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Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A425

BY

CONVERSE CONSULTANTS, INC.
EARTH SCIENCES ASSOCIATES
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May 15, 1984

Metro Rail Transit Consultants
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Attention: Mr. B.I. Maduke, Senior Geotechnical Engineer

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A425 prepared in accordance with our Contract No. 503 agreement dated September 30, 1983 between Converse Consultants, Inc. and Metro Rail Transit Consultants (MRTC). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A425.

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

Respectfully submitted,

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

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

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for Design Unit A425 which includes the proposed Universal City Station. The proposed cut-and-cover structure at the Station site will be about 560 feet long, 60 feet wide, and will require excavating some 80 to 84 feet below the existing ground surface at the Station site. The purpose of the investigation is to provide geotechnical information and recommendations to be used by design firms in preparing designs for the project. Although this report may be used for construction purposes, it is not intended to provide all of the information that may be required to construct the project.

The subsurface profile at the Station site consists of a thin pavement section which overlies generally fine-grained Alluvium that extends to depths of about 43 to 58 feet. Beneath the fine-grained Alluvium lies a relatively continuous layer of coarser-grained Alluvium which varies in thickness from about 2 to 16 feet. Underlying the coarse-grained Alluvium is the Topanga Formation bedrock. Groundwater was encountered within the Alluvium. An interpretation of the available groundwater data indicates that groundwater is about 16 feet below the existing ground surface at the south end and 23 feet below the existing ground surface at the north end of the Station site.

Construction of the Station will involve making a 80- to 84-foot deep excavation through the Alluvium and into the Topanga Formation bedrock. This will involve shoring and dewatering. The permanent structure will in essence be a concrete box bearing on the Topanga Formation bedrock and retaining Alluvium deposits.

The primary geotechnical evaluations and design criteria presented in this report include:

o EXCAVATION DEWATERING AND SUBSIDENCE: Since the excavation will extend through and below the groundwater table, a dewatering system will be required to construct the proposed excavation. Dewatering of the excavation will result in some areal subsidence. The contractor will be responsible for designing, installing and operating a suitable dewatering system. The report presents groundwater data results of a pump test performed in the vicinity of the site and general dewatering criteria to be satisfied by the contractor.

o UNDERPINNING: Most of the structures in the immediate vicinity of the Station site will be demolished for construction of the new aboveground facilities. Therefore, there does not appear to be a need for underpinning at the site.

o TEMPORARY EXCAVATION SUPPORT: The excavation system will be chosen and designed by the contractor in accordance with specified criteria and subject to review and acceptance by the Metro Rail Transit Consultants. There are several ways to construct the excavation including a conventional shoring system with underpinning, or a conservatively designed shoring system which would eliminate or minimize the need to underpin. In addition, a "tight" shoring system could eliminate the need for underpinning and site dewatering. Design criteria for various types of soldier pile shoring systems are presented in the report since these have been used successfully in the Los Angeles area in similar soil conditions. Other systems may also be appropriate and may be considered by the contractor.

- EXCAVATION INSTRUMENTATION PROGRAM: The proposed excavation should be instrumented. The recommended instrumentation program includes a preconstruction survey, surface survey control, heave monitoring, tiltmeters and inclinometer measurements, and bracing load measurements.

- ENGINEERING MATERIAL PROPERTIES: Site specific static and dynamic properties for the various materials encountered in Design Unit A425 are presented in Tables 5-2 and 6-2 of this report.

- PERMANENT FOUNDATION SYSTEM: The Station structure can be adequately supported on the underlying materials. The report presents allowable bearing pressures, pile capacities and estimates of foundation elastic heave and elastic settlement.

- LOADS ON PERMANENT SLABS AND WALLS: The report presents recommended lateral design earth pressures on the permanent structures. These include hydrostatic uplift pressures on the bottom slab.

- LIQUEFACTION POTENTIAL: The liquefaction potential of the sandy soils contained within the alluvial deposits at the Station site was evaluated using comparisons of various soil properties with those of materials which have undergone liquefaction or loss of strength during past earthquakes. The gradational characteristics, shear wave velocity, and Standard Penetration Test blow count measurements taken within the soil deposit were compared to published case histories. On this basis, it was established that some of the soils at the site have a high potential for liquefaction and may experience a severe loss in strength during or after the maximum design earthquake. Since the base of the Station structure is founded within the Topanga Formation bedrock, it should perform satisfactorily during the maximum design earthquake. However, significant increases in lateral earth pressures could develop on the walls of buried structures due to liquefaction and/or loss of strength within zones of the fine-grained Alluvium. In addition, some seismic compaction of the alluvium could occur due to dissipation of excess pore pressures after an earthquake which could result in differential settlement of shallow surface structures founded on these materials. The effects of liquefaction and loss of strength within portions of the fine-grained alluvium should be considered in the design of the permanent structures at the Universal City Station.

- SEISMIC CONSIDERATIONS: 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.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A425. The subject design unit includes the proposed Universal City Station. This structure will be part of the proposed 18.6-mile long Metro Rail Project (see Drawing 1, Vicinity Map). The purpose of the investigation is to provide geotechnical information to be used by the design firms in preparing designs for the project. Although this report may be used for construction purposes, it is not intended to provide all the geotechnical information that may be required to construct the project. The work performed for this study included field reconnaissance, drilling and logging of exploratory borings, geologic interpretation, field and laboratory testing, engineering analyses, and development of recommendations.

Additional geotechnical information on the Metro Rail Project is included in the following reports, some of which may pertain to Design Unit A425:

- "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 SCRTD in November 1981: This report presents general geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experiences and practices in the Los Angeles area.

- "Geotechnical Report, Metro Rail Project, Design Unit A430," prepared by Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in May 1984. This report presents the results of our findings for about two miles of subsurface track line proceeding south to north from the north end of the Universal City Station to the south end of the North Hollywood Station.

- "Seismological Investigation & Design Criteria, Metro Rail Project," prepared by Converse Consultants, Lindvall, Richter & Associates, Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD 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 boring data in the general vicinity of the proposed Metro Rail Project.

- "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.

- "Report of Supplementary Alignment Rotary Borings, Metro Rail Project, Contract No. 2256-2," prepared by Converse Consultants, Inc., submitted to SCRTD in September 1983. This report presents the soil, rock, and groundwater conditions encountered in 10 supplementary rotary wash borings drilled along the Metro Rail Project alignment. Results of laboratory tests performed on selected soil and rock samples are also summarized in the report.

- "Report of Man-Size Auger Boring, Metro Rail Project, Contract No. 2256-2," prepared by Converse Consultants, Inc., submitted to SCRTD in August 1983. This report presents the soil, rock, oil/gas, groundwater, and other subsurface conditions encountered in 10 large-diameter or man-sized auger holes drilled at various locations along the Metro Rail Project alignment. Results of water quality analyses are also presented.

Pertinent data from these reports have been incorporated in this report.

The design concepts discussed in this geotechnical report are based on the "General Plans, CBD to North Hollywood, Contract No. A425, Universal City Station," Sheets 1 to 14 of 20, dated July 1983, and "Report for the Development of Milestone 10: Fixed Facilities," dated September 1983 and revised plans A-63 through A-66. These documents were prepared by SCRTD.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

3.1 SITE DESCRIPTION

The proposed Universal City Station, as shown on Drawings Nos. 2 and 3, is aligned southwest to northeast. It will be located off-street in an area bounded by Lankershim Boulevard on the east, Universal Place on the south, and Bluffside Drive on the west and north. The ground surface elevation varies across the site and is at approximately Elevation 579 on the south end and Elevation 573 on the north end of the Station site.

MCA Headquarters and Universal Studios are located immediately to the east. Areas to the west are either residential or parkland. Within the Station site is the Campo de Cahuenga--a historical landmark park. The Hewlett Packard Company, which currently occupies a facility in the Station area, is relocating to new facilities in the near future. A 36-story, 700,000-square foot office building, which will be the headquarters for the Getty Oil Corporation, is under construction on the east side of Lankershim adjacent to the Hollywood Freeway. Except for the Campo de Cahuenga, the existing structures at the Station site will be demolished.

3.2 PROPOSED STATION STRUCTURE

One entrance is planned for this station and will be oriented toward Lankershim Boulevard. It will serve both parking area and pedestrian arrivals and will lead to a single mezzanine located in the center of the station. Ancillary space will be provided at each end of the Station with a traction power substation located below grade over the ancillary space at the south end of the Station.

The proposed main Station area will consist of a reinforced concrete structure about 560 feet long and 60 feet wide (outside wall dimensions). The ground surface varies from Elevation 579 feet at the south end of the Station to Elevation 573 feet at the north end. The top of rail varies between about Elevation 504 and 503 feet. The depths of excavation for the Station structure will range from 84 feet below the existing ground surface at the south end to a depth of 80 feet at the north end. After the Station is constructed, between 10 and 35 feet of fill will be placed above the Station box structure.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The information presented in this report is based primarily upon field and laboratory investigations carried out in 1981 and 1983. This information was derived from field reconnaissance, borings, geologic reports and maps, groundwater measurements, field geophysical surveys, groundwater quality tests, and laboratory tests on soil and rock samples.

4.2 BORINGS

A total of 10 exploratory boreholes have been drilled at or in relatively close proximity to, the proposed Station structure of Design Unit A425. Of the 10 borings, 9 are rotary wash type borings and 1 is a large-diameter or "man-size" auger hole. One rotary-wash boring was drilled as part of the 1981 geotechnical investigation, 3 supplementary borings were drilled in January 1983, and 5 borings were drilled for this investigation during October and November of 1983. The large-diameter borehole was also drilled in January 1983.

Locations of all the borings used in the interpretation of the subsurface conditions present at the proposed Universal City Station site are shown in Drawings 2 and 4. A detailed description of the field procedures employed in logging the boreholes as well as the edited field logs of all the borings are included in Appendix A.

Groundwater observation wells (piezometers) were installed in 5 of the borings drilled at or near the Station site. Free water was also observed in the large-diameter borehole. A summary of the groundwater levels measured in the piezometers installed at or near the site, in addition to those observed in the large-diameter borehole, is presented in Section 5.4.

4.3 GEOPHYSICAL MEASUREMENTS

Downhole and crosshole compression and shear wave velocity surveys were made in Borehole CEG-34 during the 1981 geotechnical investigation. This boring is about 1300 feet northwest of the proposed Universal City Station site.

The downhole survey was conducted down to a depth of about 200 feet and the crosshole survey was performed in a borehole array down to a depth of about 100 feet. The results of the downhole and crosshole surveys are summarized in Appendix B in addition to a discussion of the procedures employed in the field to perform these surveys.

4.4 OIL AND GAS ANALYSES

No strong natural gas odors were detected during the drilling and logging of the borings located at or near the Station site. A sulfur odor was noted at a depth of 48 feet in Boring 34-5. Oil slicks appeared on the drilling fluid during the drilling and logging of Borings 34-3, 34-4, 34-5, and 34A. The appearance of this oil suggests that the bedrock is probably slightly petroliferous at or in the vicinity of the Station site.

Some organic type odors were detected in the large-diameter borehole and several of the rotary-wash borings. However, these odors have been attributed to the decay of roots and wood fragments in the Alluvial soils (see Appendix A and Section 5.5).

4.5 WATER QUALITY ANALYSES

Chemical analyses have not been performed on any water samples obtained from the site. Water samples obtained from two boreholes located about 2000 and 3000 feet from the Universal City Station site were tested during the 1981 geotechnical investigation. Results of these tests are reported in Section 5.4 and Appendix D..

4.6 GEOTECHNICAL LABORATORY TESTING

A laboratory testing program was performed on representative soil and rock samples. These consisted of classification tests, consolidation tests, triaxial compression tests, unconfined compression tests, direct shear tests, and permeability tests.

Appendix E summarizes the testing procedures and presents the detailed results from the testing program performed as part of this investigation. Appendix E also presents, in summary form, the results of the 1981 laboratory testing program.

Section 5.0

Subsurface Conditions

CCI/ESA/GRC

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

The geologic sequence in the site area consists of Alluvium (A) and bedrock of the Topanga Formation (T_t). The geologic units include:

- o Alluvium (A): These deposits are of Holocene age and are largely Los Angeles River channel deposits. The fine-grained alluvium overlies a fairly continuous layer of coarse-grained alluvium at the site. Locally, this unit contains large boulders; however, boulders were not encountered in the boreholes drilled at the Station site.

- o Topanga Formation (T_t): The bedrock underlying the Station area is of Middle Miocene age and consists of interbedded claystone, clayey siltstone, and sandstone with some lenses of sand and silty sand. Claystone predominates this unit at the Station site.

Drawing No. 2 shows a generalized subsurface cross-section through the proposed Universal City Station. Drawing No. 4 shows a more detailed subsurface profile through the site. The subsurface profile at the Station site consists of a thin pavement section which overlies generally fine-grained Alluvium that extends to depths of about 43 to 58 feet. Beneath the fine-grained Alluvium lies a relatively continuous layer of coarser-grained Alluvium which varies in thickness from about 2 to 16 feet. Underlying the coarse-grained Alluvium is the Topanga Formation bedrock. The bedrock surface at the Station site is relatively flat over the southern half of the site and is at about Elevation 512 (refer to Drawing No. 4). From Boring 34-3 to Boring 34-4, the bedrock surface drops about 5 feet in elevation and then rises about 15 feet from Boring 34-4 to Boring 34-5. The bedrock surface at Boring 34-5 is at about Elevation 523.

5.2 SUBSOILS

As discussed in Section 4.2, the subsurface conditions at the Station site were investigated by drilling a total of 5 rotary-wash borings during the course of this investigation. In addition to these borings, three rotary-wash borings and one large-diameter or man-sized boring were drilled in relatively close proximity to the Station site during previous investigations (see Appendix A).

Specific descriptions of the soils encountered in the borings drilled at the site include:

- o Fine-Grained Alluvium: The fine-grained Alluvium at the site consists of alternating layers and lenses of sandy and silty clays, clayey and sandy silts, and clayey sands. SPT blow count measurements taken in these soils situated near or below the level of the groundwater at the site range from 1 to 27 blows per foot and are typically between 10 and 20 blows per foot. These measurements and results of laboratory tests indicate that these soils range from very soft to stiff and very loose to medium dense below the groundwater level but are generally firm to stiff and medium dense. Above the water table, these soils have SPT blow counts between 9 and 43 blows per foot with average values in the range of 20 to 25 blows per foot. These SPT data indicate that these shallower soils are stiff to very stiff and medium dense to dense.

- <u>Coarse-Grained Alluvium</u>: Within this generally coarse-grained unit, the materials were predominantly silty fine to coarse sands and gravelly sands. Some of these deposits contain cobbles reported to be up to 6 inches in size. Borings drilled in close proximity to the Station site also encountered sandy gravels. These materials generally overlie the bedrock; however, relatively thin, discontinuous layers of silty and poorly graded sand were also found to be present within the fine-grained Alluvium. Results of Standard Penetration Tests (SPT) in these soils range from 11 to over 100 blows per foot with typical values between 20 to 50 blows per foot. These measurements indicate that these soils are generally medium dense to dense.

During the drilling of the rotary wash borings at the Station site, some difficulty was experienced in the sampling of some of the fine-grained Alluvium. As noted in the description of this material type, the SPT blow counts measured in some of the soils situated below the water table were exceptionally low and, in a few cases, the SPT sampler was advanced in the hole by the weight of the drill rod and/or the weight of the hammer. Sample recovery of these soils was also sometimes poor since the soil samples tended to "pull out" of the sampler.

Some difficulty was also experienced during the drilling of Boring 34-5. Caving of this hole was noted by the geologist at a depth of about 38 feet. During installation of the piezometer in this hole, the geologist indicated that the pea gravel placed around the PVC piezometer pipe either bridged in the hole or the hole caved in.

The behavior described above, as well as the results of laboratory tests, indicate that very soft and/or or loose layers, lenses, and/or pockets of clayey and sandy materials are present within the fine-grained Alluvium close to or below the groundwater table.

One large-diameter borehole (Boring 34C) was drilled near this Station site. This boring was drilled in Weddington Park on Valleyheart Drive, about 140 feet from the intersection of Bluffside Drive and Valleyheart Drive (see Drawing No. 2 for location of Boring 34C). This boring is located about 700 feet from the northern end of the Universal City Station structure. The ground surface at the location of Boring 34C is at approximately Elevation 552, which is about 21 feet lower than the ground surface elevation at the north end of the Station site. The purpose of this boring was to determine water levels and depths of alluvium above bedrock.

Artificial fill was encountered in Boring 34C from the ground surface to a depth of 10.5 feet and consisted of loose to medium dense silty sand and sandy silt that contained a significant amount of concrete and asphalt rubble. The artificial fill was subject to caving and ravelling.

Between the depths 10.5 and 26 feet in Boring 34C, Alluvium consisting of sand and silty sand with 15 to 25 percent round cobbles was encountered. It began to cave excessively at a depth of 21 feet, where groundwater was encountered. Upon reaching a depth of 26 feet, the hole caved back to 21 feet. Bedrock was not encountered during the drilling of this hole.

5.3 BEDROCK

All the borings drilled at the location of the proposed Station structure (i.e., Borings 34-1 through 34-5) penetrated the Topanga Formation bedrock underlying the Alluvium. At all of the locations, the bedrock consisted of interbedded claystone and sandstone with thin lenses of sand and/or silty sand. The claystone which predominates the bedrock at the site was found to be little weathered to fresh and moderately fractured to massive. The claystone was generally friable to moderately hard and friable to moderately strong; however, some hard and strong well-cemented zones were reported by the geologist logging the holes. Slickensides, thin coal seams, and steep bedding were also reported within the claystone. Where observed during the logging of the hole, the dip of the bedding planes of the rock at the Station site varied from 70 to 75 degrees. However, the dip of the bedding planes observed in some of the samples tested in the laboratory were less than 10 degrees. Borings drilled in close proximity to the Station also indicated the dip of the bedding planes to be in the range of 10 to 20 degrees. This variation in the dip of the bedding planes is characteristic of folded sedimentary rocks. Strike of the bedding could not be determined from the samples of the bedrock but is believed to be in a generally east-west direction. However, the bedding exposed in 1983 in the new Getty Headquarters Building foundation, 300 feet south of the Station site, had an average strike of N35W, and a 60-degree dip to the northeast.

The sandstone which is interbedded with the claystone is thinly to thickly bedded (typically between 1/16 inch to about 1 foot), little weathered to fresh, and weak to well cemented. Hardness and strength varied throughout the depth of the boreholes but was typically friable to moderately hard, and friable to moderately strong.

South of the Station site, the bedrock encountered in Boring 34A consisted primarily of sandstone which was found to be little weathered to fresh, friable to weak in strength, friable to moderate hardness, and weakly to well cemented. Clayey siltstone beds up to about 2 feet thick were encountered in the sandstone at the location of this borehole. Siltstone and clayey siltstone were also encountered in Boring 34B.

5.4 GROUNDWATER

The proposed Universal Station site lies within the San Fernando Valley basin. The Los Angeles River flows in a concrete-lined channel and is located about 1100 feet north of the Station site. Groundwater occurs at relatively shallow depths in the Alluvium, and a map showing groundwater contours for the San Fernando Valley basin (Los Angeles Flood Control District, 1974; see Figure 4-13 of the 1981 geotechnical report) indicates that regional groundwater flow occurs towards the Los Angeles River.

Table 5-1 presents groundwater levels and fluctuations measured in the piezometers installed at the Station site (i.e., Borings 34-3 and 34-4), and other borings located in relatively close proximity to the proposed Station (refer to Drawings No. 2 and 3). The water level observed during the drilling and logging of the large-diameter borehole (Boring 34C) is also listed. The groundwater elevations summarized in Table 5-1 are in reasonable agreement with the reported direction of the regional groundwater flow.

Our interpretation of the groundwater levels measured at the Universal City Station site and vicinity are shown in Drawings Nos. 2 and 4. The groundwater elevations at the south and north sides of the Station are about 562 feet and 550 feet, respectively. The groundwater level at the south end of the Station is about 67 feet above the bottom of the Station excavation, which is at Elevation 495. At the north end, the water level is about 57 feet above the bottom of the excavation, which is at Elevation 493.

From the piezometer installed in Boring 34-5, at the north end of the Station, to the one installed in Boring 34D located about 500 feet north of the Station site, the March 1984 groundwater levels appear to drop about 12 feet to Elevation 539. Proceeding to the north, the water level measured in Boring 34C during the time of drilling was at Elevation 531. This elevation roughly corresponds to the bottom of the Los Angeles River channel.

Chemical analyses have not been performed on any groundwater samples obtained from the Universal City Station site. During the 1981 geotechnical investigation, a total of three water samples taken from Boreholes CEG-33 and CEG-35 were subjected to chemical analyses. Boring CEG-33 is located about 2000 feet southeast of the proposed Station site while Boring CEG-35 is located about 3000 feet northwest of the Station site. Results of the chemical analyses performed during the 1981 investigation are summarized in Appendix C.

Two water samples were taken from Boring CEG-33 at relatively shallow depths (i.e., at depths less than 25 feet) on February 11, 1981. The chemical analyses of these two water samples indicate that the groundwater is of poor quality. Total Dissolved Solids (TDS) of both samples were in excess of 1000 PPM. For comparison, the U.S. Environmental Protection Agency TDS standard for potable domestic drinking water is 500 PPM. Sulfate contents of the samples were 693 and 538 PPM. A sulfate content above 150 PPM is generally regarded to be deleterious to concrete lining, requiring sulfate-tolerant concrete.

One water sample was taken from Boring CEG-35 at a depth of 95 feet on February 12, 1981. The TDS of this sample was 2605 PPM and was attributed to a high sodium chloride content of 2218 PPM. The high level of sodium chloride attests to the high mineralization of the groundwater in the San

Table 5-1

GROUNDWATER OBSERVATION WELL DATA

	Groundwater Elevation[a] (feet)					
Boring	Initial (Date)	4/81	1/83-2/83	12/83	2/84	3/84
CEG-34[c]	559 (12/8/80)	555	--	--	--	--
34A	568 (2/14/83)	--	568[b]	570	569	569
34B	553 (2/11/83)	--	553[b]	550	550	--
34C[c]	531 (1/25/83)	--	531[b]	--	--	--
34D	534 (2/10/83)	--	534[b]	531	--	539
34-3	--	--	--	560	558	558
34-5	--	--	--	550	551	551

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[a] Elevations rounded to the nearest foot.

[b] Initial reading recorded at time of drilling or within a few days after drilling.

[c] No piezometer installed in this borehole but water was encountered during drilling and logging.

Fernando Valley basin since the high sodium chloride content cannot be attributed to other sources such as oil field brines.

5.5 OIL AND GAS

No strong natural gas odors were detected during the drilling and logging of the borings located in the vicinity of the Station site. A sulfur odor was noted by the geologist in the log of Boring 34-5 at a depth of about 48 feet. Some organic-type odors were also detected during the drilling of the large-diameter borehole, 34C, and several of the rotary-wash borings drilled at the site. However, these odors have been attributed to the decay of roots and wood fragment in the Alluvial soils.

An oil slick appeared in the drilling mud during the drilling of Borings 34-3, 34-4, 34-5, and 34A (see Drawing No. 2 for location of these borings). The oil slicks mentioned in the logs of 34-3, 34-4, and 34-5 first appeared at depths of about 71, 87, and 96 feet, respectively. The bedrock at the locations of these three holes consisted of interbedded claystone and sandstone. The oil was first noted at a depth of 113 feet in Boring 34A, approximately 63 feet beneath the contact between the Alluvium and weathered bedrock. The bedrock at the location of this boring was sandstone with interbedded siltstone. The appearance of this oil suggests that the bedrock is probably slightly petroliferous at and in the vicinity of the Station site.

5.6 FAULTS

The proposed Universal City Station is located north of the projected, concealed trace of the Benedict Canyon fault. The Benedict Canyon fault is not known to be active or potentially active. The location of the fault is based on topographic expression on the north flank of the Santa Monica Mountains and confirmed by additional subsurface data obtained during the course of the 1981 geotechnical investigation. Based on exposures of the bedrock in the general vicinity of the Station site, together with previous construction experience in the same geologic formation in close proximity to the Station site, faults encountered during excavation at the Station site should not present any major problems.

A more detailed description and additional information regarding the Benedict Canyon fault are contained in the 1981 geotechnical investigation report (Volume 1, Section 4.4.2, and Volume 2, Appendix D).

5.7 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.7.1 General

For purposes of our engineering evaluations, we have grouped the subsurface materials encountered at the Universal City Station site into general subsurface units. The main subsurface units affecting design include fine-grained and coarse-grained Alluvium (A), and the Topanga Formation bedrock (T_t). 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 field and laboratory test results, published data, and engineering judgment.

5.7.2 Fine-Grained Alluvium

This alluvial unit consists of interbedded silty and sandy clays, clayey and sandy silts, and clayey sands with lenses, layers, and pockets of silty sand and sand. Above the water table, these soils are generally stiff and medium dense to dense. However, close to and below the groundwater table, these soils may be soft to firm and loose to medium dense.

For these materials, both drained (effective) and undrained (total) strength parameters have been developed primarily from the results of triaxial compression tests. The recommended strength parameters given in Table 5-2 have been developed from the results of tests performed on samples obtained from the Station site, although a limited number of strength test results for similar materials, obtained from other boreholes located in other design units, were used in the development of both sets of strength parameters.

Young's Modulus or initial tangent modulus values for these materials were developed using results of triaxial compression tests performed as part of this investigation and checked for consistency with tests performed on similar material types from other design units. Modulus values were found to be a function of the mean confining pressure at the end of the consolidation process.

Permeability tests performed on triaxial test samples of fine-grained Alluvium obtained from the Station site and other design units indicate that these soils have permeabilities 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.

5.7.3 Coarse-Grained Alluvium

This alluvial unit consists primarily of silty fine to coarse sands, gravelly sands, and poorly graded sands which are generally medium dense to dense. The strength parameters listed in Table 5-2 were developed from the results of a limited number of triaxial compression tests performed on soil samples obtained from the Station site and tests performed on similar soil types from other design units. Drained (effective) strength parameters are considered appropriate for static design.

As in the case of the fine-grained Alluvium, the Young's Modulus or initial tangent modulus was found to be a function of the mean confining pressure at the end of consolidation. These materials have modulus values which are greater than those obtained for the fine-grained Alluvium.

