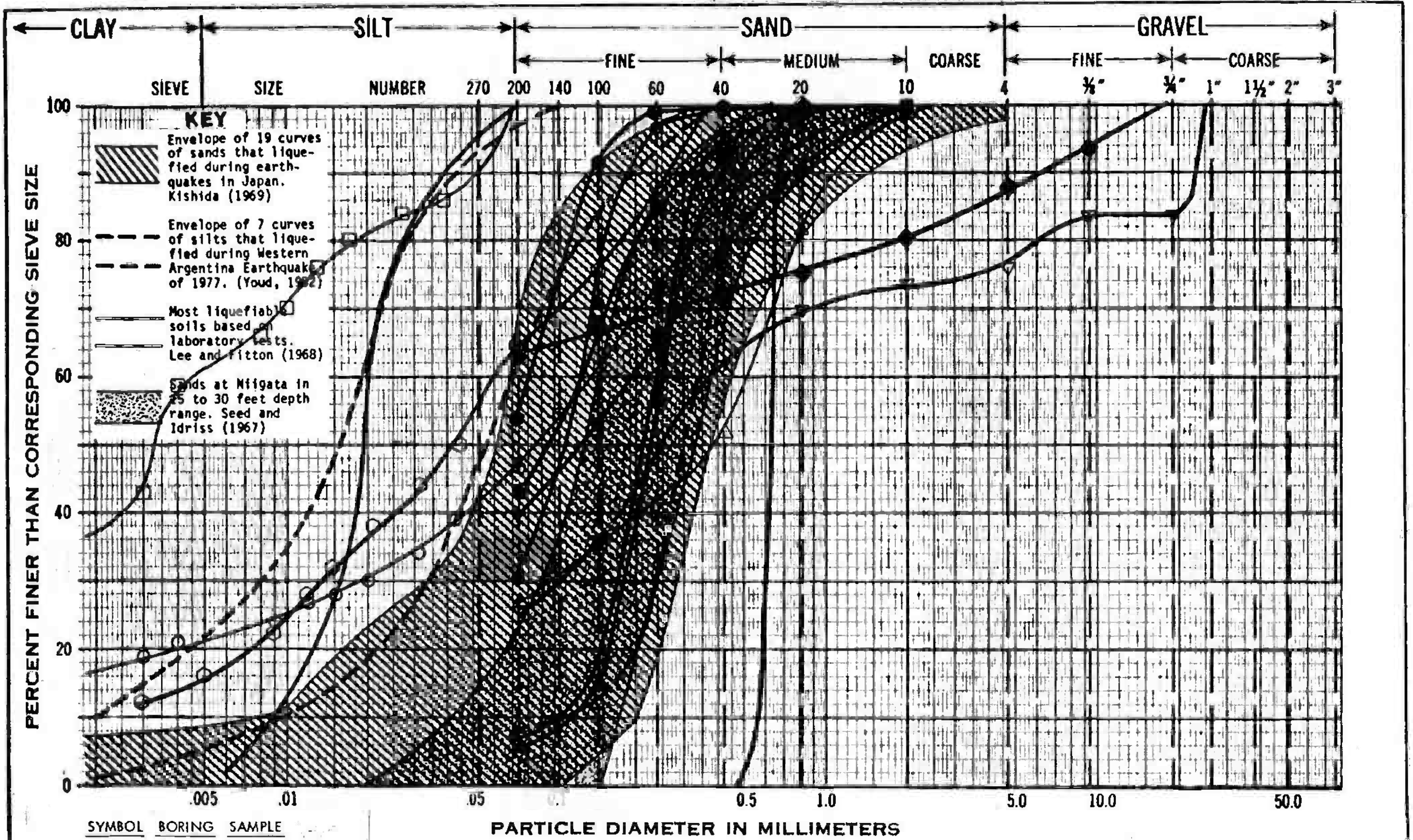
DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No.
E-2

FORM NO. D-3-10/ pproved for publication ________ by ________



SYMBOL	BORING	SAMPLE
	18-1	C-5
	18-1	C-8
	18-1	C-9
	18-1	C-10
	18-1	C-12
	18-2	C-8
	18-2	C-9
	18-2	C-11
	18-2	C-14
	18-3	C-5
	18-3	C-6
	18-3	C-8
	18-3	C-13

GRAIN-SIZE DISTRIBUTION CHART

DESIGN UNIT A245
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants
Geotechnical Engineering and Applied Sciences

Figure No.
E-3

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.

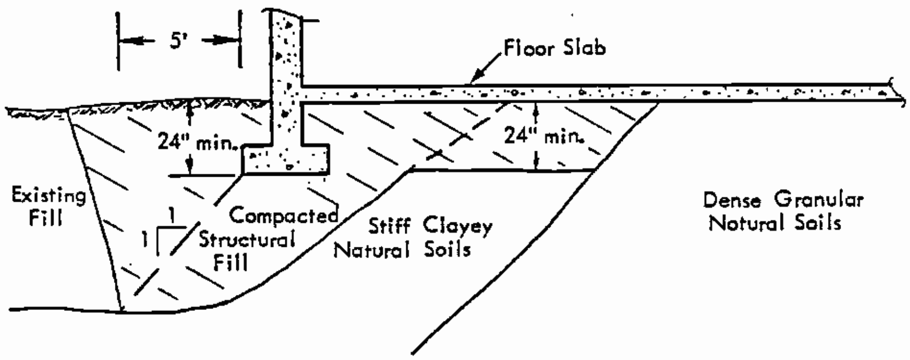
- Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed 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 at the subterranean levels may be supported directly on undisturbed dense materials. The subgrade should be proof rolled to detect soft or disturbed areas, and such areas should be excavated and replaced with structural fill. If existing fill soils are encountered in near surface subgrade areas, these materials should be excavated and replaced with properly compacted granular fill. Where clayey natural soils (near existing grade) are exposed in the subgrade, these soils should be excavated to a depth of 24 inches below the subgrade level and replaced with properly compacted granular fill. Where dense natural granular soils are exposed at slab subgrade, the slab may be supported directly on these soils. All structural fill for support of slabs or mats should be placed and compacted as recommended under "Structural Fill and Backfill".

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

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

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

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

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

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

Appendix G

Geotechnical Reports References

APPENDIX G GEOTECHNICAL REPORTS REFERENCES

REPORT No.	REPORT DATE	LOCATION	CONSULTANT
23	04/14/47	Block bounded by Wilshire, Mansfield, Carling and Citrus	L.T. Evans
24	03/04/47	Northeast corner Wilshire & Curson	L.T. Evans
25	04/22/47	Northeast corner Wilshire & Sierra Bonita	L.T. Evans
26	10/27/69	Block bounded by Wilshire, Masselin, Eighth and Curson	L.T. Evans