Permeability tests performed on a limited number of triaxial test specimens of silty sand during this and the 1981 investigation yield permeabilities varying between 10^{-4} and 10^{-6} cm/sec. However, realizing the fact that permeabilities that were measured during testing are more appropriate for vertical seepage versus horizontal seepage, and since the soils that

Table 5-2

MATERIAL PROPERTIES SELECTED FOR DESIGN

Material Property	Fine-Grained Alluvium	Coarse-Grained Alluvium	Topanga Formation
Moist Density Above Groundwater (pcf)	125	125	130
Saturated Density (pcf)	130	130	130
Effective Stress Strength			
ϕ' (degrees)	33	38	28
c' (psf)	0	0	0
Total Stress Strength[a]			
ϕ (degrees)	20[a]	--	12
c (psf)	0	--	1200
Unconfined Compressive Strength (psf)[e]	3000[f] 1500	--	4000[g]
Permeability (cm/sec)	10^{-4} to 10^{-7}	10^{-3} to 10^{-5} (c) 10^{-1} to 10^{-3} (d)	10^{-6} to 10^{-7}
Initial Tangent Modulus, E_i (psf)	$300\ \sigma'_v$[b]	$500\ \sigma'_v$[b]	2×10^6
Poisson's Ratio	0.40	0.35	0.40

---

[a] The total stress parameters should be used to determine the increase in undrained strength with depth for use in undrained strength analyses where $\phi = 0$ degrees.

[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] Range of permeabilities for poorly graded and silty fine sands.

[d] Range of permeabilities for sandy and/or silty gravels and coarse sands.

[e] Values represent lower bound unconfined strength for these materials. Samples of alluvium tested were generally sandy clays and clayey sands of low plasticity and containing in some cases lenses and seams of sand. Topanga Formation samples were generally brittle and tended to fail along slickensides, bedding plane, and sand lenses

[f] Higher strength value is applicable to fine-grained Alluvium within 10 feet of the ground surface. Lower strength value appropriate below a depth of approximately 10 feet.

[g] Unconfined strength is an average value for claystone/sandstone of Topanga Formation. Laboratory test results range from 1200 psf to 16,700 psf.

were encountered at the site are rather variable, permeability values which are somewhat higher than those reported in the laboratory test results are recommended for design. It should be noted that sandy and/or silty gravels, some containing cobbles, have been encountered near the Station site. Soil samples of these materials were not tested in the laboratory during this investigation. However, a pump test was performed about 750 feet west of the Station site in April 1983 (see Drawing No. 2, Pump Test Well PT-2). The materials that were selected for aquifer testing consisted of a bed of clean sand and gravel and fine sand. It is our judgment that these soils have hydraulic characteristics which are similar to those of the sands and gravels which directly overlie the bedrock at the Universal City Station site. The general hydraulic characteristics determined on the basis of the pump test results are as follows:

- Transmissivity: About 19,000 gpd/ft (average).

- Storage Coefficient: Computed values vary between 0.008 to 0.059 because of the short duration of the test. It should be noted that as these deposits are dewatered, a specific yield value that is considerably greater than the computed value of storativity will apply.

- Boundaries: A boundary was observed during one of the two pump tests conducted at the location of PT-2. The distance to the observed boundary could have been computed; however, it would not be applicable to the Universal City Station excavation.

- Saturated Thickness: Ranges between 12 and 15 feet.

- Average Formation Permeability: Computed to be about ~9.0 x 10^{-2} cm/sec (average). However, individual layers may have widely varying permeabilities.

A description of the general procedures and the results of the pump test are presented in Appendix C.

5.7.4 Topanga Formation Bedrock

For engineering purposes, the Topanga Formation bedrock consisting of interbedded claystone and sandstone was considered to be a very stiff to hard fine-grained soil. Due to the clayey nature of the bedrock materials and the possibility of various loading conditions, both drained (effective) and undrained (total) strength parameters were considered in developing design recommendations. Strength parameters presented in Table 5-2 were based on interpretation of triaxial, unconfined compression, and direct shear tests, as well as engineering judgment.

The unconfined compressive strength listed in Table 5-2 for the Topanga Formation bedrock represents a reasonably conservative (i.e., low) estimate for the intact rock at the Universal City Station site. The testing of the samples of the rock was, in many cases, difficult since samples did not extrude easily from the sampling tubes (or rings). In addition, many of the samples contained lenses of weakly cemented siltstone (silt) or sandstone (sand), thus making them unsuitable for testing. For these reasons, only three out of a total of eight samples scheduled for unconfined testing were tested during the course of this investigation. Consequently, the unconfined strength given in Table 5-2 is largely based on laboratory tests which were performed as part of a supplementary geotechnical investigation conducted in early to mid 1983 by Converse Consultants (CCI, 1983). Unconfined strengths from the supplementary study range between 1,200 and 16,700 psf (see Appendix E).

Bedrock elastic properties were selected based on consideration of field performance data and laboratory test data, and published information combined with engineering judgment. For this study, the bedrock was considered to have no significant modulus increase within the range of depths affected by the proposed Station.

Section 6.0

Geotechnical Evaluations and Design Criteria

6.0 GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA

6.1 GENERAL

Geotechnical design criteria for design and construction of the Universal City Station are provided in this section of the report. To the extent practical, the criteria have been generalized to consider various potential design and construction concepts. As the design is finalized and specific details are formulated, these geotechnical criteria may be subject to some revision.

The excavation for the Station will be through alluvial deposits which consist of fine-grained and coarse-grained alluvium. These alluvial deposits are underlain by bedrock of the Topanga Formation which consists of interbedded claystone and sandstone (see Drawing No. 2). A detailed description of the materials comprising these units has been presented in Section 5.0. As shown in Table 6-1, the depth of the excavation at the Station will range from 84 feet (Elevation 495) at the south end of the Station to 80 feet (Elevation 493) at the north end. The measured groundwater table is at a depth of 16 feet below the ground surface at the south end of the Station, and at a depth of 23 feet at the north end. The permanent structure will in essence be a concrete box bearing on the Topanga Formation and retaining Topanga Formation and alluvial deposits.

The primary geotechnical considerations at the Station site include:

- o Construction dewatering and subsidence considerations.
- o Selection, design, and construction of the temporary shoring system.
- o Establishing magnitude and distribution of soil and water pressures acting on the permanent structures, and designing for these loads.
- o Evaluating potential for earthquake-induced liquefaction and strength loss within zones of the alluvial deposits.

6.2 EXCAVATION DEWATERING

6.2.1 <u>General</u>

Based on an excavation bottom at about Elevation 494, the proposed excavation will extend some 57 to 67 feet below the measured groundwater levels at the site. Of this total depth, 27 to 51 feet will consist of saturated alluvial deposits which will require construction dewatering to complete the excavation. The bottom of the excavation will be within the Topanga Formation, which appears to be quite impermeable. However, the alluvial sands overlying the Topanga Formation are quite permeable and could result in significant water inflows into the excavation. Dewatering of this sandy zone will be required to prevent the possible development of high hydrostatic uplift pressures within this zone which could lead to a "blow out" as the excavation progresses downward.

Table 6-1

SUMMARY OF EXCAVATION AND GROUNDWATER DEPTHS AND ELEVATIONS DESIGN UNIT A425--UNIVERSAL STATION

	Elevation (feet)				Depth (feet)	
	Ground Surface	Top of Rail	Bottom of Excavation	Measured Water Level	Depth to Groundwater	Depth of Excavation
South End of Station	579	504	495	562	16	84
North End of Station	573	503	493	550	23	80

---

*All elevations and depths rounded to nearest foot.

It is our opinion that there are two basic methods for the control of groundwater during construction at the Station site:

Method I: Draw down the groundwater within the subsoils surrounding the site, including the clayey sands and silty sands within the fine-grained alluvium, and the coarse-grained alluvium directly overlying the Topanga Formation.

Method II: Provide a groundwater barrier or cut-off which penetrates the Topanga Formation thereby requiring dewatering only within the boundaries of the Station excavation.

If the Station site is dewatered using Method I, our evaluation indicates that significant 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.

Method II could be accomplished using a tight shoring system such as slurry wall construction which penetrates into the bedrock. The advantage of this method is that dewatering operations are greatly reduced and the risk of subsidence due to dewatering is essentially eliminated.

As previously indicated, the dewatering system must relieve the hydrostatic pressures within the alluvial sands overlying bedrock to prevent basal heave or "blow-out" of the excavation. Groundwater inflow to the dewatering system will, therefore, be primarily from the permeable coarse-grained alluvial sands. Drawdown within these sands will probably occur within a few weeks; however, complete drawdown within the overlying fine-grained alluvium may require a few months. The shape of the drawdown surface is expected to be characteristic of the more permeable sands than the fine-grained alluvium. A relatively flat drawdown surface is expected which may extend about 500 feet beyond the excavation. Geologic discontinuities, i.e., major variations in the deposits could cause variations in the phreatic surface especially during the early stages of dewatering.

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. A possible dewatering system might consist of the following:

- o Deep wells placed around the perimeter of the excavations penetrating the lower alluvial sands to bedrock.

- Use of secondary wells or wellpoints in certain localized areas within the excavation to dewater sandy layers encountered within the fine-grained alluvium; spaced more closely than the deep wells and pumping only from the upper fine-grained alluvium deposits.

- Supplementary ditch drains and sump pumps within the excavation.

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. Irrespective of the method used to dewater the excavation, the contractor should satisfy the following criteria, as applicable:

- The dewatering system should be installed and in operation for a sufficient period prior to and during excavation to adequately drawdown the groundwater table.

- The system should maintain the groundwater levels low enough to prevent piping of the alluvial soils into the excavation. Inflow quantities should be reduced to levels which can be handled by a drain/sump system and allow excavation and construction to proceed.

- The dewatering system should maintain water levels low enough to assure the stability of the bottom of the excavation against a "blow out" failure at all times during construction.

- The contractor should be responsible for disposing of the dewatering discharge. He should be made aware of the potential environmental problems; i.e., odors from dissolved gases. The contractor should be made responsible for resolving such potential problems and satisfying applicable codes and ordinances.

- Wells must be designed and developed to eliminate loss of ground from piping of soils from around the wells. The well operations should be constantly monitored for evidence of piping.

- The system should be capable of continuous operation. Emergency power and backup pumps should be required to ensure continual excavation dewatering.

6.2.4 Induced Subsidence

Up to 50 vertical feet of saturated alluvial deposits are expected to be affected by dewatering operations during construction. Potential settlements due to dewatering were calculated based on the assumption that the subsurface conditions within a few hundred feet of the site were similar to those encountered in the borings. These calculations indicate that total surface settlement would be about 3 inches for up to 50 feet of drawdown and 1 inch for up to 20 feet of drawdown. Settlement of this magnitude could damage nearby structures but total subsidence would require several weeks to months to occur due to the low permeability of the fine-grained alluvium. Local dewatering contractors indicate that significant dewatering subsidence does not occur during construction, but unless this can be verified by documented case histories or a site specific pumping test it should be assumed that significant settlement could occur over the long term. Some of the settlement caused by dewatering would rebound after dewatering is terminated and water levels reach equilibrium.

6.3 UNDERPINNING

6.3.1 General

The need to underpin and the appropriate type of underpinning for specific buildings located adjacent to a proposed excavation depends on many factors. Some of the most important factors are soil and groundwater conditions, depth of excavation, type of structure and proximity to the excavation, type of shoring, and consequences of potential ground movements.

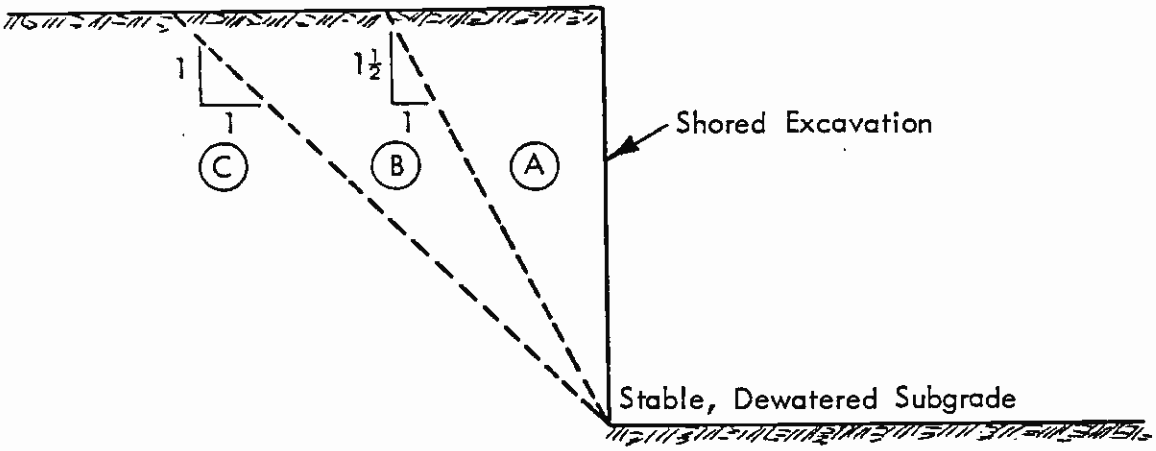
Figure 6-1 presents guidelines for assessing when underpinning needs to be considered. Based on this figure, and the fact that most of the structures in the immediate vicinity of the Station site will be demolished for construction of new above-ground facilities, there does not appear to be a need for underpinning at the Universal City Station site.

6.4 TEMPORARY SLOPED EXCAVATIONS AND SHORING SUPPORT SYSTEMS

6.4.1 General

The proposed excavation depths below the existing ground surface are tabulated in Table 6-1. A temporary support system utilizing a conventional shoring system with either tiebacks or internal bracing for lateral support is feasible at this site. Driven sheet piles are not considered feasible at this site due to the presence of the dense alluvial gravelly sands, which would make driving extremely difficult, if not impossible, in these dense materials. We understand that the shoring 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 discussion and design criteria presented in this section pertain to soldier beam and lagging type shoring systems. Other shoring support systems may also be appropriate and may be considered by the contractor.



NOTES: 1.) These guidelines are applicable only for stiff or dense stable ground conditions. Other soil and/or foundation conditions may require further analyses.

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

ZONE Ⓐ 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.

ZONE Ⓑ Generally No Special Provisions Required:

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

ZONE Ⓒ No Special Provisions

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UNDERPINNING GUIDELINES

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Figure No.
6-1

6.4.2 Sloped Excavations

Where it is practical and space permits, portions of the required excavation could be made with a sloped excavation, particularly the shallower cuts around the entry structures. Sloped excavations would significantly reduce the height of such temporary shoring. The use of sloped excavations at the site would depend on whether easements can be obtained to extend the limits of the excavation. Construction of a wide bench at the toe of the cut slope would probably be required to provide access to the shored excavation but would increase the volume of excavated soil.

The major factors which detemine the safe, stable slope include soil conditions, groundwater conditions, the weather (i.e., dry or heavy rain), construction procedures and scheduling, and others. Applicable governmental safety codes must also be complied with.

For evaluation of excavation alternatives, temporary slopes of 1.5H:1V may be assumed for the alluvium deposits above the groundwater table. These recommendations assume suitable site dewatering where necessary, no heavy loads at the top of the slope, slope protection, and some slope maintenance. In addition, these recommendations should not be construed by the contractor to be a guaranteed permissible slope since the actual safe slope will be a function of actual construction and field conditions.

6.4.3 Conventional Shoring System

A soldier pile and lagging shoring system consisting of soldier piles installed in pre-drilled holes is a common method of shoring deep excavations in the Los Angeles area. Appendix F.1 summarizes several case studies in the Los Angeles area involving soldier pile excavations to depths exceeding 100 feet.

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 F.1 have not been strictly verified by measurements during construction. However, the performance of shoring systems designed based on local practice has generally been good. Therefore, the local practice was considered in the development of our recommended design criteria.

Soldier piles have been installed in the Los Angeles area in soils similar to those encountered at the proposed Station site. Within the alluvium, particularly below the groundwater 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 such as tiebacks.

The coarse-grained alluvial soils 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.

6.4.4 Shoring Design Criteria

This section provides design criteria for a conventional shoring system consisting of soldier piles and wooden lagging supported by tiebacks or internal bracing. The soldier piles are assumed to consist of steel W or H-sections installed in predrilled circular shafts. It is assumed that the drilled shaft will be filled with structural concrete below the bottom of the excavation and lean mix above the subgrade. Thus, for computing the allowable vertical and lateral capacities, the piles are assumed to have circular concrete sections.

Specific shoring design criteria include:

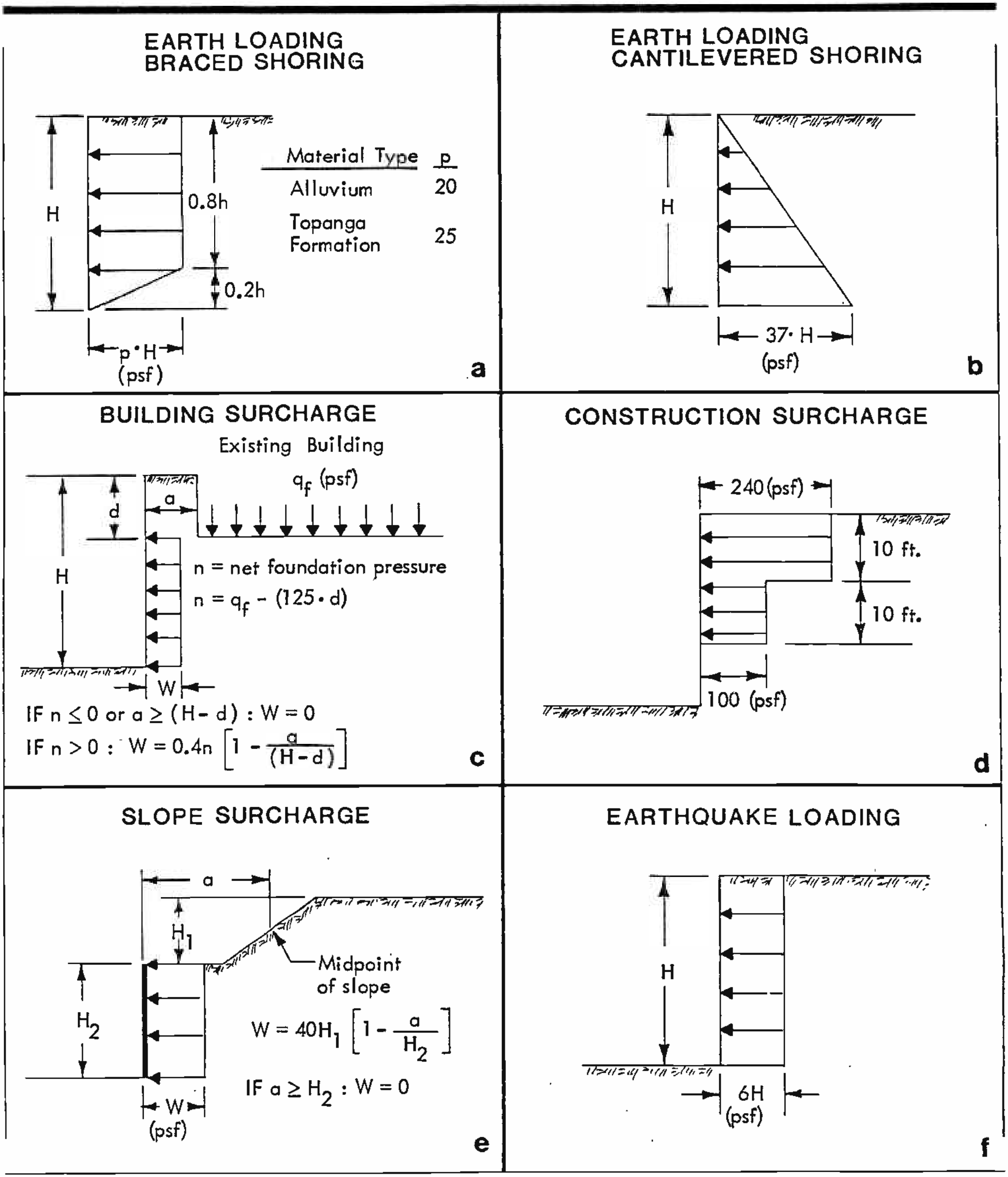
- Design Wall Pressure: Figures 6-2a and 6-2b present the recommended lateral earth pressure on the temporary shoring walls. Figure 6-2e also includes the case of partial sloped cuts. The full loading diagram should be used to determine the design loads on tieback anchors and the required depth of embedment 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 capacities 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 6-3. These values include both end bearing and shaft friction within the bearing stratum. It should be noted that all piles should penetrate at least 5 feet into the underlying Topanga Formation bedrock.

  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-2 minus the support from tiebacks 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 6-4 indicates the recommended method to compute net passive resistance.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil loads, member sizes, and costs. At the Station site the alluvial soils consist of sandy and clayey soils which may 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 temporary exposed soil height to less than 3



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No.
6-2

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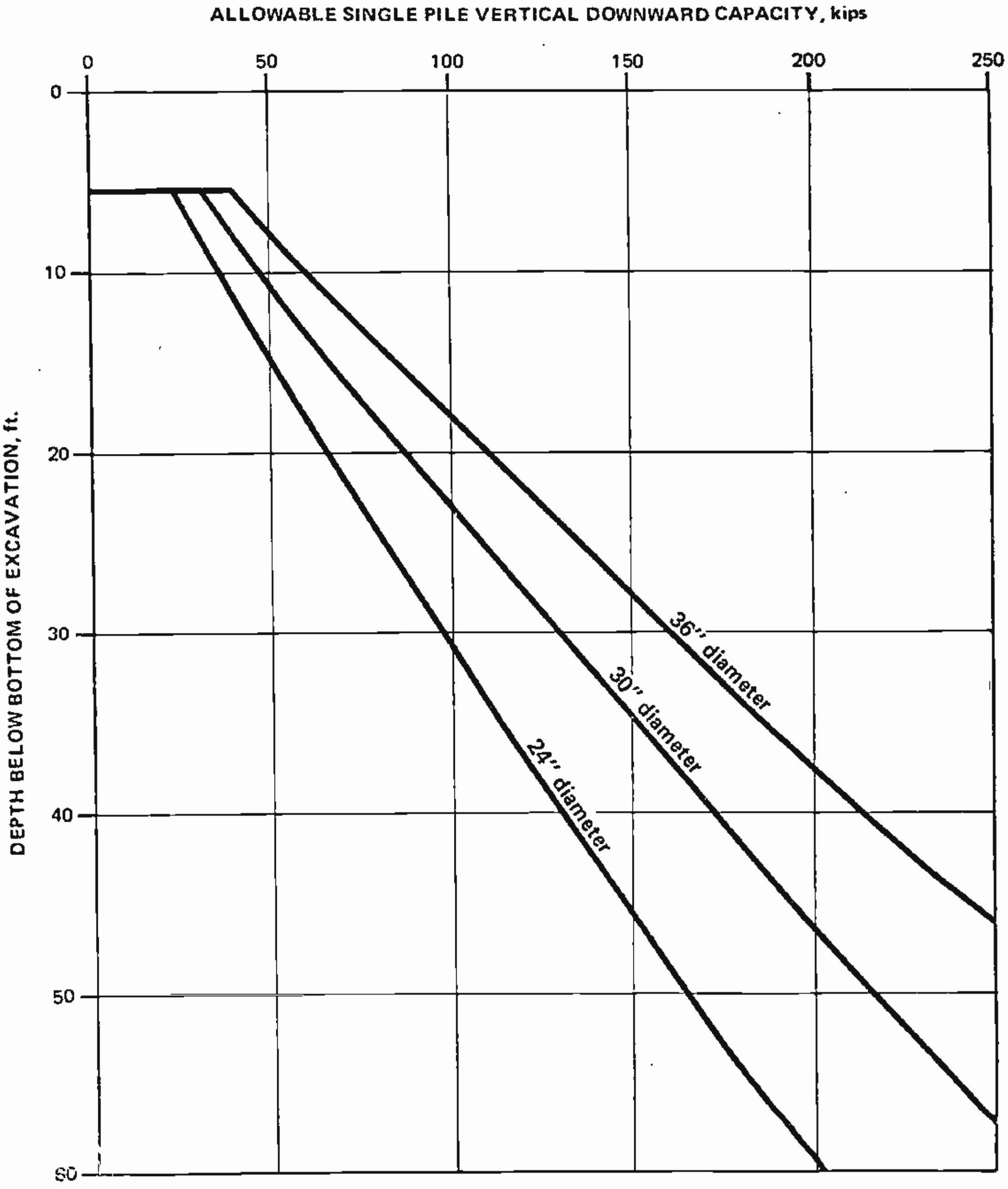

NOTES:
1) Total capacity includes contributions from both shaft frictional resistance and end-bearing.
2) For seismic design, capacities may be increased by one-third.
3) Groundwater level at bottom of excavation.
4) All piles must penetrate at least 5 feet into the bedrock.
5) Applicable only for drilled shaft piles.

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ALLOWABLE VERTICAL PILE CAPACITY IN TOPANGA FORMATION FOR SHORING

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Figure No
6-3

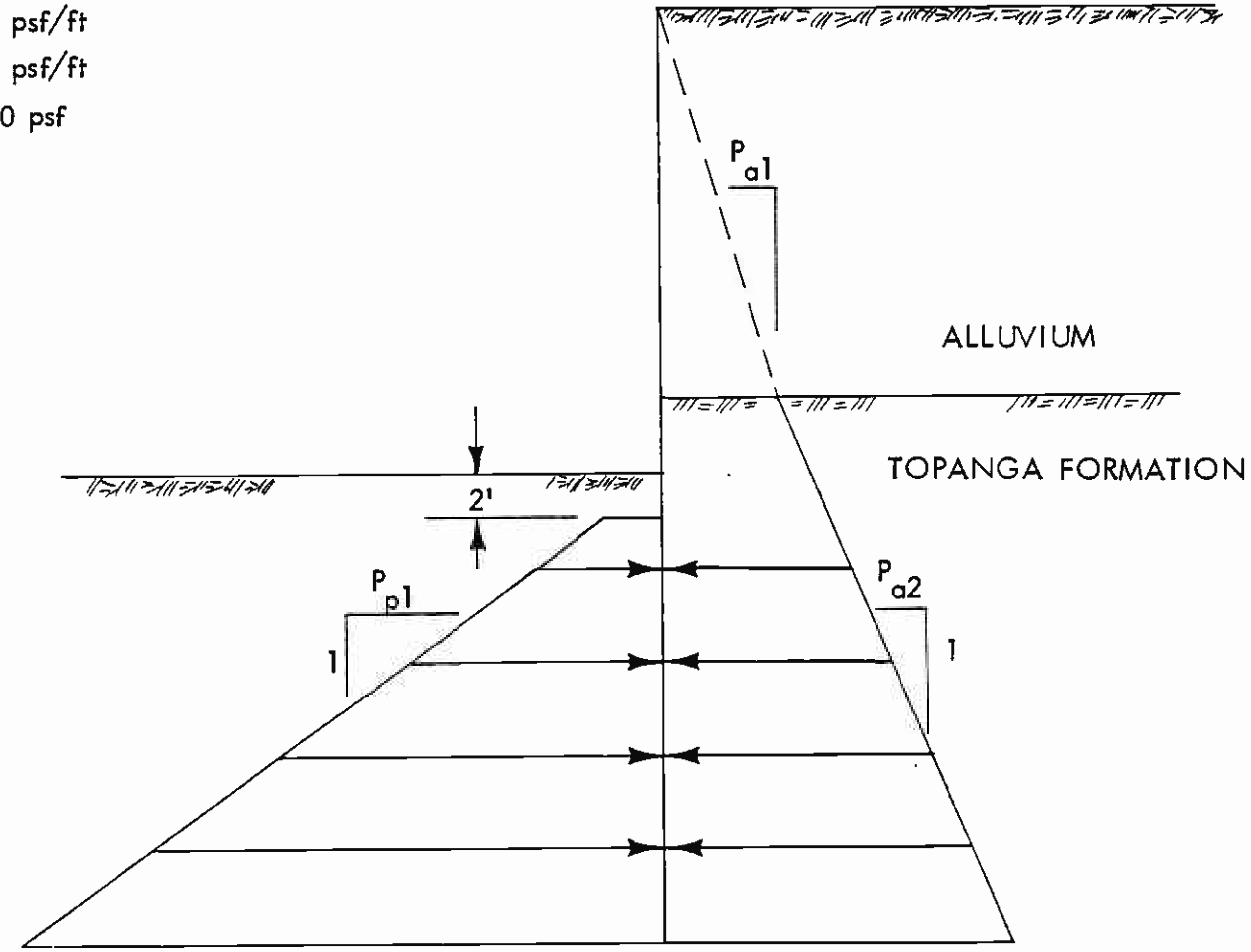
Converse Consultants Geotechnical Engineering and Applied Sciences

Recommended Unit Pressures

$P_{a1} = 37$ psf/ft

$P_{a2} = 47$ psf/ft

$P_{p1} = 250$ psf



Where: P_p = 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 $\frac{1}{2}$ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-2

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SOLDIER PILE PASSIVE RESISTANCE (TOPANGA FORMATION)

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Figure No.
6-4

feet to control ravelling problems, especially below the groundwater level.

- Shoring/Tieback System Overall Stability: Stability calculations should be performed as part of the shoring design to verify that the shoring tieback system has an adequate factor of safety against deep-seated failure.

6.4.5 Internal Bracing and Tiebacks

6.4.5.1 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.

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.

6.4.5.2 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 a maximum allowable vertical distance of 12 feet between struts.

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

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable. Stresses due to temperature variations shall be taken into account in the design of the struts.

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

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

$P = \pi DLq$ (anchor capacity)

where

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

The design adhesion value (q) can be taken equal to:

$q = 20d < 500$ psf , in fine-grained alluvium
$= 750$ psf , in coarse-grained alluvium and bedrock

where:

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

Allowable anchor capacity/length relationships for tieback types other than straight shaft friction anchors such as high pressure grouted anchors and high pressure regroutable anchors must be based on experience in the field and on the results of test anchors.

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

The anchors may be installed at angles between 20 and 50 degrees 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 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. Potential caving in the alluvium could be a problem particularly for anchors installed below the groundwater table. Uncontrolled caving not only causes installation problems but could result in surface subsidence and settlement of overlying buildings. To minimize caving, casing could be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, 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.

6.4.6 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 excavation cases in similar ground conditions, combined with our engineering judgment, we estimate that the ground movements associated with a properly designed and carefully constructed conventional wall shoring system, with either tieback anchors or internal bracing will be as follows:

- 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 vertical settlement behind the wall should be equal to about 50% to 100% of the maximum horizontal deflection and will probably occur at a distance behind the wall equal to about 25% to 50% of the excavation depth.
- 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.

6.5 SUPPORT OF TEMPORARY DECKING

Temporary street decking will not be required at this Station site since the proposed Station location is not situated on a main street, and since all above-ground structures will be removed prior to construction.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed excavation at the Station site 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 risk associated with claims against the owner/contractor. If substantial cracks are noted in the existing structures, they should be measured and periodically remeasured during the construction period.

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

- Inclinometers: It is recommended that a limited number of inclinometers be installed prior to excavation and monitored around the Station's excavations. 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 a short time after installation. Subsequent readings should be made at regular 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 Stations' structures.

  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 it from equipment, yet allow for easy access should the hole collapse.

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

- Convergence Measurements: We recommend the use of tape extensometers to measure the convergence between the 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 A means should be provided for measuring the strut temperature.

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

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

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

6.7 EXCAVATION HEAVE AND SETTLEMENT OF STRUCTURES

The proposed excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed 80- to 84-foot excavation at the Universal City Station will decrease the vertical ground stresses by about 6600 to 7000 psf. These stress reductions will cause the soils below the bottom of the excavations to rebound or heave. 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 soils adjacent to the walls to heave upward. Since the excavation will be open for an extended period, the heave is expected to be completed prior to construction of the Station. Construction of the Station structures and subsequent backfilling will reload the soils. We estimate that the Station and backfill loads will be in the range of 6000 to 7000 psf.

The maximum heave at the center of the excavations was calculated to be on the order of 2 to 4 inches. We also believe that the majority of this heave will occur during the excavation phase of construction. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (elastic heave) within the bedrock underlying the proposed excavation.

Settlement on the order of 2 to 3 inches were computed due to the imposed loads from the structures and backfill. This will occur even though the weight of the excavated material exceeds the weight of the completed structure and backfill. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/2 inch over half the structure width. These calculations are based on the assumption of a uniform foundation bearing pressure and a perfectly flexible structure. The actual differential settlement will be less than the theoretical flexible foundation case because of the rigid type Station structure.

We understand that MRTC is contemplating modification 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 simplifying uniform pressure approach is left to the discretion of MRTC and the Section Designer.

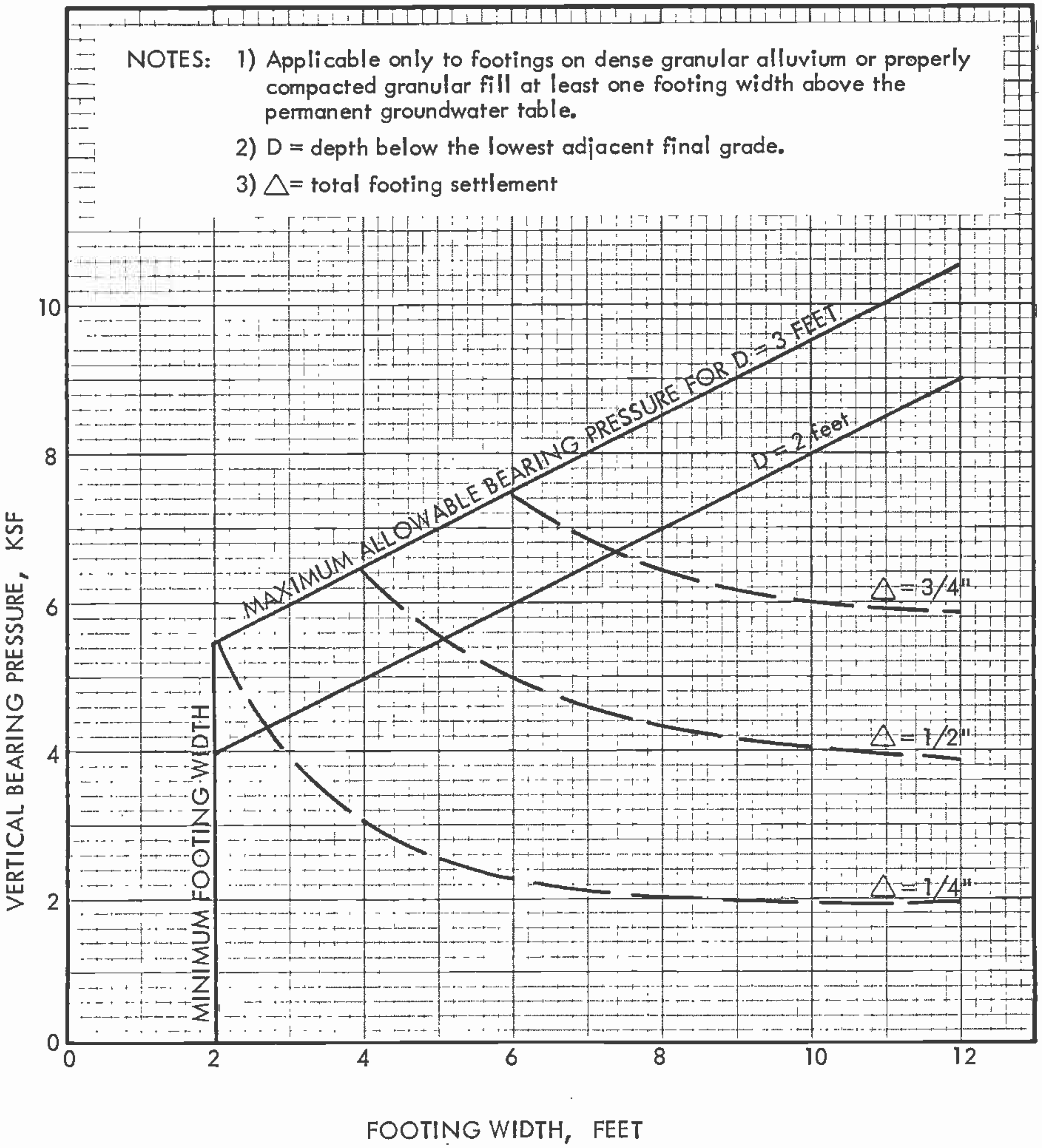
6.8 PERMANENT FOUNDATION SYSTEMS

6.8.1 Main Station

The base of the proposed Station structure will function as a massive mat foundation. At the proposed foundation levels, the mat will be bearing on the Topanga Formation. We estimate that the average net foundation bearing pressures for the Station will range from about 1500 to 2500 psf. In our opinion the Station can be adequately supported on a mat foundation bearing on the underlying Topanga Formation, as indicated in the previous section.

6.8.2 Support of Surface Structures

Surface structures can be generally supported on conventional spread footings founded on properly compacted fill or on undisturbed stiff or dense alluvium. Allowable bearing pressures and estimated total settlements of spread footings can be estimated based on Figures 6-5 and 6-6. Figure 6-6 is only applicable for shallow footings at depths less than 10 feet. At greater depths, the strength of the fine-grained alluvium decreases significantly due to saturation. At these depths, footings should be founded on properly compacted granular fill. These figures are generally conservative due to lack of detailed information on structural loadings and site



ALLOWABLE BEARING & SETTLEMENT FOR SPREAD FOOTING ON GRANULAR SOILS

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Project No. 83-1140

Figure No. 6-5

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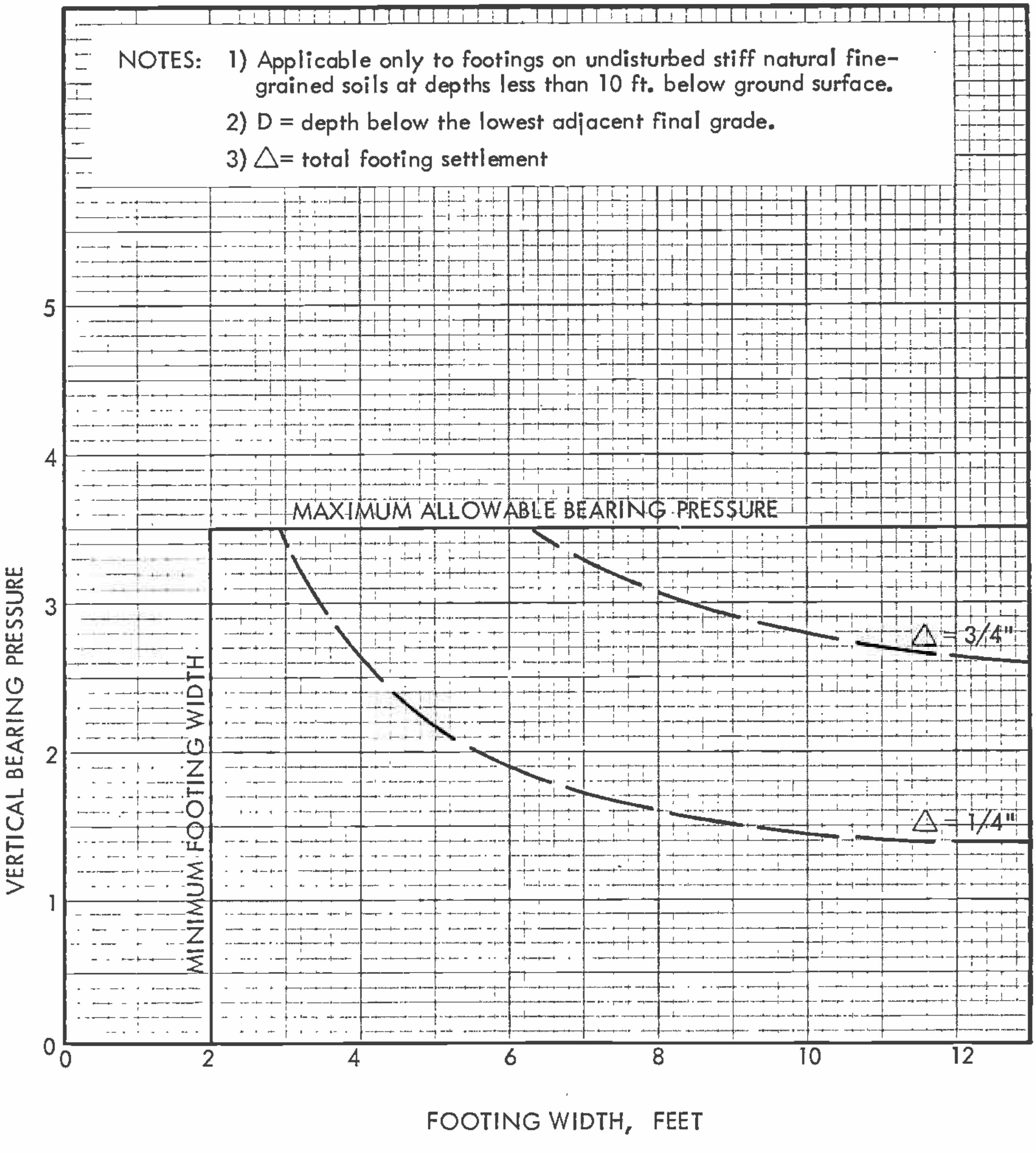

NOTES:
1) Applicable only to footings on undisturbed stiff natural fine-grained soils at depths less than 10 ft. below ground surface.
2) D = depth below the lowest adjacent final grade.
3) △= total footing settlement

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

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Figure No.
6-6

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conditions at the surface structure location. Where necessary, 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 6-5 and 6-6 are for full dead load and frequently applied live load. For wind loads, the bearing values can be increased by one-third. This increase should not be allowed for seismic loading conditions. Differential settlements between adjacent footings should be estimated as 1/2 of the average total settlements or the difference in the estimated total settlements shown on Figures 6-5 and 6-6, whichever is larger.

For design, resistance to lateral loads on surface structures can be assumed to be provided by passive earth pressure and friction acting on the foundations. An allowable passive pressure of 300 psf/ft may be used for the sides of footings poured neat against dense or stiff alluvium or properly compacted fill. Frictional resistance at the base of foundations should be determined using a frictional coefficient with 0.4 with dead load forces.

6.9 PERMANENT GROUNDWATER PROVISIONS

We understand that the Station will be designed to be water-tight and to resist the full permanent hydrostatic pressures. We recommend that full waterproofing be carried at least 5 feet above the anticipated maximum groundwater levels given in Section 6.10.

6.10 STATIC LOADS ON PERMANENT SLAB AND WALLS

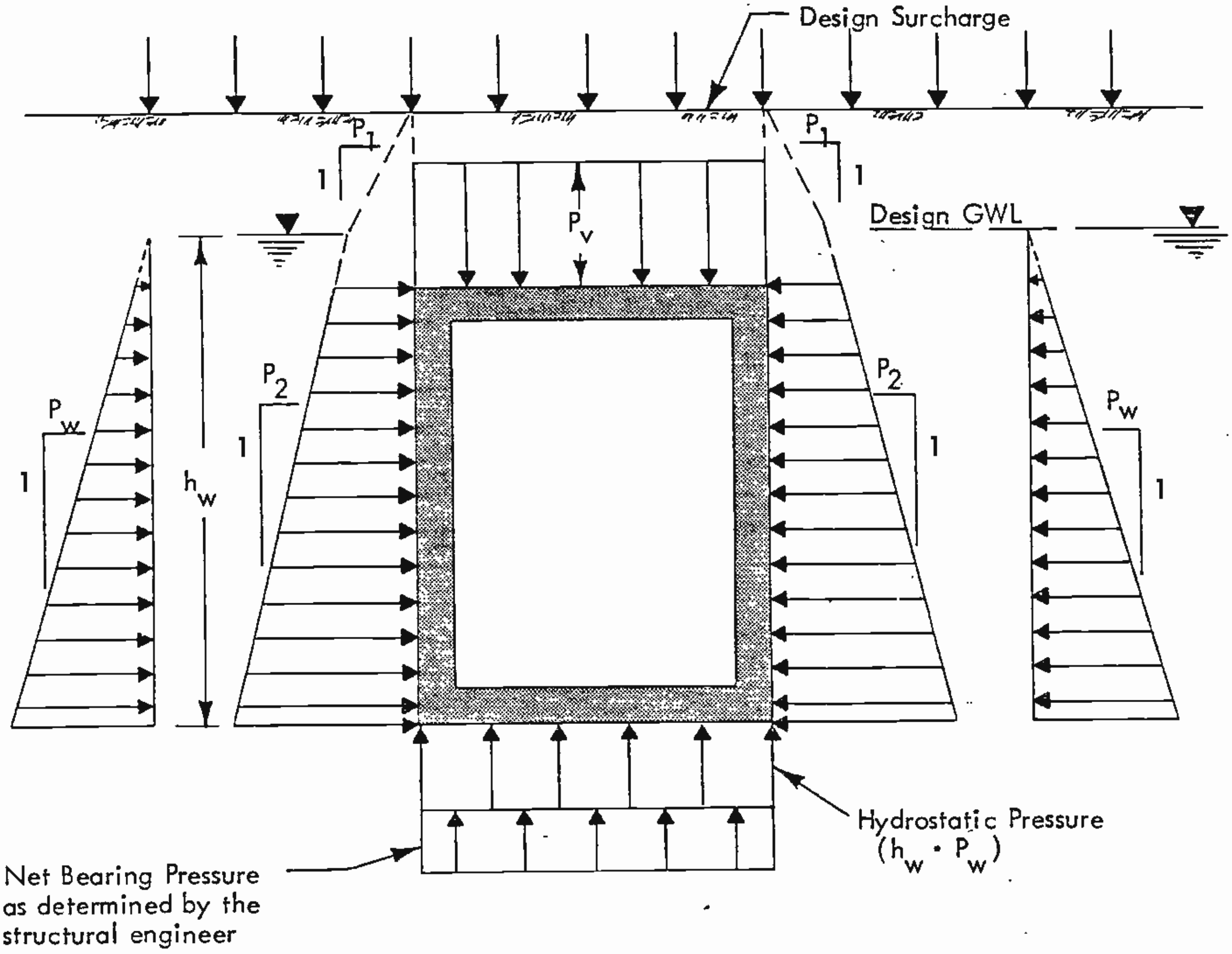
6.10.1 Hydrostatic Pressures

As tabulated in Table 6-1, the groundwater levels as measured within the borings drilled at the Station site in 1983 ranged from Elevation 562 at the south end of the Station to Elevation 550 at the north end. These levels are considered to represent close to the maximum levels to be expected. It is recommended that for design the maximum groundwater levels be assumed to be approximately five feet higher than the 1983 measured levels.

6.10.2 Permanent Static Earth Pressures

The static lateral and vertical earth pressures recommended for design of permanent buried structure are tabulated in Figure 6-7.

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



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	37	20	62.4	(1)	(2)
Long Term	60	$31^{(4)}$ $36^{(5)}$	62.4	(1)	(3)
Side sway	37/60	$20/31^{(4)}$ $20/36^{(5)}$	62.4	(1)	(2)

(1) P_v = full overburden pressure (depth x total density) plus surcharge
(2) For end of construction and side sway loading conditions the designer should use a GWL (between the base of the slab and long term water elevation) which will be critical for the loading condition.
(3) For long term the GWL varies linearly from Elev. 567 at the south end to Elev. 555 at the north end.
(4) Applicable to alluvium
(5) Applicable to Topanga Formation

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LOADS ON PERMANENT WALLS

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Figure No.
6-7

6.10.3 Surcharge Loads

Lateral surcharge loads from existing surface structures above an elevation equal to the invert of the Station must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-2. Vertical surcharge loads due to 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 PARAMETERS FOR SEISMIC DESIGN

6.11.1 General

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the Southern California Rapid Transit District (SCRT) report entitled "Guidelines for Seismic Design of Underground Structures," dated March 1984. The evaluation of the seismological conditions which may impact the project and the earthquake intensities which may be anticipated in the Los Angeles area are described in the SCRT 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

Values of apparent wave propagation velocities for use in travelling wave analyses have been previously recommended in the May 1983 seismic design criteria report. Other dynamic soil parameters may also be required for use with various types of seismic analyses. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels. In addition, certain types of equivalent linear analyses require that the variation of dynamic shear modulus and soil hysteretic damping with the level of shear strain be known.

Average values of compression and shear wave velocities based on interpretation of limited crosshole geophysical surveys performed in Boring CEG-34, and other borings in similar materials during the 1981 investigation are presented in Table 6-2. These velocities have been used together with the tabulated values of density and Poisson's ratio to establish appropriate modulus values at low strain levels. Computed modulus values for the fine-grained and coarse-grained alluvium and the Topanga Formation are tabulated in Table 6-2.

The variation of dynamic shear modulus, expressed as the ratio of G/G_{max}, with the level of shear strain is presented in Figure 6-8 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 6-9. These relationships were developed from the results of field geophysical surveys, resonant column tests, and cyclic triaxial tests performed in the field and in the laboratory on representative samples of the various geologic units, together with published data for similar materials.

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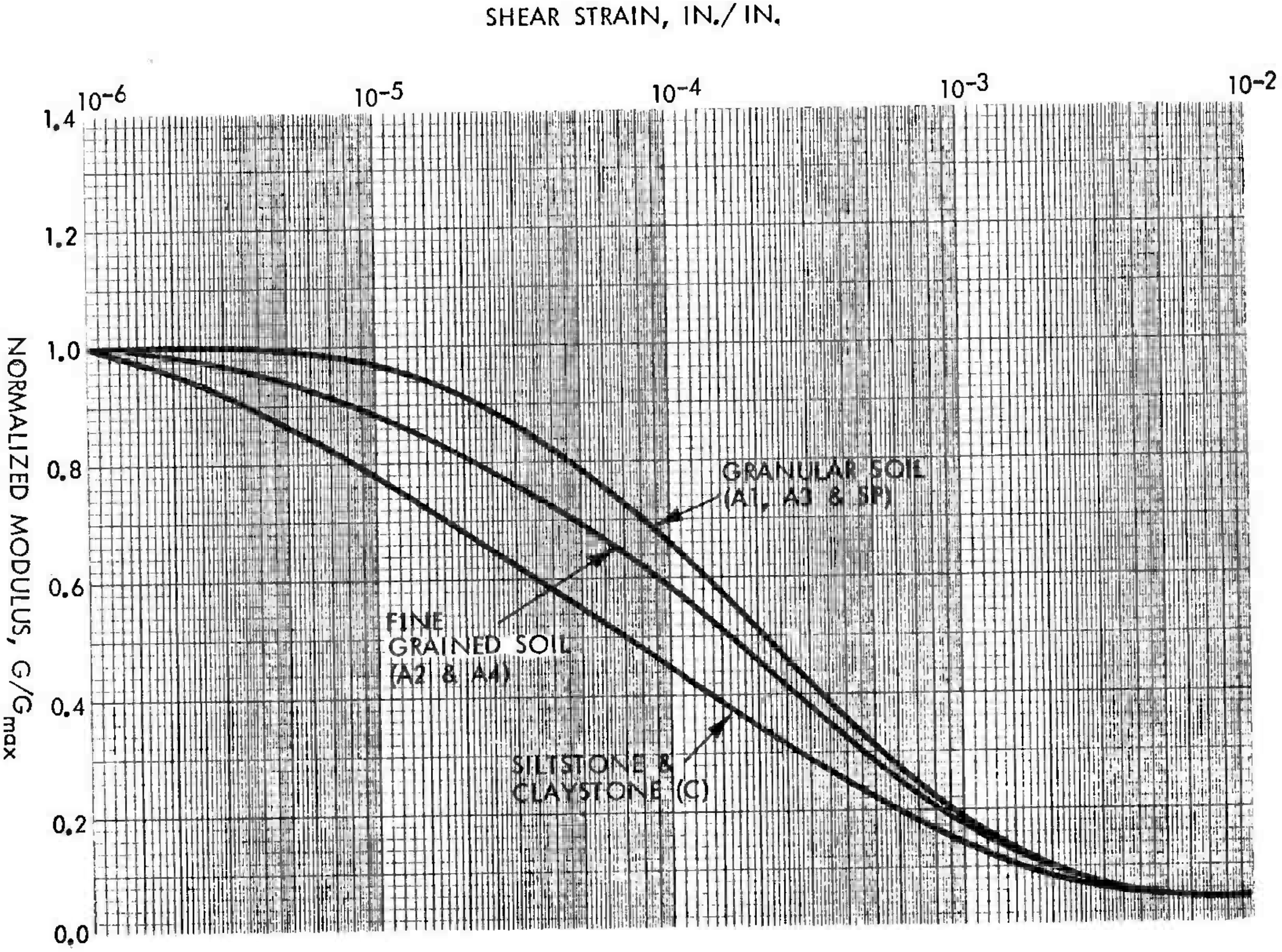

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RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
6-8

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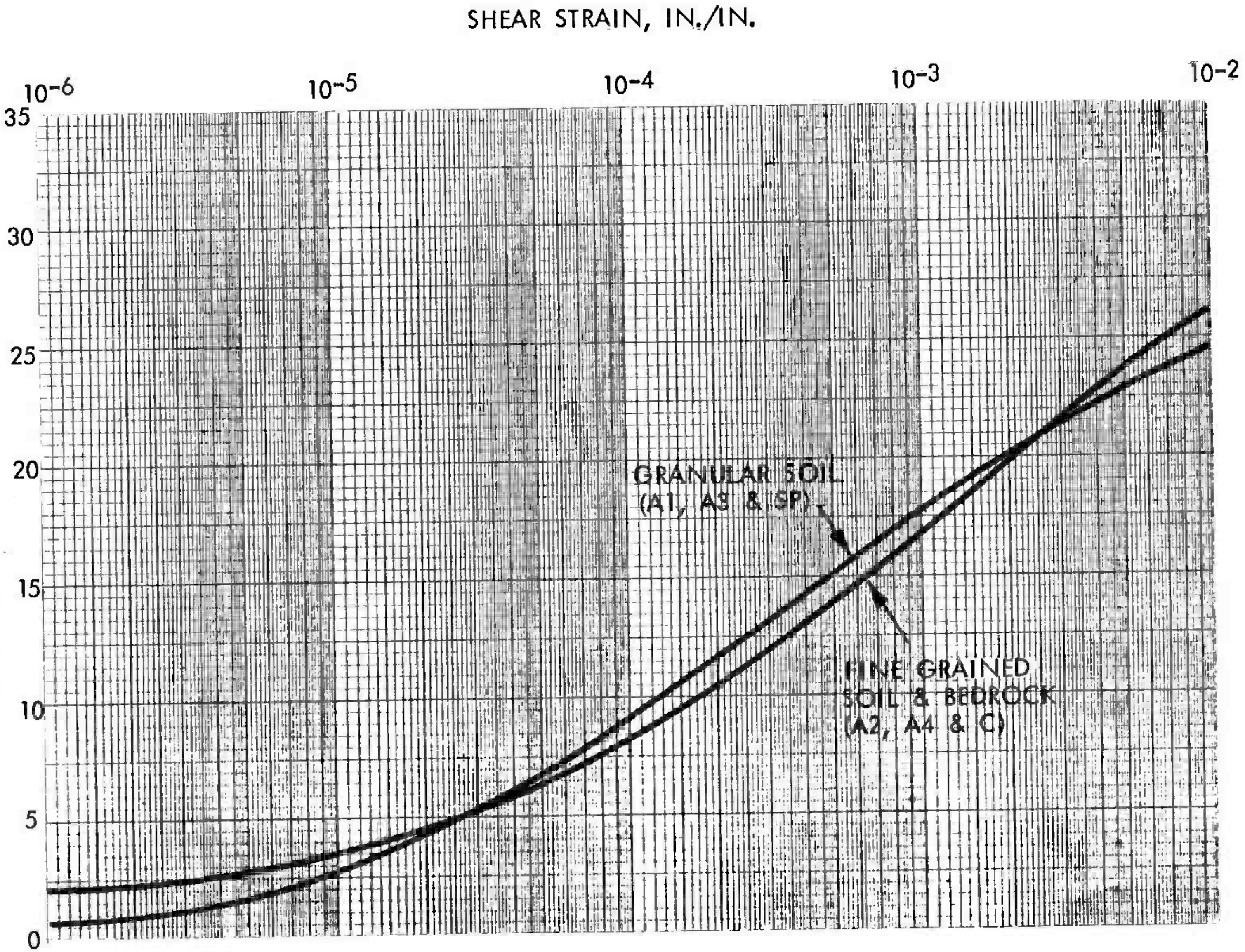

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RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No. 6-9

Table 6-2

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR SUBSURFACE MATERIALS FOR USE IN DESIGN

Property	Fine-Grained Alluvium	Coarse-Grained Alluvium	Topanga Formation
Average Compression Wave Velocity, V_p, ft/sec	850 (moist) 5,000 (sat.)	5,900	5,900
Average Shear Wave Velocity, V_s, ft/sec	700	1,100	1,200
Poisson's Ratio	0.40	0.35	0.40
Young's Modulus, E, psi	9,100 (moist) 330,000 (sat.)	565,000	450,000
Constrained Modulus, E_c, psi	19,500 (moist) 700,000 (sat.)	975,000	975,000
Shear Modulus, G_{max}, psi	13,500	34,000	40,000

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

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6.11.3 Liquefaction Potential

A generalized subsurface cross section at the Station site is shown in Drawing 2, and the subsurface conditions have been described in Section 5.0. The groundwater levels at the Station site are at depths ranging from 16 to 23 feet below the ground surface. The soils that are saturated and, therefore, must evaluated for liquefaction potential include the sandy materials within the fine-grained alluvial deposits, and the lower coarse-grained alluvium overlying the Topanga Formation.

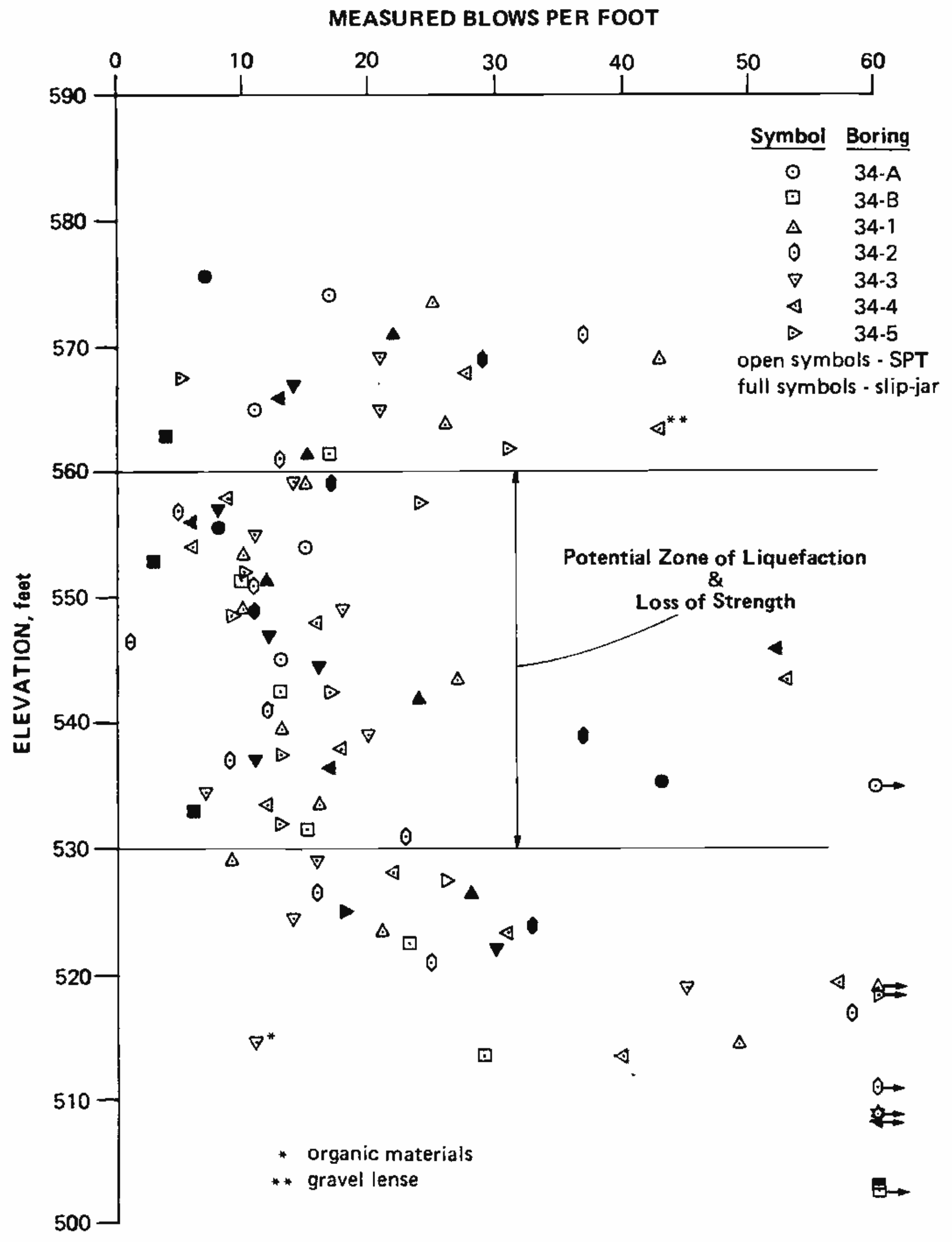
A plot of Standard Penetration Test (SPT) and slip-jar blow counts measured in the boreholes drilled at the Station site during the field investigations are plotted in Figure 6-10. The SPT blow counts were obtained by dropping a 140-lb hammer a distance of 30 inches above the ground surface, whereas the slip-jar is lowered by a cable inside the borehole. The slip-jar device weights about 340 lbs and is dropped over a distance of 24 inches. It is of interest to note from Figure 6-10 that the SPT and slip-jar blow counts recorded at the Station site agree reasonably well.

A review of the blow count data presented in Figure 6-10 indicates that within the fine-grained alluvium, between Elevations 560 and 530, the blow counts are generally less than 20 with most values in the range of 10 to 20. Because of the significant level of ground shaking which has been postulated at the Station from the maximum design earthquake, and the nature of the materials comprising the fine-grained alluvium, there exists a high potential for liquefaction and loss of strength within these materials. Above the groundwater table (Elevation greater than 560) the materials are not completely saturated and the blow counts are high enough that the potential for liquefaction is quite low in this zone. This is also true within the deeper coarse-grained alluvium since the SPT blow counts in these materials are generally greater than 25 blows per foot.

Comparison of gradations obtained from representative samples of the fine-grained alluvium, with soils considered susceptible to liquefaction (based on past historical occurrences), also appears to substantiate the high liquefaction potential for some of these materials (see Figure 6-11). This is the case even though some of the materials comprising the fine-grained alluvial deposits have percentages of fines generally greater than those which have undergone liquefaction during past earthquakes.

Values of shear wave velocities tabulated in Table 6-1 can also be used as an index for evaluating the liquefaction potential of saturated cohesionless soils. Based on correlations developed by Seed (1983) between average shear wave velocities, induced cyclic stress ratio and earthquake magnitude, the measured value of shear wave velocity of 1100 feet per second in the lower coarse-grained alluvium indicates that it is dense and would not be subject to liquefaction. The shear wave velocity of 700 feet per second corresponding to the fine-grained alluvium would appear to indicate potential liquefaction problems in these materials.

Based on our review of available data, the saturated fine-grained alluvium within Elevation 530 to 560 has a high potential for liquefaction and subsequent loss of strength during the postulated maximum design



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MEASURED BLOW COUNTS

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Figure No.
6-10

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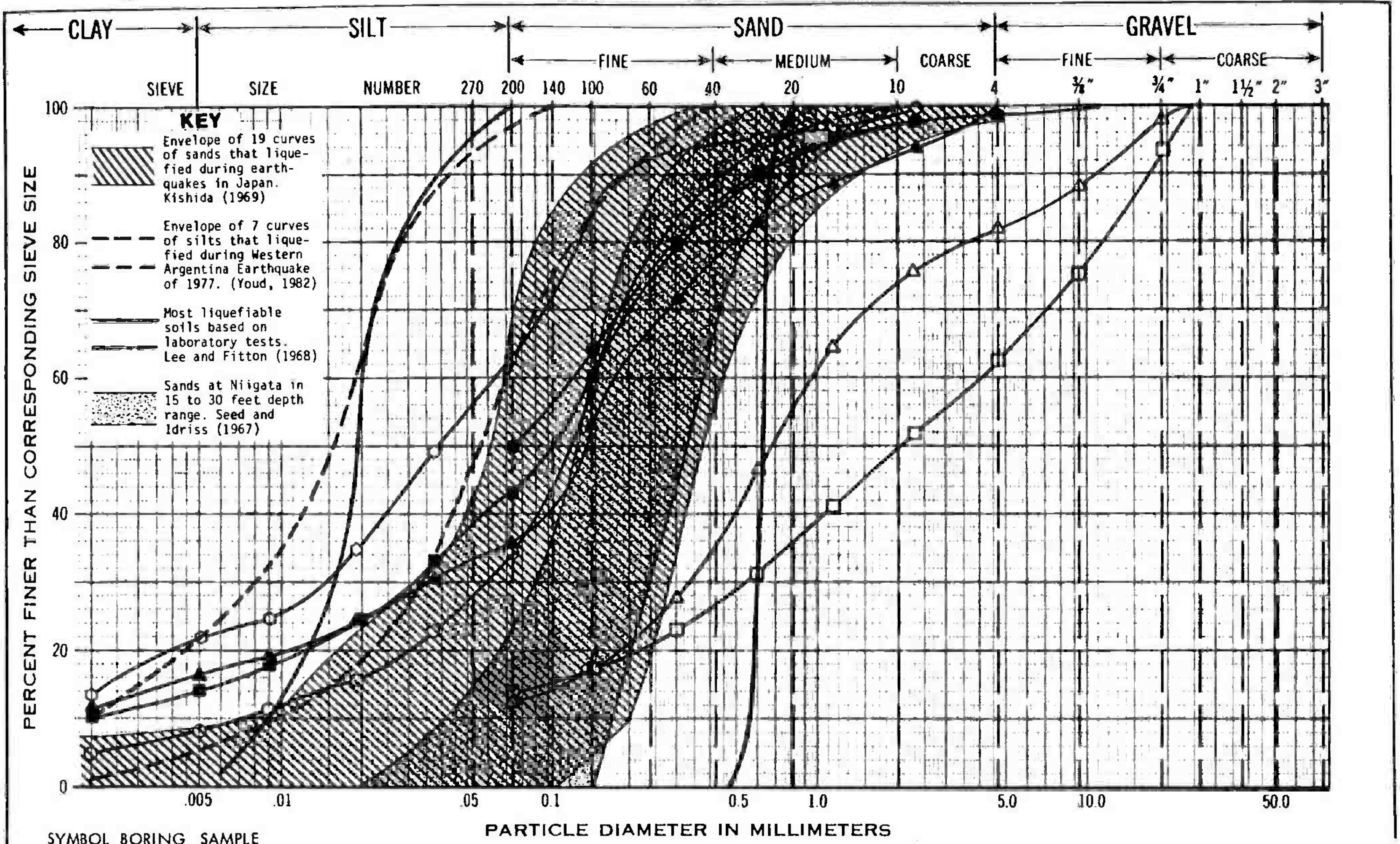

SYMBOL	BORING	SAMPLE
▲	34-1	PB-1
△	34-1	PB-5
●	34-2	PB-2
○	34-2	C-5
■	34-4	PB-1
□	34-4	PB-5
○	34-3	PB-3

COMPARISONS OF GRADATIONS

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Figure No.
6-11

earthquake. The coarse-grained alluvium overlying the bedrock at the site would not be subject to liquefaction due to its dense nature. These conclusions are based, in part, on procedures which are commonly employed to estimate the liquefaction potential of saturated cohesionless soil deposits (Seed et al., 1983), as well as other considerations and engineering judgment.

Since the base of the Station structure is founded within the Topanga Formation bedrock, it should perform satisfactorily during the maximum design earthquake. However, significant increases in lateral earth pressures could develop on the walls of buried structures due to liquefaction and/or loss of strength within zones of the fine-grained alluvium. In addition, some seismic compaction of the alluvium could occur due to dissipation of excess pore pressures after an earthquake which could result in differential settlement of shallow surface structures founded on these materials. The effects of liquefaction and loss of strength within portions of the fine-grained alluvium should be considered in the design of the permanent structures at the Universal City Station.

6.12 EARTHWORK CRITERIA

Site development at the Station site is expected to consist primarily of excavation for the subterranean structures but will also include general site preparation, foundation preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations and dewatering are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill are presented in Appendix G. Recommended specifications for compaction of fill are also presented in Appendix G. 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 fine-grained materials are not considered suitable because these materials will make compaction difficult and could lead to fill settlement problems after construction. If granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the soils engineer.

It should be understood that some settlement of the backfill will occur even if the fill soils are properly placed and compacted. Cracking and/or settlement of pavement on and around the backfilled excavations should be expected to occur for at least the first year following construction. Placement of the final pavement section should be delayed at least one year.

6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

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

- <u>Observation Well Monitoring</u>: The groundwater 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 groundwater levels. They will also provide valuable data to the contractor in determining his construction schedule and procedures.

- <u>Review Final Design Plans and Specifications</u>: 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.

- <u>Shoring Design Review</u>: Assuming that the shoring system is designed by the contractor, a qualified geotechnical engineer should review the proposed system in detail including review of engineering computations. This review would not be a certification of the contractor's plans but rather an independent review made with respect to the owner's interests.

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

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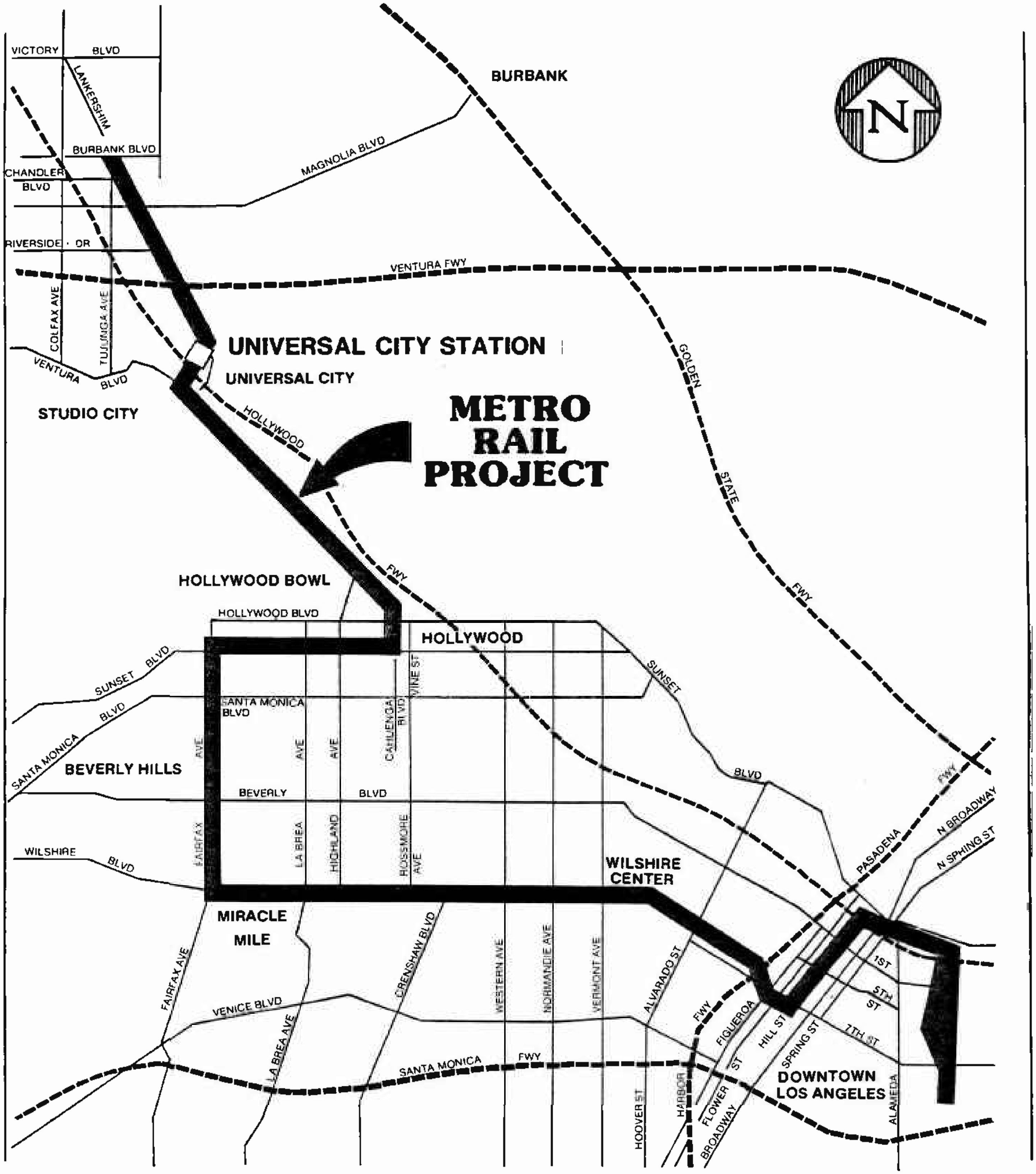
Drawings



VICINITY MAP

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Drawing No
1

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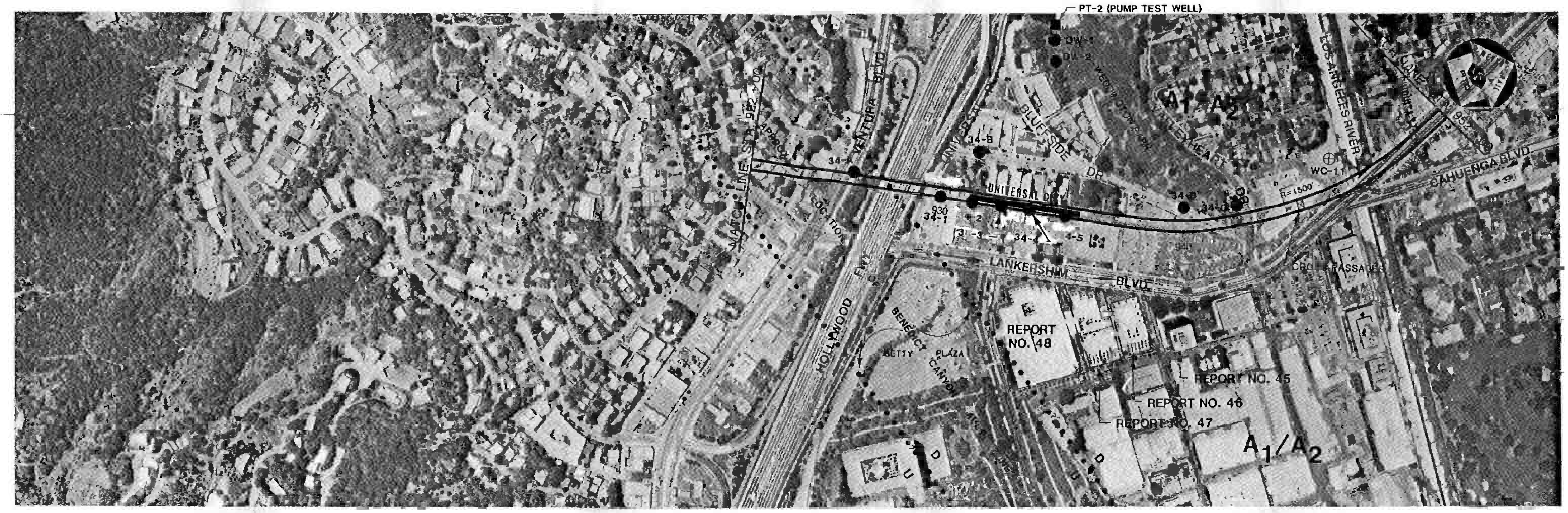
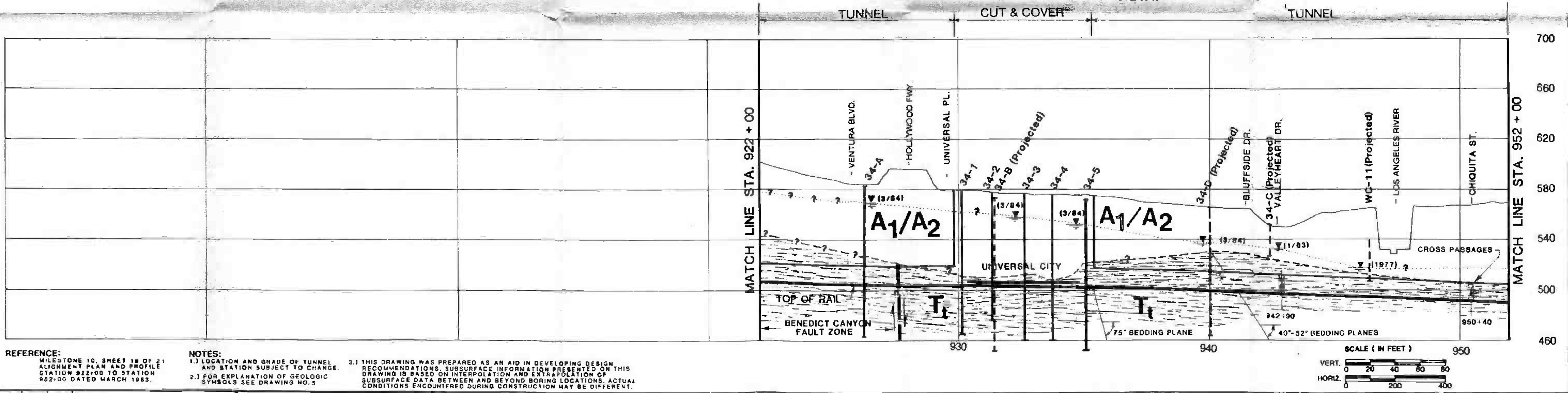


PLAN



REFERENCE:
MILESTONE 10, SHEET 18 OF 21
ALIGNMENT PLAN AND PROFILE
STATION 922+00 TO STATION
952+00 DATED MARCH 1983.

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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DATE

SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted RM Price Date 5-14-84

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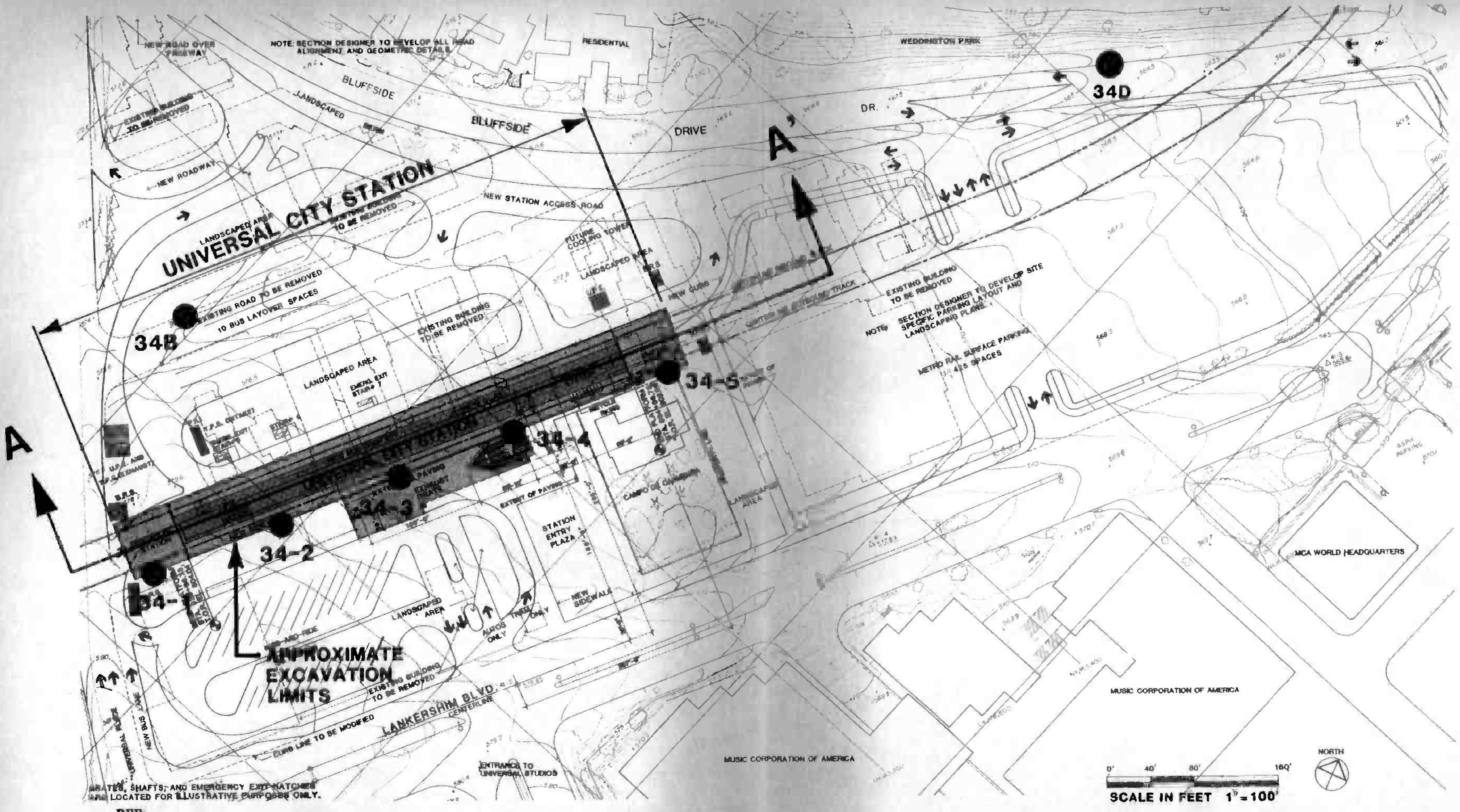
GENERAL CONSULTANTS
APPROVED

DESIGN UNIT A425
LOCATION OF BORINGS AND GEOLOGIC SECTION

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



REF:

"PRELIMINARY UNIVERSAL CITY STATION SITE PLAN", DRAWING #A-64, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40', REDUCED TO 1" = 100', DATED 3-15-83.

NOTES: 1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 4

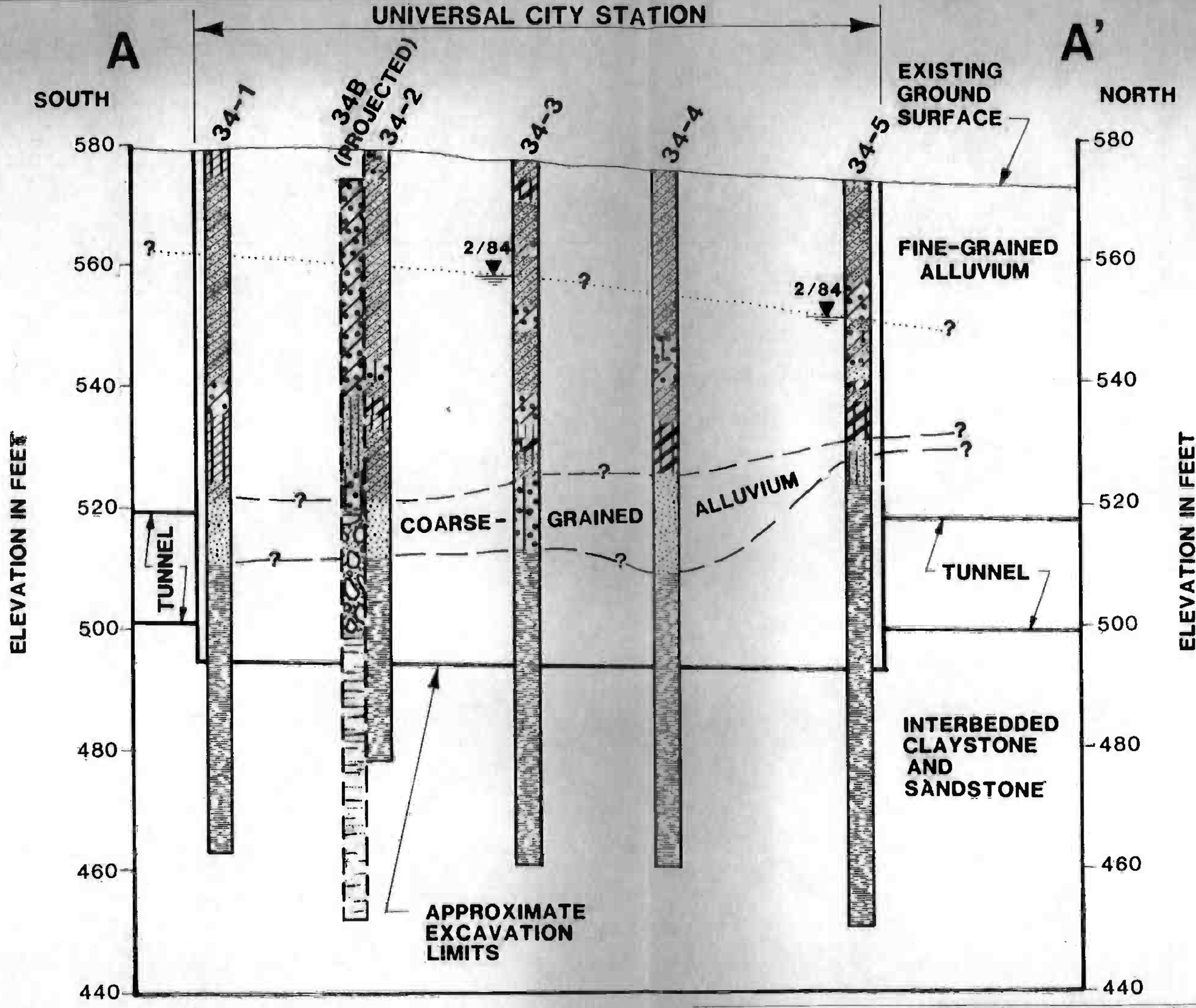
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

LOCATION OF BORINGS

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

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



SCALE: HOR. 100 0 100 200

VER. 20 0 20 40

scale feet

NOTES:

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

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

SUBSURFACE SECTION A-A'

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale As Shown
Date MAY, 1984
Prepared by CSJ
Checked by RG
Approved by JAD

Project No 83-1140
Drawing No 4

Form D60/78

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

QUATERNARY	HOLOCENE	A_1	YOUNG ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, sandy gravels, and locally contains cobbles and boulders. Primarily dense, but ranges from loose to very dense.
		A_2	YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.
	PLEISTOCENE	A_3	OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.
		A_4	OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from firm to hard.
		SP	SAN PEDRO FORMATION: Predominantly clean, cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium dense to very dense. Locally impregnated with oil or tar.
TERTIARY	PLIOCENE	C	FERNANDO AND PUENTE FORMATIONS: Claystone, siltstone, and sandstone; thinly to thickly bedded. Primarily low hardness, weak to moderately strong. Locally contains very hard, thin cemented beds and cemented nodules.

ROCK TUNNELLING
(Terzaghi Rock Condition Numbers apply)*

TERTIARY		3	Terzaghi Rock Condition Number
		⌇	Approximate boundary between Terzaghi numbers
	MIOCENE	2-5	TOPANGA FORMATION: Conglomerate, sandstone, and siltstone: thickly bedded; primarily hard and strong (Geologic symbol Tt).
		1-5	TOPANGA FORMATION: Basalt; intrusive, primarily hard and strong (Geologic symbol Tb).

TERZACHI ROCK CONDITION NUMBERS:*

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

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

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

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

SYMBOLS

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

NOTES:

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

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

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

▥	SILT
▨	CLAY
▥	SANDY SILT
▨	SANDY CLAY
▥	CLAYEY SILT
▨	SILTY CLAY
▦	SILTY SAND
▩	CLAYEY SAND
⛆	SAND
▩	GRAVELLY SAND
▩	SANDY GRAVEL
° °	GRAVEL
▨	GRAVELLY CLAY
≋	TAR SILT & CLAY
≡	TAR SAND
▩	FILL
▭	SILTSTONE
▤	CLAYSTONE
☰	INTERBEDDED SANDSTONE WITH SILTSTONE OR CLAYSTONE
⁙	SANDSTONE
° °	SANDSTONE, CONGLOMERATE
xxxxxxx	CEMENTED ZONE
▩	META—SANDSTONE
⸪	BASALT
∴	BRECCIA
▤	SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

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

Appendix A

Field Exploration

APPENDIX A   FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A425 include information obtained from borings drilled for this and previous geotechnical investigations. Table A-1 summarizes pertinent information on 10 exploratory boreholes that have been drilled at, or in relative close proximity to, the proposed Universal City Station site. The locations of all boreholes listed in Table A-1, except for CEG-34, are shown on Drawing No. 2. Boring CEG-34 was drilled during the 1981 geotechnical investigation and is located about 1300 feet northwest of the present Station site. The log of this hole is included at the end of this appendix because the information provided in the log of this borehole has been judged to be generally representative of the subsurface conditions that exist at the Station site. The location of Boring CEG-34 is shown on Drawing No. 1 of the 1981 geotechnical investigation report.

Of the 10 borings that have been drilled at or near the Universal City Station site, 9 are rotary wash type borings and 1 is a large-diameter or "man-size" auger hole. One rotary wash boring was drilled as part of the 1981 geotechnical investigation, three were drilled in January and February 1983, and 5 borings were drilled for this investigation during October and November of 1983. The large-diameter borehole was drilled in January 1983. Edited field logs for the borings listed in Table A-1 are included at the end of this appendix.

Groundwater observation wells (piezometers) were installed in 5 of the borings drilled at or near the Station site (see Table A-1). Groundwater samples were not obtained from any of the borings listed in Table A-1. Consequently, chemical analyses were not performed on water samples obtained from the Station site. Oil slicks appeared on the drilling fluid in four of the borings listed in Table A-1. Strong organic and/or sulfur odors were also noted in the logs of some of the boreholes.

Most rotary wash borings were sampled at regular intervals using the Converse ring sampler, Pitcher Barrel sampler, and the Standard Split Spoon (SPT) sampler. Soil sample recovery was sometimes poor in the soils encountered at or below groundwater. Bedrock core recovery was generally good. The large-diameter or "man-sized" auger hole was logged by a downhole observer(s); however, soil samples were not obtained from this hole.

The following subsections describe the field exploration procedures and provide explanations of symbols and notations used in preparing the field boring logs. Copies of the edited field boring logs follow the text of this appendix.

TABLE A-1
BORING LOG SUMMARY
DESIGN UNIT A425

BORING NUMBER	DATE DRILLED (Mo/Yr)	TYPE[1]	GROUND[2] SURFACE ELEVATION (ft.)	TOTAL DEPTH (ft.)	PIEZOMETER		WATER SAMPLE TESTED	OIL AND/OR NATURAL GAS	COMMENTS
						INSTALLED DEPTH (ft.)			
CEG-34[3]	12/80	RW	574	200.0	No	–	No	No	Downhole & Crosshole
34A	2/83	RW	586	120.3	Yes	0.0-120.0	No	Yes[4]	
34B	1/83	RW	574	121.0	Yes	0.0-120.0	No	No	
34C	1/83	LD	552	26.0	No	–	No	No	Caving & Raveling Strong organic odor
34D	1/83	RW	565	101.0	Yes	0.0-101.0	No	No	
34-1	11/83	RW	580	114.5	No	–	No	No	
34-2	11/83	RW	579	100.5	No	–	No	No	
34-3	10/83	RW	577	115.5	Yes	3.0-115.0	No	Yes[4]	
34-4	10/83	RW	575	114.0	No	–	No	Yes[4]	
34-5	10/83	RW	573	122.6	Yes	0.0-120.0	No	Yes[4]	

NOTES: (1) Types – RW: Rotary wash boring (small diameter).
LO: Large diameter auger boring (36 diameter).

(2) Ground surface elevations approximate and rounded to nearest foot.

(3) Boring drilled about 1300 feet from proposed station site.

(4) Oil slick in drilling mud suggesting bedrock may be petroliferous.

A.2 ROTARY WASH BORINGS

A.2.1 Technical Staff

Members of three firms (CWDD/ESA/GRC) participated in the drilling exploration program. The field geologist continuously supervised each rotary wash boring during the drilling and sampling operation. The geologist was also responsible for preparing a detailed lithologic log of the rotary wash cuttings 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

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.

A.2.3 Sampling and Logging Procedures

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

As indicated in Table A-1, Boring CEG-34 was drilled during the 1981 geotechnical investigation. The soils encountered in this boring were sampled about every 5 feet using a Standard Split Spoon (SPT) sampler driven with a standard 30-inch stroke, 140-pound hammer. At about each 20-foot interval and prior to the SPT sampler, an undisturbed Converse ring sample was obtained using a downhole slip-jar hammer.

When bedrock was first encountered, the SPT and Pitcher Barrel samples were used and samples were taken at intervals of about 5 feet. Below a depth of about 116 feet, the bedrock was almost continuously sampled using either the Pitcher Barrel or NX core barrel. The choice of using the Pitcher Barrel or NX core barrel was made during drilling depending on the ground conditions encountered.

Three rotary wash borings (Borings 34A, 34B, and 34D) were drilled near the Universal City Station site during January and February 1983. The purpose of these borings was to provide supplemental geotechnical information for this Station site and along the tunnel alignment just north and south of the site. Soils were sampled about every 10 feet with the SPT, Converse ring, and Pitcher Barrel samplers.

Five rotary wash borings were drilled at the Station site during the months of October and November of 1983. Borings 34-1 through 34-5 were drilled to depths ranging between 101 and 123 feet. With the exception of Boring 34-5, all soils encountered in the borings were sampled at about 10-foot intervals using the Converse ring sampler. Between this interval and at about every 10 feet, Pitcher Barrel samples were taken and were followed by the SPT sampler. SPT samples were also taken between the Converse ring and Pitcher barrel samplers. In Boring 34-5, the Pitcher Barrel sampling techniques were utilized at intervals of about 20 feet and the soils were sampled, on the average, twice every 10 feet by the SPT sampler.

When bedrock was encountered in Borings 34-1 through 34-5, the Pitcher Barrel sampler was generally used every 10 feet. Converse ring and/or SPT samples were also taken between this interval when it was judged that these methods would be successful in retrieving samples.

All of the sampling intervals described above were sometimes altered during the course of the drilling operations if a change in material types was detected by the geologist logging the hole or if sample recovery of the previous soil sample was poor. As was previously mentioned, some of the soils encountered at or below the groundwater level at the site tended to fall or pull out of the sampler as it was being brought to the ground surface. Another common cause for loss of samples or altering the sampling interval was when gravels were 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 cuttings suggested a change in formation.

The sampling program was also sometimes modified when dense soil deposits were encountered. In this case, the Converse ring sampler was not used. Instead, the Pitcher Barrel sampler, which is generally a better technique when sampling dense soil deposits, was substituted for the Converse ring sampler in order to obtain higher quality undisturbed samples.

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 LARGE-DIAMETER BORINGS

A.3.1 Technical Staff

Personnel of Converse Consultants, Inc. (Converse, 1983) directed the drilling and performed the logging of Boring 34C which was a large-diameter or "man-size" borehole. Since the purpose of the large-diameter auger borings was to allow consultants and RTD personnel to make first-hand downhole observations of the geologic conditions along the proposed project route, a number of people participated in this exploration program. They include personnel from the Southern California Rapid Transit District, MRTC, Lindvall Richter & Associates, and other independent consultants.

A.3.2 Drilling Contractor and Equipment

Drilling was performed by A&W Drilling Company of La Habra using a bucket auger drilling rig with a 36-inch bucket.

A.3.3 Drilling Operations

These operations consisted of drilling the auger boring to a depth of 26 feet. Drilling was stopped when a significant inflow of water occurred at 21 feet and the hole started to experience caving. Corrugated metal pipes (sections 20 feet long) with windows cut on 5-foot vertical intervals were used to case the hole. The windows were 1-foot square and permitted observations of material types, caving, groundwater, and gas/oil conditions. Casing was installed over the total open depth of the hole.

Before entering the hole, a "gas detector" meter was used to evaluate the lack of oxygen and/or the presence of combustible gases. The borings were then logged by personnel of Converse Consultants prior to any other observers entering the hole. Loggers and all observers were equipped with safety equipment as required by the California Occupational Safety and Health Administration.

A.4 FIELD CLASSIFICATION OF SOILS

All soil types were classified in the field by the site 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.

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

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

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

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

A.5 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 .
- Mineralogy, textural, and structural features.
- Any other distinctive features which aid in correlating or interpreting the geology.

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

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

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

TABLE A-3 Bedrock Description Terms

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

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

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

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

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

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

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

BORING LOG CEG 34

Proj: DESIGN UNIT A425
Date Drilled 12/2-8/80
Ground Elev. 574'
Drill Rig Failing 1500
Logged By S. Testa
Total Depth 200.5'
Hole Diameter 4 7/8"
Hammer Weight & Fall DR: 240 lb @ 18", SS: 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
0		0.0-0.5 CONCRETE			RD	12/2/80 clear day hole drilled with water
	ML	ALLUVIUM 0.5-34.0 SANDY SILT: dark yellowish brown; fine sand; moist				
2						
4						
6						
8						
10		yellow brown; stiff; moist				
			J-1		SS	1.0/1.5 recovery
12					RD	
14						
						pocket penetrometer 2.0 tsf 2/9/81 1.2/1.5 recovery
16		becomes very stiff; moist; trace gravel	J-2		SS	
					RD	
18						
20						

Sheet 1 of 9

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
20	ML	0.5-34.0 <u>SANDY SILT</u>: (continued)	C-1		DR	1.5/1.5 recovery
22			J-3		SS	1.5/1.5 recovery
24					RD	12/4/80 moderate to heavy rain hole drilled with bentonite drilling fluid
26		color change to dusky brown; trace fine gravel; stiff; fine roots	J-4		SS	1.5/1.5 recovery
28					RD	
30		color change to dark yellow brown; gravel to 1.5"; fine roots	C-2		DR	1.5/1.5 recovery
32		stiff	J-5		SS	1.5/1.5 recovery
					RD	
34	SM	34.0-38.5 <u>SILTY SAND</u>: dark yellowish brown; fine grained; dense				
36			J-6		SS	1.3/1.5 recovery
					RD	
38	SM	38.5-44.0 <u>GRAVELLY SAND</u>: pale yellow brown; medium to coarse sand; dense to very dense				minor rod chatter, from 38.5 to 40.0'
40			J-7		SS	1.0/1.5 recovery
42					RD	
44						rod chatter at 44.0'

Sheet 2 of 9

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
44	GP	44.0-50.5 GRAVEL: subangular to subrounded to 1.5"; poorly graded			RD	
			J-8		SS	.25/1.0 recovery refusal at 11.5"
46					RD	
48						considerable rod chatter from 48.0-50.0'; hard drilling resistance at 49.0'
50			C-3		DR	0.3/1.0 recovery refusal at 12"
		TOPANGA FORMATION - BEDROCK 50.5-200.5 INTERBEDDED SHALE AND SANDSTONE: medium to very thin laminae of primarily olive gray sandy clay and subordinate dark greenish gray; fine to medium sand with lesser very thin laminae of siltstone; trace organics; thin sand lenses				
52			J-9		SS	1.5/1.5 recovery
54						
		55.0' color change-olive black Physical Conditions: little fractured; low hardness; friable to weak strength; fresh	J-10		SS	1.5/1.5 recovery pocket penetrometer >4.5 tsf 2/9/81
56					RD	
58						
60		color change to dark greenish gray				
	10-15°		J-11		SS	pocket penetrometer 4.0 tsf (broke apart) 2/9/81
62					RD	
64						
		color change to olive gray; trace organics; becoming sandier with depth	J-12		SS	pocket penetrometer >4.5 tsf 2/9/81 refusal at 15"
66					RD	
68						

Sheet 3 of 9

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
68		50.5-200.5' INTERBEDDED SHALE AND SANDSTONE: (continued)			RD	
70	0-15°		PB-1	1	PB	pocket penetrometer >4.5 tsf 2/9/81 2.3/2.5 recovery rod chatter from 72.0 to 75.0'
72		olive gray; fine to medium sand; trace fines			RD	
74						
76		color change to dusky brown; sandy claystone with very thin to thin laminae of dark greenish gray sand	J-13		SS	1.3/1.3 recovery
					RD	
78						
80		color change to dusky yellowish brown	J-14		SS	0.7/1.0 recovery
82	60°					
84						rod chatter from 84.0 to 85.0'
			J-15		SS	0.9/1.0 recovery
86	15-20°	non-parallel medium to very thin laminae; dark greenish gray fine sand from 85.0-85.3; dusky brown sandy claystone from 85.3 to 85.8; dark greenish gray sand from 85.8 to 85.9'; hard			RD	
88						
90			PB-2	2	PB	2.1/2.5 recovery pocket penetrometer >4.5 tsf 2/9/81
92						

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
92		50.5-200.5 INTERBEDDED SHALE AND SANDSTONE: (continued)	PB-2	2	PB	
					RD	
94		Physical Conditions: (continued) little fractured; friable to low hardness; friable to weak strength; fresh				0.4/0.4 recovery
			J-16		SS	pocket penetrometer >4.5 tsf 2/9/81
96		color change to olive black; 95.2' very thin sandy claystone lens			RD	12-5-80
98						
100						
		101.0-101.4 well cemented sandstone	PB-3	3	PB	pocket penetrometer >4.5 tsf 2/9/81 1.4/2.5 recovery
102						
					RD	
104						
		non-parallel medium to very thin laminae of olive black sandy claystone and greenish gray friable fine to medium sandstone	J-17A J-17B		SS	pocket penetrometer >4.5 tsf 2/9/81 1.3/1.4 recovery ~~refusal at 16.5"~~
106						
					RD	
108						
110	45°					
			PB-4	4	PB	pocket penetrometer >4.5 tsf 2/9/81 2.5/2.5 recovery
112						
	80°	dusky yellowish brown sandy claystone	PB-5	5	PB	2.0/2.5 recovery
114						
	45°		PB-6	6	PB	2.5/2.5 recovery
116						

Sheet 5 of 9

Project DESIGN UNIT A425 **Date Drilled** 12/2-8/80 **Hole No.** CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
116		50.5-200.5 INTERBEDDED SHALE AND SANDSTONE: (continued)	PB-6	6	PB	pocket penetrometer >4.5 tsf 2/9/81 2.5/2.5 recovery
118		117.5-118.4 thin to very thin alternating shale and sandstone laminae; from 119.0 olive black shale	PB-7	7	PB	1.8/2.5 recovery
120		121.4-122.4 moderate to well cemented sandstone	PB-8	8	PB	2.5/2.5 recovery
122 124		Physical Conditions: (continued) little fractured primarily along bedding planes; low hardness; friable to weak strength; fresh	PB-9	9	PB	2.2/2.5 recovery
	10-15°		PB-10	10	PB	1.5/1.5 recovery
126					RD	variable resistance from 126-126.5'; refusal at 126.5'
128		medium to very thin laminae of alternating shale and subordinate sandstone	PB-11	11	PB	2.0/2.5 recovery pocket penetrometer >4.5 tsf 2/9/81
130			PB-12	12	PB	2.5/2.5 recovery
132		well cemented sandstone from 131.0-132.0, 133.2 to 133.5	PB-13	13	PB	12-6-80 0.5/0.5 recovery
134					RD	133.0-133.5' rod chatter
	10-15°					
136		shale from 137.0 to 137.5'	PB-14	14	PB	2.5/2.5 recovery
			PB-15	15	PB	0.2/0.2 recovery
138			PB-16	16	PB	2.1/2.1 recovery
140						

Sheet 6 of 9

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
140		50.5-200.5 INTERBEDDED SHALE AND SANDSTONE: (continued) alternating medium to very thin laminae of shale and subordinate sandstone	PB-17	17	PB	2.5/2.5 recovery
142		shale from 143.0 to 145.0'	PB-18	18	PB	2.5/2.5 recovery minor rod chatter from 143.5 to 145.0'
144			PB-19	19	PB	pocket penetrometer >4.5 tsf 2/9/81 1.5/1.5 recovery
146			PB-20	20	PB	0.1/0.1 recovery
					RD	moderate rod chatter; resumed pitcher sampling at 147.5-150.0'
148	10-15°	friable sandstone from 147.7 to 148.1'	PB-21	21	PB	2.5/2.5 recovery
150		sandstone from 150.5 to 150.8'; 152.0 to 152.5'	PB-22	22	PB	0.8/0.8 recovery moderate rod chatter 150.8 to 151.8'
					RD	
152			PB-23	23	PB	pocket penetrometer >4.5 tsf 2/9/81 2.5/2.5 recovery
154		154.5' well cemented sandstone			RD	
156		Physical Condition: (continued) little fractured; low hardness; friable to weak strength; fresh	PB-24	24	PB	2.5/2.5 recovery
158		shale from 158.2 to 158.8', 159.2 to 160.6'	PB-25	25	PB	moderate rod chatter 2.5/2.5 recovery
160	50°		PB-26	26	PB	2.5/2.5 recovery
162						2.0/2.5 recovery
164			PB-27	27	PB	

Sheet 7 of 9

Project DESIGN UNIT A425 Date Drilled 12/2-8/80 Hole No. CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
164		50.5-200.5 <u>INTERBEDDED SHALE AND SANDSTONE:</u> (continued) alternating medium to very thin laminae of shale and subordinate sandstone with lesser very thin siltstone laminae.	PB-27	27	PB	
166		sandstone from 167.0 to 168.0', 169.5 to 170.0'	PB-28	28	PB	2.5/2.5 recovery moderate rod chatter from 167.0-168.0' pocket penetrometer >4.5 tsf 2/9/81
168	10-20°	Physical Condition: little fractured along bedding planes; friable to low hardness; friable to weak strength; fresh	PB-29	29	PB	2.5/2.5 recovery
170					RD	considerable rod chatter
172			PB-30	30	PB	0.2/0.2 recovery
					RD	
174		174.0-174.4' medium to coarse sandstone	PB-31	31	PB	12-7-80 0.8/1.0 recovery moderate rod chatter
			PB-32	32	PB	0.9/1.0 recovery
176		shale from 175.6 to 177.0'	PB-33	33	PB	1.5/1.5 recovery considerable rod chatter
					RD	
178	10-20°		PB-34	34	PB	2.5/2.5 recovery
180		180.6 to 181.2' well cemented sandstone	PB-35	35	PB	0.6/0.6 recovery
					RD	
182		well cemented sandstone 181.5 to 182.4', 183.0 to 183.2'	PB-36	36	PB	0.4/0.4 recovery
					RD	
184	46° 40°	184.0 to 200.0' alternating shale, sandstone and subordinate siltstone; siltstone - moderate brown, sandstone - olive gray very thin to thick parallel laminae, shale - dusky yellowish brown to olive black very thin to medium parallel laminae	PB-37	37	PB	
186	80° 80°		Box #1	1	C	
188						

Project DESIGN UNIT A425 **Date Drilled** 12/2-8/80 **Hole No.** CEG 34

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	RUN NO.	DRILL MODE	REMARKS
188	15-20°	50.5-200.5 INTERBEDDED SHALE AND SANDSTONE: (continued) very thin shale laminae from 188.0 to 188.7'	Box #2	1	C	4.8/5.0 recovery
190	45°	Physical Condition: little fractured; friable to low hardness; friable to weak strength; fresh		2		4.2/4.2 recovery
192	15-20°	189.0-190.8' dusky brown very thin shale laminae				
194		194.6 to 197.6 dark greenish gray; fine to coarse sandstone	Box #3	3		5.0/5.0 recovery
196	55°					
198	75°	very thin shale laminae from 198.0 to 198.5' ; very thin fine sandstone from 198.5 to 199.7'		4		12-8-80 1.5/1.5 recovery
200		B.H. 200.5' Terminated hole				Installed 100.0' of 4" PVC and grouted; pushed 3.0' of 6" ID PVC ½ below sidewalk surface, steel water cover was then set flush with concrete surface.
202						
204						
206						
208						
210						
212						

Sheet 9 of 9

Converse Consultants, Inc.
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BORING LOG 34-1

Proj: DESIGN UNIT A 425
Date Drilled 11/5-6/83
Ground Elev. 580'
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 114.5'
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.3 ASPHALT			RD	start drilling 4:15
	RD	0.3-0.5 BASEROCK				
	ML	ALLUVIUM 0.5-4.0 CLAYEY SILT:dusky brown; low to moderately plastic fines; trace fine sand; firm; moist				
2						
4	CL	4.0-37.6 SANDY CLAY: moderate brown, moderate plastic fines, trace fine sand; very stiff; moist increased sand content at 5'	J-1	5 11 14	SS	Recovery 1.2/1.5
6					RD	set up tub and cased to ~ 5' mixed mud
			C-1	9 13	DR	Recovery 1.0/1.0
8						
		hard			RD	
			J-2	10 18 25	SS	Recovery 1.0/1.5 5:30 11-5-83
10					RD	7:00 11-6-83
12		some medium to coarse sand	PB-1		PB	Recovery 1.8/2.5 pocket pen: 3.25 tsf
14	(SC)	14.5' clayey sand				
			J-3	7 11 15	SS	Recovery 1.4/1.5
16					RD	
		increased content of fine sand	C-2	6 9	DR	Recovery 1.0/1.0
18					RD	
				5	SS	Recovery 0.0/1.5
20		stiff		6		

Project DESIGN UNIT A 425 **Date Drilled** 11/5-6/83 **Hole No.** 34-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	4.0-37.6 SANDY CLAY: (continued)		9	SS	
					RD	
22		sand content decreases with depth	PB-2		PB	Recovery 2.4/2.5
24			J-6	2 5 5	SS	Recovery 1.5/1.5
26					RD	
			C-3	5 7	DR	Recovery 1.0/1.0
28					RD	
				0 2 8	SS	1st 10"-weight of hammer Recovery 0.0/1.5
30		becomes soft to firm			RD	
32			PB-3		PB	Recovery 2.5/2.5
34	(SM)	34.0-34.3 silty sand lens	J-6	6 9 18	SS	Recovery 1.0/1.5
		35.0-silty clay lens; very stiff				
36					RD	
			C-4	7 17	DR	Recovery 1.0/1.0
38	SC	37.6-43.0 CLAYEY SAND: dark yellowish brown; fine to coarse sand; occasional gravel; medium; dense; moist to wet			RD	
			J-7	6 5 8	SS	Recovery 0.3/1.5
40					RD	
42	(SM)	silty sand lens	PB-4		PB	
	ML	43.0-54.0 CLAYEY SILT: dusky yellowish brown (see next page)				
44						

Sheet 2 of 5

Project DESIGN UNIT A 425 Date Drilled 11/5-6/83 Hole No. 34-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	43.0-54.0 CLAYEY SILT: (continued) low plastic fines; trace fine sand medium dense; moist		[illegible]	PB	
			J-8	2 6 10	SS	Recovery 1.5/1.5
46					RD	
48		becomes loose				
				0 1 8	SS	weight of hammer for 11" Recovery 0.0/1.5 sample pulled out
50					RD	
52	(ML) (SM)	52.0-53.0 sandy silt/silty sand lens increased clay content with depth	C-5	11 17	DR	Recovery 1.0/1.0
					RD	
54	CL	54.0-57.5 SANDY CLAY: olive grey; moderate plastic; fines; fine sand; very stiff; moist	J-9	2 5 16	SS	Recovery 0.6/1.5
56	(SC)	grades to clayey sand with depth contains some gravel			RD	
58	SP/ SM	57.5-68.0 SAND/SILTY SAND: salt and pepper; fine sand; occasional gravelly sand lenses; very dense; wet some bedding apparent	J-10	18 34 38	SS	Recovery 1.0/1.5
60					RD	
62			PB-5		PB	Recovery 0.9/1.8
	(CL)	63.8- sandy clay/silty sand in tip of sample				
64	(SC)	63.8-65.3-clayey sand; sand; gravel	J-11	45 31 18	SS	Recovery 0.5/1.5
66		6" cobble			RD	
68						

Sheet 3 of 5

Project DESIGN UNIT A 425 **Date Drilled** 11/5-6-/83 **Hole No.** 34-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		TOPANGA FORMATION 68.0-114.5 <u>INTERBEDDED CLAYSTONE & SAND-STONE</u>: brownish black; fe stained mottled; contains sand lenses of varying thickness; occasional cemented zones; steeply dipping ~ 70°			RD	
			J-12	35 33 33	SS	
70						Recovery 0.7/1.5
					RD	
72		physical condition: little fractured; friable to low hardness; friable to weak strength; little weathered		55	DR	Recovery 0.7/0.9
			C-6	90-5	"	
					RD	
74						
76						
78		thinly bedded 1/8"- 1/4"				
			J-13	54	SS	Recovery 0.5/0.5
					RD	
80						
82		becomes massive				
			PB-6		PB	Recovery 1.1/2.5
84						
					RD	
86						
88		interbedded; weakly to moderately cemented		78	DR	
			C-7	80-2.5"		Recovery 0.6/0.7
					RD	
90						
92						

Sheet 4 of 5

Project DESIGN UNIT A425 Date Drilled 11/5-6/83 Hole No. 34-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		68.0-114.5 <u>INTERBEDDED CLAYSTONE & SANDSTONE:</u> (continued)	PB-7		PB	Recovery 1.8/2.5
94					RD	
96						
98				66		
			C-8	100-4.5"		Recovery 0.8/0.9
100					RD	occasional chatter
102						
104			B-1		PB	tube smashed; cut thin sample ~ 1½" d x 7" long
		interbedded: slicken sides on some fracture surfaces in massive claystone			RD	
106						occasional chatter
108						
110						
112			PB-8		PB	Recovery 2.1/2.5
114						
	BH	114.5 Terminated Hole				Complete drilling 3:30 tremied grout to surface
116						

Sheet 5 of 5

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BORING LOG 34-2

Proj: DESIGN UNIT A425
Date Drilled 11/5/83
Ground Elev. 579'
Drill Rig Failing 1500
Logged By L. Schoeberlein
Total Depth 100.5'
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			RD	start drilling 7:30
	GP	0.5-0.7 BASEROCK				
	SM	ALLUVIUM 0.7-1.5 SILTY SAND: greyish brown; fine sand				
2	CL	1.5-3.5 SANDY CLAY: greyish brown; moderately plastic fines; trace fine sand; occasional organics; very stiff; moist				
4	SC	3.5-5.0 CLAYEY SAND: dark yellowish brown; fine sand; dense; moist	J-1	10	SS	0.8/1.5 recovery
	CL SC	5.0-33.5 SANDY CLAY/CLAYEY SAND: dark yellowish brown; moderately plastic fines; fine sand; very stiff to hard; moist; medium dense		16		
6				21		
					RD	set tub and cased to ±5'
				10	DR	0.9/1.0 recovery pocket penetrometer 3.25 tsf
8			C-1	19		
					RD	
10		sand content varies with depth; becomes firm to stiff				
12			PB-1		PB	2.5/2.5 recovery
14						
			J-2	3	SS	1.5/1.5 recovery
				5		
16				8		
					RD	
				6	DR	1.0/1.0 recovery pocket penetrometer
	(SC)	17.5-18.0' clayey sand	C-2	11		
18					RD	1.25 tsf
		becomes soft to firm; loose to medium dense		2	SS	1.5/1.5 recovery
20			J-3	2		

Sheet 1 of 5

Project DESIGN UNIT A425 **Date Drilled** 11/5/83 **Hole No.** 34-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL SC	5.0-33.5 SANDY CLAY/CLAYEY SAND: (cont.)		3	SS	
					RD	
22			PB-2		PB	2.5/2.5 recovery
24	(SC)	becomes stiff to medium dense; 6" clayey sand lens	J-4	2 4 7	SS	1.5/1.5 recovery
26					RD	
			C-3	5 6	DR	1.0/1.0 recovery pocket penetrometer 0.5 tsf
28					RD	
	(SC)	4" clayey sand lens; becomes very soft to very loose	J-5	0 0 1	SS	1.5/1.5 recovery weight of rods and hammer pushed sampler
30					RD	
32			PB-3		PB	2.5/2.5 recovery
34	SM	33.5-37.0 SILTY SAND: dark yellowish brown; fine sand; low plastic fines; sand content varies in interbeds; loose to medium dense; wet		0 4 8	SS	0.0/1.5 recovery
36					RD	
	CL	37.0-40.0 SANDY CLAY: dark yellowish brown; moderately plastic fines; fine sand; stiff; wet	C-4	8 29	DR	0.3/1.0 recovery
38					RD	
			J-6	1 4 5	SS	1.5/1.5 recovery
40	CL	40.0-43.0 SILTY CLAY: dark yellowish brown; moderately plastic fines; trace fine sand; stiff; wet			RD	
42			PB-4		PB	2.5/2.5 recovery
	ML	43.0-45.5 CLAYEY SILT: dusky yellowish brown; low plastic fines;				
44						

Sheet 2 **of** 5

Project DESIGN UNIT A425 **Date Drilled** 11/5/83 **Hole No.** 34-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	43.0-45.5 CLAYEY SILT: (continued) medium dense; wet			PB	
			J-7	1 10 13	SS	1.5/1.5 recovery
46	CL (ML) (SM)	45.5-57.0 SANDY CLAY: moderate olive grey; moderately plastic fines; fine sand; stiff; moist; interbeds with silts and sands			RD	
48						
			J-8	3 4 12	SS	1.0/1.5 recovery
50	(SP)	increased clay content with depth with sand lenses 2" thick; sands are wet			RD	
52				9	DR	1.0/1.0 recovery
	(ML) (SM) (CL)	interbedded silty clay, clayey silt; sandy clay and sandy silt contains wood fragments	C-5	24		pocket penetrometer 0.5 tsf
					RD	
54						
	(SP)	6" sand lens; saturated	J-9	12 12 13	SS	1.0/1.5 recovery
56					RD	
						rig chatter
	SP	57.0-67.0 SAND: salt and pepper; fine to medium sand; trace silt; trace coarse sand / fine gravel in interbeds; very dense; wet; various origins				
58						
			J-10	13 27 33	SS	0.3/1.0 recovery
60					RD	
62		occasional cobbles				
			PB-5		PB	1.4/2.5 recovery
64						
			J-11	19 43 50-4.5	SS	0.5/1.5 recovery
66		coarse gravels			RD	
		TOPANGA FORMATION 67.0-100.5 CLAYSTONE: light grey; very				
68						

Sheet 3 of 5

Project DESIGN UNIT A425 **Date Drilled** 11/5/83 **Hole No.** 34-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		67.0-100.5 <u>CLAYSTONE</u>: (continued) stiff to hard; moist; occasional thin interbeds; uncemented to weakly cemented			RD	
			J-12	7 25 49	SS	0.0/1.5 recovery full of gravel with small sliced sample of grey clay
70		Physical Condition: little fractured to massive; friable to low hardness; friable to weak strength; fresh to little weathered			RD	
72				14 43	DR	0.0/1.0 recovery sample fell out
		becoming light olive grey with interbeds of light grey			RD	
74						
			J-13	9 20 28	SS	1.5/1.5 recovery
76					RD	
78						
80			PB-6		PB	2.0/2.0 recovery
82			J-14	13 30 46	SS	1.5/1.5 recovery
		becomes weakly cemented			RD	
84			C-6	136	DR	0.5/0.5 recovery
86					RD	
88			PB-7		PB	2.5/2.5 recovery
90						
			J-15	14 28 41	SS	1.5/1.5 recovery
92						

Sheet 4 **of** 5

Project DESIGN UNIT A425 **Date Drilled** 11/5/83 **Hole No.** 34-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		67.0-100.5 <u>CLAYSTONE</u>: (continued) grades to olive grey with medium dark grey interbeds steeply dipping ±70°			RD	
94				43	DR	0.9/1.0 recovery
			C-7	83		
					RD	
96						
98						
		steeply dipping thin interbeds of weakly cemented sand	PB-8		PB	2.5/2.5 recovery
100						rolled tube under
		B.H. 100.5' Terminated hole				complete drilling 2:30 11/5/83 tremied grout to surface
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.

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BORING LOG 34-3

Proj: DESIGN UNIT A425 **Date Drilled** 10/26-27/83 **Ground Elev.** 577'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 115.5'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.4 ASPHALT			RD	start drilling 7:30
		ALLUVIUM				
	CL	0.4-3.0 SANDY CLAY: olive black; moderately plastic fines; fine sand; soft; moist				
2						
	CL	3.0-6.5 SILTY CLAY: greyish brown; moderately plastic fines; very stiff; moist				
4						
		color change to moderate brown; some very fine sand	J-1	2 8 13	SS	1.2/1.5 recovery
6						
					RD	set tub and cased to ~5'
	CL	6.5-13.5 SANDY CLAY: moderate brown; moderately plastic fines; fine sand; very stiff; moist	C-1	5 9	DR	0.8/1.0 recovery
8					RD	pocket pen: > 4.5 tsf
		sand content decreases	J-2	9 7 14	SS	0.7/1.5 recovery
10						
					RD	
12						
			PB-1		PB	2.5/2.5 recovery
14	SC	13.5-15.0 CLAYEY SAND: moderate brown; fine sand; moderately plastic fines; medium dense; moist	J-3	2 5 9	SS	1.0/1.5 recovery
	CL	15.0-23.5 SANDY CLAY: moderate brown; fine sand; stiff; moist				
16					RD	
		17.5' decrease sand content becomes silty clay	C-2	3 5	DR	1.0/1.0 recovery pocket pen: 1.25 tsf
18					RD	
		increased sand content	J-4	2 4	SS	0.5/1.5 recovery
20						

Sheet 1 of 5

Project DESIGN UNIT A425 **Date Drilled** 10/26-27/83 **Hole No.** 34-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	15.0-23.5 SANDY CLAY: (continued)		7	SS	
					RD	
22			PB-2		PB	2.5/2.5 recovery
24	SC	23.5-27.5 CLAYEY SAND: moderate brown; fine sand; moderately plastic fines; medium dense; moist to wet	J-5	3 5 13	SS	0.7/1.5 recovery
26		decrease fine content			RD	
			C-3	7 5	DR	0.8/1.0 recovery
28	CL	27.5-37.8 SANDY CLAY: moderate brown; moderately plastic fines; fine sand; occasional gravel; moist to wet; stiff			RD	
			J-6	5 8 8	SS	1.5/1.5 recovery
30		3" silty clay lens; stiff			RD	
32		color change to olive grey	PB-3		PB	2.5/2.5 recovery
34		becomes stiff to very stiff	J-7	1 7 13	SS	1.5/1.5 recovery
36		decreased sand content			RD	
			C-4	4 7	DR	1.0/1.0 recovery
38	SC	37.8-43.0 CLAYEY SAND: moderate brown; fine sand; low to moderately plastic fines; moist to wet; loose			RD	
				1 2 5	SS	0.0/1.5 recovery fell out
40					RD	
42			PB-4		PB	2.2/2.5 recovery
44	ML	43.0-45.2 SILT: olive grey; low plastic				

Sheet 2 **of** 5

Project DESIGN UNIT A425 **Date Drilled** 10/26-27/83 **Hole No.** 34-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	43.0-45.2 SILT: (continued) fines; medium dense; wet			PB	
			J-8	3 5 11	SS	1.3/1.5 recovery
46	CL	45.2-47.5 SILTY CLAY: olive grey; moderately plastic fines; stiff to very stiff; moist to wet			RD	
48	ML	47.5-50.0 SANDY SILT: olive grey; low plastic fines; very fine sand moist to wet				
			J-9	2 12	SS	1.5/1.5 recovery first 6" sample sinks by weight of rod
50	CL	50.0-51.5 SANDY CLAY: olive grey; moderately plastic fines; fine sand; stiff; moist			RD	
52	SM	51.5-53.5 SILTY SAND: olive grey; very fine sand; dense; moist to wet	C-5	7 23	DR	1.0/1.0 recovery
54	SM	53.5-63.5 SILTY SAND: olive grey; fine to coarse sand, angular; some silty clay 1" lenses; dense; wet			RD	
			J-10	15 20 25	SS	0.9/1.5 recovery
56					RD	
58		grades to dark grey				
	(SP)	some sand lenses	J-11	8 6 5	SS	1.0/1.5 recovery
60		organic odor, contains wood fragments; becomes medium dense			RD	
62	(GM)	gravelly lens	PB-5		PB	2.0/2.5 recovery
64		TOPANGA FORMATION 63.5-115.5 INTERBEDDED CLAYSTONE AND SANDSTONE: claystone olive grey; sandstone bluish grey; thinly bedded sandstone beds in thinly to thickly bedded claystone; weakly to moderately cemented; minor offsets of beds~¼"				
	75°		J-12	10 27 48	SS	0.7/1.5 recovery
66					RD	
68		68' massive claystone		38	DR	

Sheet 3 of 5

Project DESIGN UNIT A425 **Date Drilled** 10/26-27/83 **Hole No.** 34-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		63.5-115.5 <u>INTERBEDDED CLAYSTONE AND SANDSTONE</u>: (continued)	C-6	50-4"	RD	0.4/0.8 recovery
		Physical Condition: moderately fractured; low hardness; moderate strong; fresh				
70		contains some siltstone lenses				minor oil on tub
72					PB	rig chatter sliced whole sample smashed tube at start
74		contains chert? hard nodules continued steep dip	PB-6		PB	re-cut new sample 2.2/2.4 recovery
76			J-13	50-3"	SS	0.2/0.2 recovery
78					RD	
80						
82						
84		increased claystone content, coal seam 1/16" wide	PB-7		PB	2.4/2.5 recovery
86						10/26/83
					RD	10/27/83 cleaned tub water at 18'
88						
90		well cemented zone 8" thick, increased sand in this zone				rig chatter
92						

Project DESIGN UNIT A425 **Date Drilled** 10/26-27/83 **Hole No.** 34-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		63.5-115.5 <u>INTERBEDDED CLAYSTONE AND SANDSTONE</u>: (continued)			RD	
94		massive claystone	PB-8		PB	2.1/2.1 recovery too hard to cut, stopped short
					RD	
96						
98						
100						
102						
104		primarily sandstone, contains irregular organics (probably wood)	PB-9		PB	2.3/2.4 recovery
106					RD	intense rig chatter
		massive claystone				quiet
108		well cemented zone				intense chatter
110						quiet
						rig chatter 3"
112						
						1.4/2.5 recovery
114						completed drilling installed piezometer to bottom; 95-115' slotted
		well cemented sandstone	PB-10		PB	
116		B.H. 115.5' Terminated hole				

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
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BORING LOG 34-4

Proj: DESIGN UNIT A 425 **Date Drilled** 10/27-28/83 **Ground Elev.** 575'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 114.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.4 ASPHALT			RD	start drilling 1:45
	GP	0.4-0.7 BASEROCK				
	CL	ALLUVIUM				
		0.7-26.5 SANDY CLAY: greyish brown; low to moderately plastic fines, fine sand; stiff; moist				
2						
4		grading to moderate brown color				
			J-1	5 11 17	SS	Recovery 1.2/1.5
6					RD	set tub & cased to 5'
				5	DR	
8		grading to moderate brown; sand content increases; but variable	C-1	8		recovery 0.8/1.0
					RD	
			J-2	7 19 24	SS	recovery 1.0/1.5
10	(GM)	occasional thin (2") gravel lenses				
					RD	
12						
			PB-1		PB	recovery 1.7/2.5
14	(SC)	14.0-14.5 clayey sand				
			J-3	2 4 5	SS	
16						recovery 1.0/1.5
					RD	
				3	DR	
		17.5-18.0 silty clay	C-2	3		recovery 1.0/1.0 pocket pen : 0.5 tsf
18					RD	
		1' lens with increased sand content	J-4	1 3	SS	
20						

Sheet 1 of 5

Project DESIGN UNIT A425 **Date Drilled** 10/27-28/83 **Hole No.** 34-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	0.7-26.5 SANDY CLAY: (continued)		3	SS	recovery 1.5/1.5
					RD	
22			PB-2		PB	recovery 2.0/2.5 pocket pen: 0.5 tsf
24			J-5	2 6 10	SS	recovery 1.0/1.5
26					RD	
	SC	26.5-27.0 CLAYEY SAND: moderate brown fine sand; dense; moist to wet	C-3	25 27	DR	recovery 1.0/1.0
28	SM SP	27.0-31.5 SILTY SAND/SAND: moderate brown; fine sand; very dense; wet; silt content increases in lenses and some lenses coarse sand encountered			RD	
				13 27 26	SS	recovery 0.0/1.5
30					RD	
32	SC (SM)	31.5-35.6 CLAYEY SAND: olive grey; very fine sand; ocassional silty sand lenses; medium dense; moist	PB-3		PB	recovery 2.5/2.5
34			J-6	5 6 12	SS	recovery 1.2/1.5
36	CL	35.6-41.5 SANDY CLAY: dark olive grey; moderately plastic fines; very fine sand; stiff; moist to wet			RD	
			C-4	6 11	DR	recovery 1.0/1.0
38	(SC)	sand content increases with some clayey sand lenses			RD	
			J-7	3 4 8	SS	recovery 1.5/1.5
40					RD	
42	CL	41.5-50.0 SILTY CLAY: olive grey; moderately plastic fines; stiff; moist	PB-4		PB	recovery 2.5/2.5
44						

Sheet 2 of 5

Project DESIGN UNIT A 425 **Date Drilled** 10/27-28/83 **Hole No.** 34-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	41.5-50.0 SILTY CLAY: (continued) interbedded silt; sandy clay; silty sand; lenses 1/8" to 6"; organic odor some wood fragments			PB	
	(ML) (SM)		J-8	3 8 14		recovery 1.2/1.5
46					RD	
48						
		silty very fine sand	J-9	10 17 14	SS	recovery 0.3/1.5
50	SP	50.0-66.5 SAND:olive grey; fine sand; trace silt; very dense; wet			RD	
52			C-5	23 49	DR	recovery 0.9/1.0 drilled out and took SS sample
			J-10	15 25 32		recovery 0.3/1.5
54					RD	
56						
58		contains some coarse sand; fine gravel; dense	J-11	11 16 24	SS	recovery 0.7/1.5
60					RD	
62			PB-5		PB	recovery 1.5/2.5
64	(SM/ SP)	64.0-65.0 saturated silty sand/sand				
	(GM)	gravel lenses 3" thick	J-12	20 53	SS	recovery 1.0/1.0
66		TOPANGA FORMATION 66.5-114.0 INTERBEDDED CLAYSTONE/SILTSTONE: olive grey; claystone; bluish grey; thinly to thickly bedded; sandstone ½" to 6";dominantly (continued)			RD	rig chatter
68						

Sheet 3 **of** 5

Project DESIGN UNIT A425 **Date Drilled** 10/27-28/83 **Hole No.** 34-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	41.5-50.6 SILTY CLAY: (continued)claystone; interbeds ¼" to massive; weakly to moderately cemented			RD	
			J-13	50-2.5"	SS	5:30 10-27-83 7:00 10-28-83
					RD	
70						ocassional chatter
72		Physical Condition: moderately fractured; low hardness to hard; moderately strong to strong; fresh	C-6	106	DR	recovery 0.4/0.6 partial
					RD	rig chatter
74						
76						
78						rig chatter
80						
82		massive claystone	PB-6		PB	recovery 2.5/2.5
84		massive claystone	J-14	47 50-4"	SS	recovery 0.8/0.8
					RD	
86						oil on tub
88						
90						
92					PB	

Sheet 4 **of** 5

Project DESIGN UNIT A425 **Date Drilled** 10/27-28/83 **Hole No.** 34-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		66.0-114.0 INTERBEDDED CLAYSTONE/SANDSTONE (continued) massive claystone	PB-7		PB	recovery 2.5/2.5
94					RD	
96						
		cemented sand lens 6"				rig chatter
98						
100		cemented				rig chatter
		interbedded	PB-8		PB	
102						recovery 2.7/2.7
104					RD	
106		cemented sand				rig chatter
108						
110						end chatter
112		interbedded	PB-9		PB	recovery 1.9/2.0
114	BH	114.0' Terminated Hole				completed drilling 10:30, 10-28-83 grouted to surface
116						

Sheet 5 of 5

Converse Consultants, Inc.
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BORING LOG 34-5

Proj: DESIGN UNIT A425
Date Drilled 10/24-25/83
Ground Elev. 573'
Drill Rig Failing 1500
Logged By L. Schoeberlein
Total Depth 122.6'
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			AD	start drilling 11:45
	CL	ALLUVIUM 0.5-1.0 SILTY CLAY: greyish brown				
	CL	1.0-17.0 SANDY CLAY: greyish brown; moderately plastic fines; fine sand, occasional coarse sand; soft to firm; moist				
2			J-1	2 2 3	SS	1.2/1.5 recovery
					AD	
4		color change to dark yellowish brown				
6						
8		very stiff	J-2	6 12 19	SS	set tub & cased to 6' 1.0/1.5 recovery
10					RD	
12		color change to moderate yellowish brown; sand content increases	J-3	4 8 16	SS	1.2/1.5 recovery
14					RD	
			PB-1		PB	2.5/2.5 recovery
16		16.0-17.0 silty clay				
	SC	17.0-23.0 CLAYEY SAND: moderate yellowish brown; fine sand; loose; wet				
18			J-4	3 5 5	SS	1.2/1.5 recovery
					RD	
20						

Sheet 1 of 6

Project DESIGN UNIT A425 **Date Drilled** 10/24-25/83 **Hole No.** 34-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SC	17.0-23.0 CLAYEY SAND: (continued)			RD	
22			J-5	3 4 5	SS	0.7/1.5 recovery
	CL	23.0-25.0 SANDY CLAY: moderate yellowish brown; fine sand; stiff; moist				
24					RD	slight chatter
	(SP)	24.5-25.0 cemented sand zone				
	SC	25.0-27.0 CLAYEY SAND: moderate yellowish brown; fine sand; loose to medium dense; wet				
26			PB-2		PB	2.5/2.5 recovery
	CL	27.0-28.5 SANDY CLAY: moderate yellowish brown; fine sand; stiff; moist				
28			J-6	4 10 7	SS	0.7/1.5 recovery
	SC	28.5-31.0 CLAYEY SAND: moderate yellowish brown; fine sand; plastic fines; loose to medium dense; wet			RD	
30						
	SP	31.0-33.0 SAND: moderate yellowish brown; fine sand; trace silt; loose to medium dense; saturated				
32			J-7	4 7 6	SS	1.0/1.5 recovery
	CL	33.0-34.5 SILTY CLAY: moderate yellowish brown; stiff; moist			RD	
34						
	CL	34.5-36.5 SANDY CLAY: moderate yellowish brown; fine sand; firm; moist to wet	PB-3		PB	2.3/2.5 recovery
36						
	CL	36.5-43.0 SILTY CLAY: greenish black; moderately to highly plastic fines; stiff; moist				
38			J-8	2 5 8	SS	0.6/1.5 recovery hole caving in, mixed mud, from 31'
					RD	
40						
42			J-9	1 11 15	SS	1.5/1.5 recovery
	SP	43.0-44.5 SAND: greenish black;				
44					RD	

Sheet 2 **of** 6

Project DESIGN UNIT A425 **Date Drilled** 10/24-25/83 **Hole No.** 34-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	43.0-44.5 SAND: (continued) fine to medium dense sand; trace silt; medium dense; saturated			RD	
				6	DR	1.0/1.0 recovery
			C-1	12		
46	ML	44.5-50.0 SANDY SILT: greenish black; very fine sand; medium dense; moist; contains wood fragments and rootlets; sulfur odor			RD	
48						
	(GM)	color change, gravelly lense				rig chatter
50		TOPANGA FORMATION 50.0-122.6 INTERBEDDED CLAYSTONE AND SANDSTONE: olive grey claystone and medium bluish grey sandstone; thinly bedded sand to ~ ½" with thinly to thickly bedded claystone; weakly cemented, well cemented in places; sand lenses very fine grained				
	75°		J-10	15 27 57	SS	
52					RD	
54						rig chatter
56		Physical Condition: moderately fractured to massive; low hardness; weak strength; little weathered to fresh	PB-4		PB	2.5/2.5 recovery
58		sandy claystone lense 2' thick	J-11	29 50-5"	SS	0.9/0.9 recovery
					RD	
60						
	75°	interbedded, thin sandstone lenses	J-12	24 50-4"	SS	0.8/0.8 recovery 10/24/83
62					RD	10/25/83
64						
		massive	C-2	48 50-4"	DR	0.5/0.8 recovery
66					RD	
68						

Sheet 3 of 6

Project DESIGN UNIT A425 Date Drilled 10/24-25/83 Hole No. 34-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		50.0-122.6 INTERBEDDED CLAYSTONE AND SANDSTONE: (continued)			RD	
70						
		massive claystone	PB-5		PB	2.4/2.5 recovery
72						
		calcite in fractures	J-13	32 50-5"	SS	0.8/0.9 recovery
74					RD	
			C-3	61 50-4"	DR	0.8/0.8 recovery
76		massive, slight increase in cementation, slicks on some fracture surfaces and bedding planes			RD	
78		cemented zone (hard) sandstone lenses (thin)				rig chatter
80		interbedded primarily claystone	PB-6		PB	2.4/2.5 recovery
82						
					RD	
84						
			C-4	42 50-4.5"	DR	0.7/0.9 recovery
86		interbedded primarily claystone;slicks on some fracture surfaces and bedding planes			RD	
88		well cemented (hard) 1' sandy claystone/claystone				
90						
		moderately well cemented sandy claystone	PB-7		PB	2.2/2.5 recovery
92						

Sheet 4 of 6

Project DESIGN UNIT A425 **Date Drilled** 10/24-25/83 **Hole No.** 34-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		50.0-122.6 INTERBEDDED CLAYSTONE AND SANDSTONE: (continued)	PB-7		PB	
					RD	
94						intense rig chatter
		cemented zone (hard)				
				40	DR	0.7/0.9 recovery
		massive claystone bedding planes but no interbeds	C-5	50-5"		
96					RD	
						slight oil slick on tub
98						
						rig chatter mud too thick cleaned out tub
100		cemented zone slightly metamorphosed sandstone, contains numerous quartz veins; slicken sides in shale beds	PB-8		PB	1.3/1.6 recovery hard cutting pulled out early
102					RD	heavy chatter
104						heavy chatter
		continued well cemented partially metamorphosed sandstone and shale	C-6	120-4.5"	DR	
106					RD	
108						
						heavy chatter
110						smoother
		weakly to moderately cemented shale, sandy claystone, sandstone	PB-9		PB	2.5/2.5 recovery
112						
					RD	
114						0.3/0.4 recovery
		115.4' sandstone	C-7	125-5"	DR	
116					RD	

Sheet 5 **of** 6

Project DESIGN UNIT A425 **Date Drilled** 10/24-25/83 **Hole No.** 34-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		50.0-122.6 INTERBEDDED CLAYSTONE AND SANDSTONE: (continued)			RD	rig chatter
118						
120			PB-10		PB	2.6/2.6 recovery
122		B.H. 122.6' Terminated hole				complete drilling 10/25 installed piezometer to bottom, 100-120' slotted, backfilled with peagravel, bridged at top or hole caved in during peagravel placement
124						
126						
128						
130						
132						
134						
136						
138						
140						

Sheet 6 of 6

Converse Consultants, Inc.
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BORING LOG 34A

Proj: DESIGN UNIT A425
Date Drilled 2/8-9/83
Ground Elev. 586'
Drill Rig Mayhew 1000
Logged By G. Halbert
Total Depth 120'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS: 140 lb @ 30", DR: 340 lb @ 24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.4 A.C. PAVEMENT			RD	
		FILL				
	GW	0.4-2.5 SANDY GRAVEL: angular; very dense				4 7/8" rotary wash tri-cone
2						
		2.5-4.0 CONCRETE				
4	CL	4.0-8.0 SANDY CLAY: olive gray; very stiff to hard; moist to wet				
6						
		color change				
8		ALLUVIUM				
	SC	8.0-17.0 CLAYEY SAND: moderate to dark yellowish brown; moderately plastic fines; medium dense; moist				
10	(CL)	10.0-11.0' sandy clay	C-1	3 4	DR	1.0/1.0 recovery
			J-1	4 7 10	SS	1.25/1.5 recovery pocket penetrometer 3.0-4.0 tsf
12						
					RD	
14						
16						
	SP	17.0-30.0 SAND: moderate yellow brown; fine to medium sand; medium dense; wet; basalt rock fragments				
18						
20						

Sheet 1 of 6

Project DESIGN UNIT A425 **Date Drilled** 2/8-9/83 **Hole No.** 34A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	17.0-30.0 SAND: (continued)	J-2	5 5 6	SS	1.25/1.5 recovery
		occasional gravel to ½"; gravel is angular basalt, shale, sandstone fragments				
22					RD	
24						
	(GP/GM)	gravel layer				
26						
		fines increase with depth				
28	(GM)	gravel layer				
		transitional change				
30	SM/SC	30.0-50.0 SILTY SAND/CLAYEY SAND: moderate yellowish brown; with clay binder; medium dense; wet	C-2	4 4	DR	1.0/1.0 recovery
			J-3	4 5 10	SS	1.1/1.5 recovery
32					RD	1" basalt fragment in top of sample barrel (cuttings)
34						
36		grading finer				
38						
40		color change to dark yellowish brown; fine sand; slightly plastic	J-4	5 5 8	SS	1.5/1.5 recovery
42					RD	heavy chatter
44		basalt fragments - cobble size				

Project DESIGN UNIT A425 **Date Drilled** 2/8-9/83 **Hole No.** 34A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM SC	30.0-50.0 SILTY SAND/CLAYEY SAND: (cont)			RD	
46						
		basalt fragments				heavy chatter 6"
48						
50		WEATHERED BEDROCK 50.0-56.0 SILTY SANDSTONE: mottled light and moderate yellow brown; cemented, dark stained joints; moist; uneven horizontal parting	C-3	18 25	DR	0.7/1.0 recovery
			J-5	20 40/2"	SS	0.7/0.7 recovery 1" piece of basalt in sample
52					RD	
54						pieces of basalt in cuttings; frequent moderate rig chatter, harder drilling
		color change				
56		TOPANGA FORMATION 56.0-120.3 SANDSTONE: medium dark gray; massive; moist; well cemented; slightly calcareous; trace fine; fine to medium sand; Physical Condition: friable; weak strength; poorly to moderately indurated				
58						rig chatter, slower, harder drilling, hard dark gray, well cemented sandstone fragments in cuttings
60			C-4	80/3"	DR	no recovery too hard for SPT relatively hard, slow drilling
62						
64						continued rig chatter, hard drilling
		65.0-66.0 sandy siltstone; light brown				easier drilling 1'
66						
68						

Sheet 3 **of** 6

Project DESIGN UNIT A425 Date Drilled 2/8-9/83 Hole No. 34A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		56.0-120.3 SANDSTONE: (continued) color change to medium gray; massive; well-graded sand; dominately sub angular quartz; well cemented; slightly calcareous.			RD	continued moderate chatter
	<5°					too hard for SPT
70		Physical Condition: friable; poorly to moderately indurated; weak strength; low to moderate hardness; near horizontal bedding; occasional clayey silt laminae	PB-1		PB	0.9/2.5 recovery sample disturbed by rotation cutting of sample barrel (spinning at silty layers), bottom of tube ripped,
72					RD	relatively hard drilling; nearly continuous moderate right chatter, occasionally heavy
74						
		75.0-76.0 siltstone layer, olive gray				no chatter for 1'
76						
						occasional light brown silty cuttings
78						
80						difficult to sample
			C-5	80/4"	DR	0.3/0.3 recovery
			↻ J-6	100/2"	SS	no recovery
		81.5-82.5 silty zone			RD	no chatter for about 1'
82						
						continuous moderate chatter, heavy at times
84		sand dominantly quartz with minor gray granite and micaceous grains				
						2-8-83
86		86.0-87.6 clayey siltstone layer, olive gray, plastic, softer				2-9-83 no chatter for 1.5'
88						
		color change to medium dark gray, fine to medium grained poorly graded sand with trace fines				
90			PB-2		PB	1.25/2.0 recovery too hard for SPT
92						

Sheet 4 of 6

Project DESIGN UNIT A425 **Date Drilled** 2/8-9/83 **Hole No.** 34A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		56.0-120.3 SANDSTONE: (continued)			RD	moderate chatter, heavy at times, fairly difficult drilling, occasional brown silty sand cuttings (weathered fractures)
94		94.0-96.0 clayey siltstone layer with sand, dark gray, plastic				occasional light chatter, slightly easier drilling for 2'
96		color change to medium gray; well graded sand; subangular quartz (trace biolite grains); trace fines				
98						
100			C-6	100/3"	DR	0.25/0.25 recovery
					RD	
102		102.0-102.4 very hard zone				heavy chatter; brown silty sand cuttings mixed with gray sandstone.
104						light to moderate rig chatter, fairly consistent
		becomes silty sandstone, color changes to moderate brown; some sand sized angular volcanic fragments (red brown and black); sand mostly quartz				
106						
		color changes back to medium gray; sand is sub-rounded quartz, calcareous				
108						
110	65°	moderately hard to hard; moderately strong to strong; contact dip angle 65° from fabric change	PB-3		PB	heavy chatter, slow advance with Pitcher Barrel, refusal after 17" advance 1.4/1.4 recovery
112		111.6-113.0 hard white rock (intrusion), light gray; hard; moderately strong			RD	good hand specimen at bottom of barrel, saved sample of cuttings, 112'.
114		color change to medium dark gray, poorly graded fine quartz sand				heavy rig chatter, 3/4" fragments in cuttings. moderate chatter
116						

Project DESIGN UNIT A425 **Date Drilled** 2/8-9/83 **Hole No.** 34A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		56.0-120.3 SANDSTONE: (continued) fine to medium quartz sand; weak to moderately strong; friable to low hardness; slightly petroliferous			RD	115' - oil in mud light to moderate rig chatter
118						oil in mud
120			C-7	80/3"	DR	too hard for SPT 0.0/0.25 recovery
		B.H. 120.3' Terminated hole				no sample retained in rings
122						complete drilling 2/9
124						
126						
128						
130						
132						
134						
136						
138						
140						

Sheet 6 of 6

Converse Consultants, Inc.
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BORING LOG 34B

Proj: DESIGN UNIT A425
Date Drilled 1/4-7/83
Ground Elev. 573.5
Drill Rig Mayhew 1000
Logged By G. Halbert
Total Depth 121'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS: 140 lb @ 30",DR:340 lb. @24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 CONCRETE PAVEMENT			AD	6" core barrell
	SC	ALLUVIUM 0.4-36.0 CLAYEY SAND: moderate yellowish brown; fine to medium size sand; medium dense; moist				
2					RD	6" tri-cone, rotary wash to 10'
4						
6						
8						10', 4 7/8" rotary wash tri-cone bit
						lost sample (suction)
10				4 4	DR	recovery 0.0/1.0 pocket pen :1.0-1.5 tsf
			J-1	4 7 10	SS	recovery 1.3/1.5
12		slightly more coarse sand				
					RD	
14						groundwater between 10 & 20'
16						
18						
20						

Sheet 1 of 6

Project DESIGN UNIT A 425 **Date Drilled** 1/4-7/83 **Hole No.** 34 B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SC	0.4-36.0 CLAYEY SAND: (continued)	C-1	1 2	DR	recovery 1.0/1.0
	(CL)	20.5-21.0 sandy clay; moderate yellow brown; stiff; moist to wet; porous, 1/8" rootlets	J-2	3 4 6	SS	recovery 1.5/1.5
22					RD	pocket pen : 1.0-1.5 tsf
24						
26		ocassional silty zones				
28						
30			J-3	4 5 8	SS	
	(SP)	31.4-32.6 fine to medium sand				
32					RD	
34						
36	ML	36.0-47.5 <u>SANDY SILT:</u> olive grey; non-plastic; very fine micaceous sand; medium dense; wet				
38						
40			C-2	3 3	DR	recovery 1.0/1.0 pocket pen : 3.5-4.0 tsf
			J-4	4 6 9	SS	recovery 1.5/1.5
42		grading sandier			RD	
44						

Sheet 2 **of** 6

Project DESIGN UNIT A425 **Date Drilled** 2/3-7/83 **Hole No.** 34B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	36.0-47.5 SANDY SILT: (continued)			RD	
46		grading to silty sand				
48	SM	47.5-54.0 SILTY SAND: dark greenish grey; medium dense; wet				
50			J-5	4 9 14	SS	
52		grading cleaner			RD	
54	GM	54.0-73.5 SANDY GRAVEL: white, black & light brown; coarse sand; fine gravel; mostly hard quartz and basalt; pebbles; some sandstone				
56						
58		58.0-58.6 cobble				
60				4 25	DR RD	no recovery (too coarse) did not attempt SPT
62	(ML)	62.0-63.5 sandy silt lens				
64	(ML) (SM)	interbedded silt and sand lenses				moderate to heavy rig chatter at times
66						
68						

Sheet 3 of 6

Project DESIGN UNIT A425 Date Drilled 2/4-7/83 Hole No. 34B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GM	54.0-73.5 <u>SANDY GRAVEL</u>: (continued) contains interbedded gravelly sand, sand and silt			RD	continued intermittent rig chatter
	(SM) (ML)					
70		(channel alluvium)	C-3	30 30	DR	recovery 0.4/1.0
			J-6	15 47	SS	recovery 0.0/1.1
72					RD	
74		<u>TOPANGA FORMATION</u> 73.5-121.0 <u>CLAYEY SILTSTONE</u>: with interbedded sandstone; olive grey; very finely laminated clayey siltstone calcareous; very moist				
76						
78		<u>Physical Condition</u>: weak strength; plastic when remolded				
80			PB-1		PB	recovery 2.2/2.3
82	75°		J-7	45/4"	SS	recovery 0.3/0.3 pocket pen : 4.0 tsf
84						
86						
		87.5-90.0 poorly to moderately indurated cemented zone				
88						light to moderate rig chatter 87.5-90.0'
90			C-4	11 30	DR	
		3/4" diameter calcareous nodule siltstone is slightly calcareous			RD	stopped drill.2-4-83
92						

Project DESIGN UNIT A425 **Date Drilled** 2-7-83 **Hole No.** 34B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		73.5-121.0 CLAYEY SILTSTONE: with interbedded sandstone (continued) color change to olive black steeply dipping; laminated bulk sand (fine to medium)			RD	began @ 90' 2-7-83
94						
96						
98						
100						
			PB 2		PB	recovery 1.5/1.5
102		gradual slight increase in drilling hardness with depth				
					RD	
104						harder cemented zones not noticed during drilling
106						
108						
						slightly harder drilling action
110						
112						
114						
116						

Sheet 5 of 6

Project DESIGN UNIT A 425 **Date Drilled** 2/4-7/83 **Hole No.** 34B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		73.0-121.0 <u>CLAYEY SILTSTONE</u>: with SANDSTONE, minor offset shearing; siltstone layers nearly massive; steeply dipping (60° to 70°)			RD	
118						
120			C-5	25 25	DR	recovery 1.0/1.0 pocket pen : 4.0 tsf completed drilling 2-7-83
	BH	121.0 Terminated Hole				
122						
124						
126						
128						
130						
132						
134						
136						
138						
140						

Sheet 6 of 6

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 34C

Proj: DESIGN UNIT A 425
Date Drilled 1-25-83
Ground Elev. 552'
Drill Rig
Logged By D. Gillette
Total Depth 76.0'
Hole Diameter 36"
Hammer Weight & Fall N/A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	FILL	ARTIFICIAL FILL 0.0-10.5 SILTY SAND/SANDY SILT: contains pieces and chunks of asphalt and concrete; dusky brown; loose to medium dense; moist to wet (as observed on walls)			AD	Observation hole no samples required.
2						Difficult for auger drilling due to large chunks of concrete (curb and sidewalks asphalt) Note: bore hole subject to caving and raveling from 0-10.5'
4						
6						
8						
10	SP/ SM	ALLUVIUM 10.5-23.0 SAND/SILTY SAND: consists of sand with silty sand and clayey sand streaks; medium to dark grey; moist to very moist; loose to medium dense; readily caves and ravels; contains cobbles (well rounded to 5½") contain micaceous sand				Easier auger drilling
12						
14						
16		minor content of roots				
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A 425 **Date Drilled** 1-25-83 **Hole No.** 34C

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP/ SM	1.05-23.0 <u>SAND/SILTY SAND</u>: (continued) contains coarse sand layers organic odor			AD	H_2O at 21.0'; flows in from all sides at approximately 20-25 gpm. Note: Bore hole subject to excessive caving at & below water table
22						
	(CL)	23.0-24.5 sandy clay layer				
24						Drilled to 26.0';hole caved back to 21.0' before placing casing
26	BH	26.0 Terminated				finished drilling at 10am; 1-25-83. Placed 30" CMP casing backfilled hole with native material
28						
30						
32						
34						
36						
38						
40						
42						
44						

Sheet 2 **of** 2

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

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

BORING LOG 34D

Proj: DESIGN UNIT A425
Date Drilled 1/2-3/83
Ground Elev. 565'
Drill Rig MAYHEW 1000
Logged By G. Halbert
Total Depth 101'
Hole Diameter 6"
Hammer Weight & Fall SPT 140lb. 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 A.C.PAVEMENT			RD	
	ML	ALLUVIUM 0.4-20.0 CLAYEY SILT: dark yellowish brown moderately plastic; stiff; medium dense; moist				
2						
4	(SM)	ocassional very think bedded sandy layers (1"-2" thick; 1'-2' apart)				
6						
8						
10			J-1	3 6 9	SS	recovery 1.5/1.5 pocket pen. 3.0 tsf
12					RD	
	(SM)	alternating sandy and silty layers				
14						
16						
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A 425 **Date Drilled** 2/1-2-3/83 **Hole No.** 34D

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM/ (SC)	20.0-27.0 SILTY SAND: mottled yellow brown and orange; fine sand medium dense; moist to wet		1 3	DR	Bad sample, hammer stuck, lifted sampler cuttings in sample
22			J-2	4 8 10	SS	pocket pen: 2.5 tsf recovery 1.5/1.5
		clayey layers			RD	
24						
26						
	SW	27.0/29.0 GRAVELLY SAND: light brown				moderate rig chatter
28						
	SP	29.0-36.0 SAND: moderate to dark yellowish brown; fine to medium sand medium dense; very moist; ocassional fine gravel				
30						
32			C-1	7 10	DR	recovery 1.0/1.0
34			J-3	11 9 15	SS	recovery 1.2/1.5 chatter
	(GM)/ (GP)	gravelly			RD	
36		TOPANGA FORMATION 36.0-101.0 CLAYEY SILTSTONE: olive grey with very thin beds (< 2") of brownish black fissile shale and medium dark grey sandstone; moist Physical Condition: hard soil consistency; poorly indurated; weak rock strength; plastic				light chatter shale pieces in cuttings
38						
40	DIP 49° to 52°		J-4	16 54	SS	recovery 1.0/1.0 harder drilling more chatter @ 40'
					RD	
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A425 **Date Drilled** 2/1-2-3/83 **Hole No.** 34D

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		36.0-101.0 CLAYEY SILTSTONE: (continued)			RD	
46						
48		general gradual increase in hardness				
50		hard zone 50.8-51.4 CLAYEY SILTSTONE: color change to olive grey; well-cemented, finely laminated; jointed; (approximately 1" square rock fragments in bottom of sample)	PB-1		PB RD	recovery 1.0/1.7 stopped at 20" because of very hard driving bottom of pitcher tube bent and scratched; only 1' of sample in top of tube; bottom contained fragment of harder rock as described @ 51' probably too hard for drive sample, kept sample in jar
52						
54		becomes interbedded siltstone, sandstone and shale, weak strength dominantly clayey siltstone: medium dark grey; thinly bedded (4" to 6") faint, non-parallel; finely laminated (1 mm); micaceous; plastic; slightly calcaieous				
56						
58						
60	40°	subordinate sandstone: medium grey; silty; with thin bedding (1" thick); very friable	PB-2 C-2	 15 30	PB DR	recovery 0.5/2.5
62						pocket pen. will not penetrate (>4.0tsf)
64						
66		66.0-66.5 hard zone similar to that at 51'				light rig chatter
68						

Sheet 3 of 5

Project DESIGN UNIT A425 **Date Drilled** 2/1-2-3/83 **Hole No.** 34D

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		36.0-101.0 CLAYEY SILTSTONE: (continued)			RD	
70						recovery 2.1/2.1
			PB-3		PB	hit hard zone at bottom of PB sample; too hard for SPT- did not attempt. Kept rock fragments from bottom of sample tube in jar 6" (rig chatter)
72		72.0-72.5 hard zone sandstone layer; medium dark to dark grey; fine silica sand; jointed			RD	
74						
76						
78		gradual increase in hardness and sand content				
80			C-3	20	DR	recovery 1.0/1.0
		sandstone layers more frequent and thicker (2" to 3" thick)		30		
				45	SS	pocket pen. will not penetrate (>4.0 tsf)
82			J-5	40/1"		
					RD	
		83.0-83.7 hard zone similar to zone at 72'; (well cemented silica sandstone)				moderate rig chatter
84						
86						
88						
90						
92						

Sheet 4 of 5

Project DESIGN UNIT A 425 Date Drilled 2/1-2-3/83 Hole No. 34D

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		36.0-101.0 <u>CLAYEY SILTSTONE</u>: (continued) color change to olive black			RD	
94						
96		96.0-96.4 hard zone well cemented; silicia sandstone				moderate rig chatter
98						
100		generally massive; faintly jointed	C-4	12 18	DR	pocket penetrometer will not penetrate (> 4.0 tsf)
	BH	101.0 Terminated Hole				
102						
104						
106						
108						
110						
112						
114						
116						

Appendix B

Geophysical Explorations

APPENDIX B GEOPHYSICAL EXPLORATIONS

B.1 DOWNHOLE SURVEY

B.1.1 Summary

A downhole shear wave velocity survey was performed in Boring CEG-34 during the 1981 geotechnical investigation of the Metro Rail Project. It should be noted that this boring is about 1300 feet northwest of the proposed location of the Universal City Station. The results of the survey conducted in this borehole is, however, included in this appendix since it is considered generally representative of the types of soil and rock conditions present at the Station site of Design Unit A425. Measurements were made at 5-foot intervals from the ground surface to depths up to 200 feet. A description of the technique and a summary of the results are presented in this appendix.

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 each borehole. A 12-channel signal enhancement seismograph (Geometrics Model ES 1210) 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

The downhole travel time profiles for both compressional and shear waves obtained from the downhole survey are shown in Figure B-1. Velocity estimates are based on selection of linear portions of these downhole arrival time profiles. The slopes of the linear portions yield the average compressional and shear velocities for the appropriate depth interval. Although it is possible to calculate the velocity for each 5-foot interval, this procedure would result in an assumed accuracy for velocity estimates that is unwarranted by the limitations of the survey techniques. More meaningful shear velocity estimates are made by averaging a series of arrivals that appear to be associated with materials of similar physical properties.

B.1.4 Discussions of Results

The estimated velocity profile for the downhole survey is summarized in Table B-1. Velocity estimates are based on selections of linear portions of the downhole arrival time curves.

The error analysis performed for these surveys involved a least squares fit of these data by estimating the mean of the slope (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* and Vs* 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.

The average shear wave velocity of the near-surface soils was found to be about 810 fps. At about 35 feet, the average shear wave velocity increased to 1410 fps.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

Crosshole measurements for the determination of compressional and shear wave velocities were also performed in Borings CEG-34 during the 1981 geotechnical investigation. As in the case of the downhole survey, the velocity measurements obtained from the crosshole survey are considered reasonably representative of the soil and bedrock conditions present at the Universal City Station site. Both compressional and shear velocity estimates were performed in an array of three boreholes spaced approximately 15 feet apart up to depths of 100 feet. Compressional wave and shear wave velocities obtained from the survey are summarized in Table B-2.

B.2.2 Field Procedure

The shear wave hammer is placed in an end hole of the array, and 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 to the walls 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.

B.2.3 Data Analysis

Actual crosshole distances were measured 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.

B.2.4 Discussion of Results

Wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet. The wave velocity is equal to the difference in travel path distance from the generating 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 Figures B-5 through B-8 and are summarized in Table B-2.

TABLE B-1
DOWN-HOLE VELOCITIES

Boring No.	Depth (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
34	10-35	1100	24	55	6	1100±80	807	31	40	5	810±70
	35-195	6243	451	312	31	6240±760	1412	142	71	24	1410±210

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

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

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

Es = estimated accuracy of shear survey

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

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

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

TABLE B-2

CROSS-HOLE VELOCITIES

Boring No.	Depth (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
34	10	1120	51	56	14	1120±110	830	14	41	16	830±60
	15	1240		120		1240±120	744	4	37	6	740±40
	20						634	5	32	6	630±40
	25	1252	8	63	4	1250±70	673	14	34	8	670±50
	30	2900		290		2900±290	793	10	40	19	790±50
	35	2322	132	116	3	2320±250	799	5	40	9	800±50
	40	3570	81	179	8	3570±260	810	2	41	24	810±40
	45	3630	158	161	3	3630±340	841	28	42	9	840±70
	50	5096	165	255	14	5100±420	1033	11	52	12	1030±60
	55	6048	0	301	4	6050±300	1140	15	57	3	1140±70
	60	5818	137	291	16	5820+430	1164	15	58	10	1160+70
	65						1109	14	55	4	1110±70
	70	6291	260	315	6	6290±570	1147	9	57	11	1150+70
	75	5446	310	272	4	5450±580	1260		126	2	1260±130
	80	5930	160	207	8	5930±460	1237	13	62	18	1240±80
	85	5100		510	1	5100±510	1536	161	77	4	1540±240
	90	6156	1061	308	6	6160+1370	1245	49	62	12	1250±110
	97	5757	138	288	9	5760±430	1333	37	67	18	1330±100

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

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

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

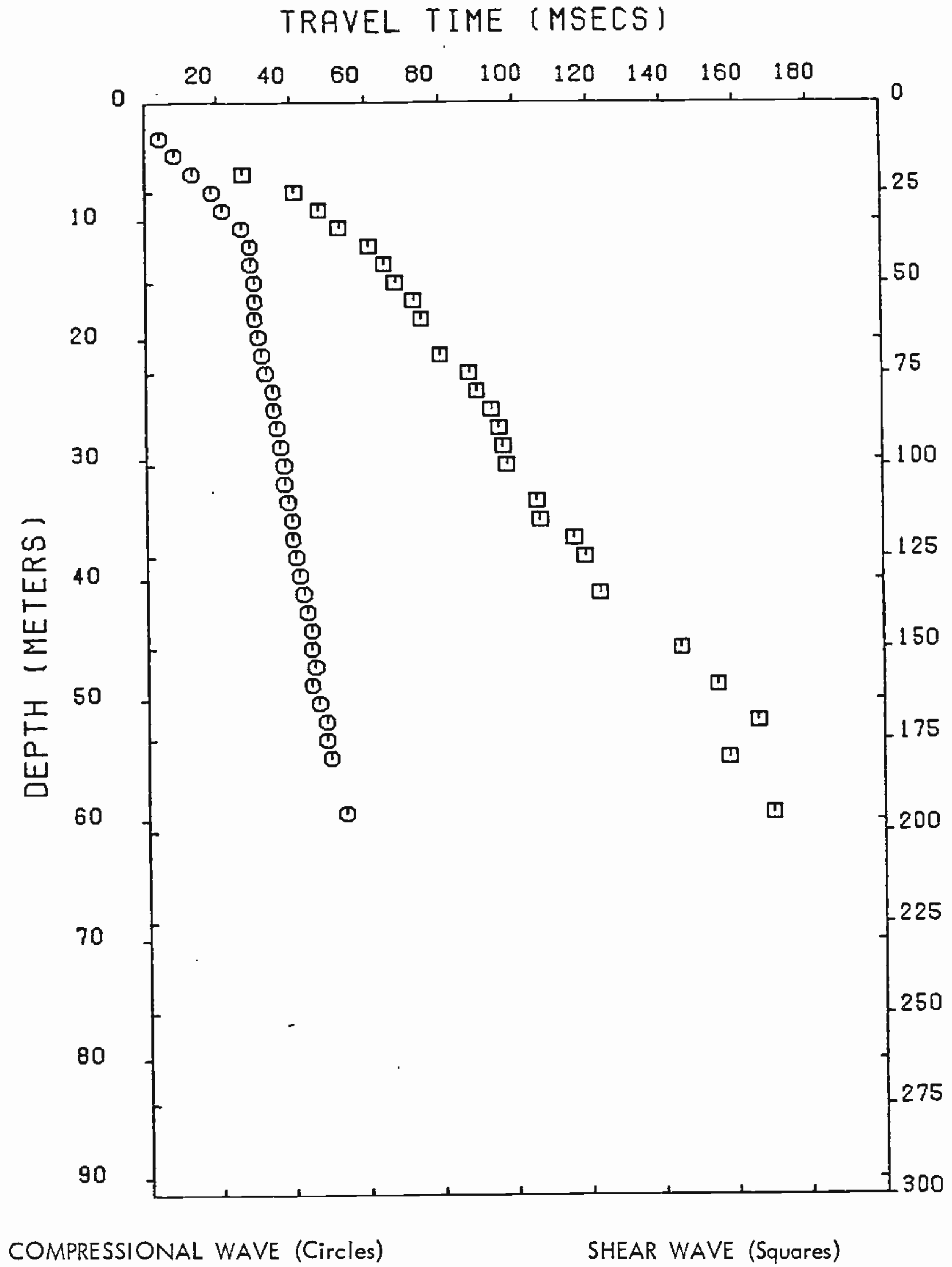
Es = estimated accuracy of shear survey

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

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

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



DOWNHOLE TRAVEL TIME PROFILE - BORING 34

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
B-1

Converse Consultants Geotechnical Engineering and Applied Sciences

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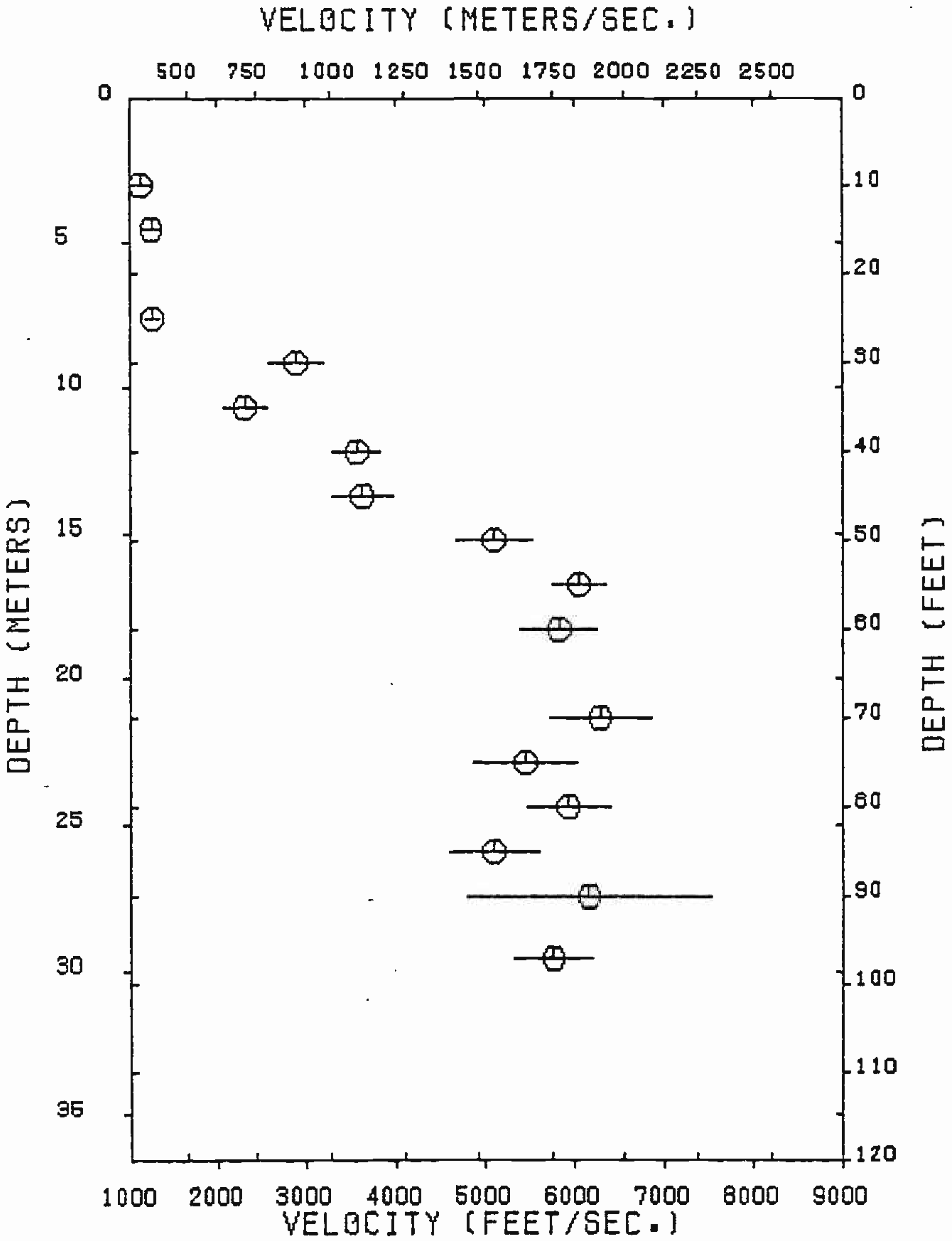

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 34

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B-2

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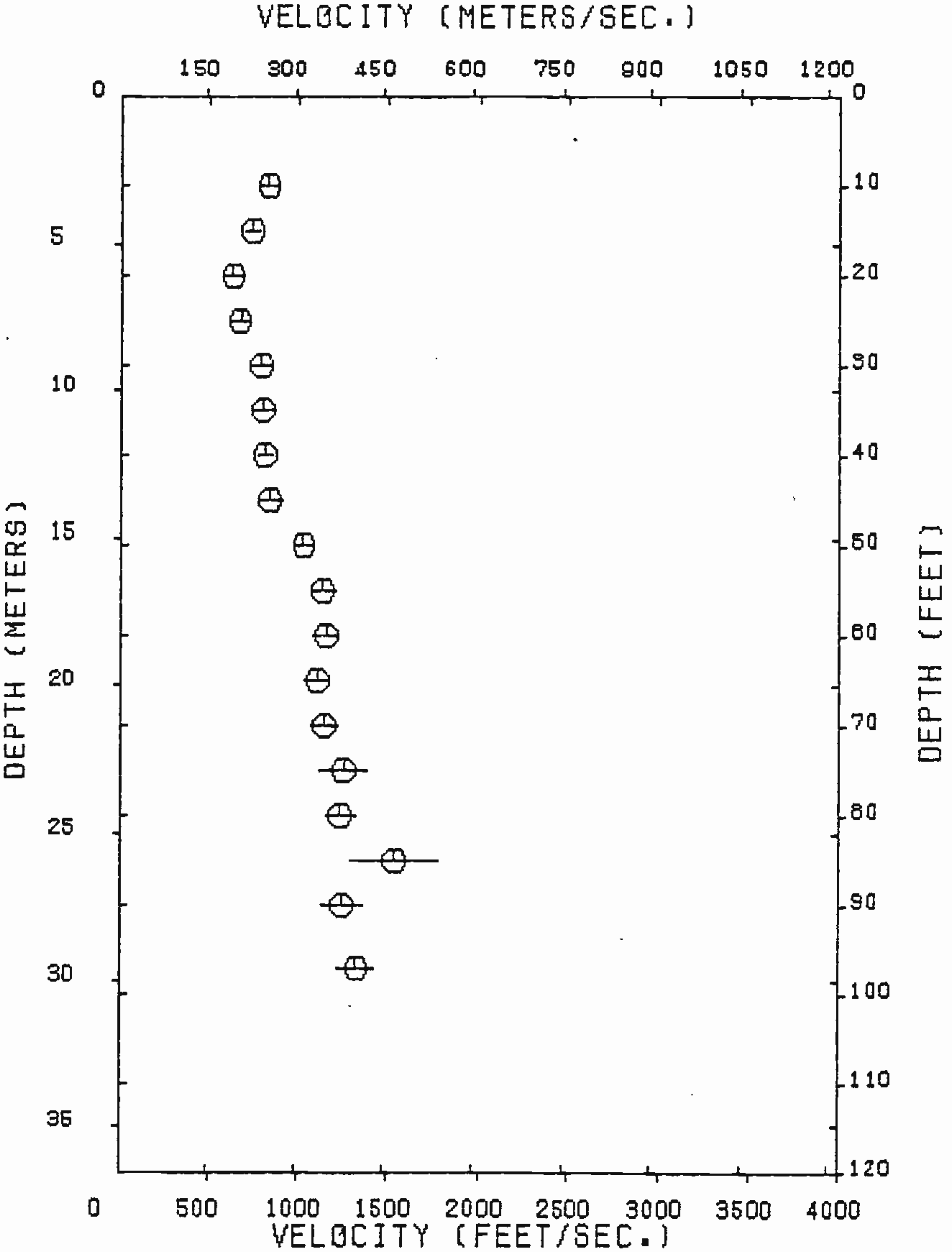

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 34

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B-3

Approved for publication ______ by ______

Appendix C

Pump Test Results

APPENDIX C PUMP TEST RESULTS

C.1 INTRODUCTION AND SUMMARY

A pump test was performed about 750 feet west of the proposed location of the Universal City Station to provide data for construction dewatering. Two pump tests were run at the same well to determine aquifer properties and boundary conditions and to confirm test results. The location of the pumping well is shown on Drawing No. 2 (Pump Test Well PT-2).

The methodology used for the test consisted of constant discharge tests with time-drawdown measurements in the observation wells. These measurements were plotted on log-log paper as drawdown versus t/r^2 where t = time in days and r = the radial distance of the observation well from the pumped well in feet. The data plots for the test were matched to a family of type curves by Newman (1975) for wells fully penetrating an unconfined aquifer. Under these conditions the typical log of drawdown versus the log of time response is an S-shaped curve with delayed drainage causing a flattening of the curve between early and late responses. Data plots are presented at the end of this appendix for each test along with matching curves, formulas used, and computations. Aquifer test data sheets for each test and observation well are also included in this appendix.

C.2 SITE CONDITIONS

The location of the multiple well pump test for Universal City Station is north of the end of Bluffside Drive as shown on Drawing 2. The test well was located in the southeast corner of a parking lot and two observation wells were located to the east in Weddington Park. Bedrock penetrated in the wells consists of sandstone of the Topanga Formation. The sandstone was encountered at depths ranging from 63 feet (at PT-2) to 48 feet (at OW-2) and was penetrated only a few feet.

The sandstone is overlain by alluvium of an old Los Angeles River channel that ranges in composition from sandy clay to clean sand and gravel. These deposits appear to be irregular in thickness and are probably lenticular. A clean sand and gravel bed that appears to be continuous between the test well and the two observation wells to the east was selected for aquifer testing. At test well PT-2, the sand and gravel is 12.5 feet thick, overlain by 2.5 feet of fine sand for a total aquifer thickness of 15 feet. Above the fine sand is 18 feet of unsaturated silt and clay. Underlying the sand and gravel aquifer is 30 feet of sandy clay which has a relatively low permeability.

At Observation Well OW-1, which is 66 feet east of PT-2, the aquifer is 12 feet thick. At Observation Well OW-2, the aquifer is 13 feet thick. OW-2 is 166 feet east of PT-2. The aquifer occurs at depths between 18 and 35 feet where penetrated by the three wells.

The static water level is close to the top of the aquifer at PT-2 and a few feet above the top of the aquifer in the two observation wells. The aquifer is under slight artesian pressure. The areal extent of the aquifer is unknown, but geologic boundaries are expected to be close because of the narrow sinuous nature of the stream channel deposits.

C.3 WELL CONSTRUCTION AND DEVELOPMENT

Well PT-2 was drilled by the cable tool method to a depth of 63 feet. The driven 12-inch casing was perforated from 20 to 33 feet with 12 punched slots per foot. The slots are 1-1/4 inches by 5/32 inch and are in staggered rows. The two observation wells were drilled by the rotary wash method. PVC casing 4 inches in diameter was installed in the 6-inch boring with a pea gravel filter and surface seal installed in the annulus. Originally, these wells were completed to bedrock with perforated casing. Later, they were backfilled and sealed with cement grout to a depth of approximately 35 feet.

All of the wells were developed to flush mud and cuttings and to provide hydraulic communication with the aquifer. The 12-inch well was surged with a bailor and then developed for two days with the test pump. The limited available drawdown (<15 feet) made well development difficult. Drawdown measurements for the test well are not available and the hydraulic efficiency of this well is unknown.

C.4 PUMP TESTING PROCEDURE

Because of expected boundary effects, two relatively short duration, constant discharge tests were conducted. The first test was run on April 14, 1983 for approximately 695 minutes at an average discharge rate of 30 gpm. The discharge, however, fluctuated between 25 and 45 gpm. The second test was performed on April 16, 1983, also at an average discharge of 30 gpm, for approximately 470 minutes as a check for the first test. Also, there was a broken water line near OW-2 during the first test that could have caused some recharge in the area of this well.

The test well was pumped with a limeshaft turbine pump and discharges were measured with an orifice plate and a bucket. Water was discharged into a storm drain.

Drawdowns were measured in the two observation wells with Stevens Recorders. Times were recorded manually on the chart paper at intervals to provide suitable logrithmic distributions.

Recovery measurements were made after the first test but the results were not useful. There was a very long time lag in water level responses partially because of the relatively long distance to observation wells and the relatively low pumping rate. A much higher test well yield was expected and utility lines were encountered at the intended location of OW-2 forcing it to be placed further from the test well. Also, there appeared to be a delayed response especially in OW-1, due to incomplete well development. The far well (OW-2) responded quicker than the near well (OW-1). This trend should have been reversed.

C.5 TEST INTERPRETATIONS

Time-drawdown data were plotted on log-log graphs as shown on the interpretation charts. Figure C-1 shows the plots for the first test for both observation wells. The log of drawdown(s) is plotted against t/r^2 where t is in days and r is the radial distance from the pumped well to the observation well in feet. These data plots were matched to the artesian type curve and appropriate match points were selected to determine values of s and t/r^2 for corresponding values of $W(\mu)$ and $1/\mu$. Calculations for transmissivity (T) and storativity (S) are shown. Figure C-2 shows data plots, match points, and calculations for the second test for both observation wells.

During the first test, both data plots have good initial matches with the artesian type curve (see Figure C-1). Also, both wells show responses to a barrier boundary in the latter part of the test. Water level responses indicate an increased rate of drawdown as the boundary is encountered as shown by the upward deflection on the data plot. Relatively poor matches were obtained during the second test, especially for OW-1 (see Figure C-2). The boundary effect was not well defined during the second test, in part due to the shorter duration of the test. Also, there was poor consistency in the shape of the responses that should have been identical. At least part of this inconsistency was probably due to the difficulty in maintaining a constant discharge during both tests. Both plots indicate delayed responses which was especially severe for OW-1. The delayed response merged with the boundary effect which makes data from OW-1 unreliable.

A check interpretation is shown on Figure C-3 which shows distance drawdown plots for both tests. The first test was influenced by boundary effects resulting in a relatively low transmissivity. The second test is probably high in terms of transmissivity. However, the average of these two interpretations is probably reasonable. Table C-1 summarizes the more reliable test results.

The mean transmissivity from Table C-1 is approximately 24,000 gpd/ft and the mean hydraulic conductivity is about 1,900 gpd/ft^2 (~9.0 x 10^{-2} cm/sec). Storativities are relatively high for initial responses suggesting unconfined conditions. As these deposits are dewatered, a specific yield value will apply that is considerably higher than the computed values of storativity. Specific yields of 0.20 to 0.25 are probably reasonable.

C.6 COMMENTS ON TEST RESULTS

Distance to the observed barrier boundary were not computed. This can be done, but it would not apply at the Universal City Station excavation. Barrier boundaries will have a beneficial influence on construction dewatering. Boundary effects may reduce the effective transmissivity by a factor of 3 to 4 depending on distances involved from the dewatering system to the boundaries.

The transmissivity data and average hydraulic conductivities appear quite reasonable in spite of delayed responses of OW-1 and the less than planned stress on the aquifer. Prior to well development, the anticipated pumping

Table C-1

RESULTS OF PUMP TEST PT-2

Test	Observation Well	Curve Match	Transmissivity (gpd/ft)	Average Hydraulic Conductivity (gpd/ft^2)	Storativity
1st	OW-1	Artesian T.C.	22,920	1,910	0.059
1st	OW-2	Artesian T.C.	24,557	1,889	0.014
2nd	OW-1		poor match - not valid		
2nd	OW-2	Artesian T.C.	28,650	2,203	0.008
1st & 2nd	OW-1, OW-2	Dist. d.d. (2 tests)	19,293 (ave.)	1,543 (ave.)	0.008

rates were several hundred gallons per minute and observation well spacings were determined on that basis. In retrospect, spacings of about 50 and 25 feet would have been better for the 30 gpm pumping rate and the thinner than expected aquifer.

Aquifer thickness is somewhat different at the construction site. However, the computed average hydraulic conductivity of 1,900 gpd/ft^2 is probably reasonable for the sands and gravels encountered at the Station site. Transmissivity can be estimated by multiplying the hydraulic conductivity times the aquifer thickness (clean sands and gravels). The silts and clays will of course have much lower hydraulic conductivities (by several orders of magnitude).

It is beyond the scope of this report to recommend specific dewatering systems. However, the limited aquifer thickness (i.e., up to 16 feet of sands and gravels overlying the Topanga Formation bedrock) at the Station site, suggest that well points would appear applicable. If wells are used, regardless of type, the limited available drawdown will require development of efficient wells. This requires well screens with adequate open areas along with good well development techniques.

AQUIFER TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Universal City Station

Date of Test 04/14/83

Static Water Level 17.95

Observed By TDH

Radius from Pumped Well 62.1

Average Discharge 30 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
7:40 a	0	--	--	17.950	0.0	
	2.5	$1.74x10^{-3}$	$4.51x10^{-7}$	17.955	0.005	
	10	$6.94x10^{-3}$	$1.80x10^{-6}$	17.960	0.010	
8:00	20	$1.39x10^{-2}$	$3.60x10^{-6}$	17.970	0.020	
8:04	24	$1.67x10^{-2}$	$4.33x10^{-6}$	17.975	0.025	
8:07	27	$1.88x10^{-2}$	$4.88x10^{-6}$	17.980	0.030	
8:10	30	$2.08x10^{-2}$	$5.39x10^{-6}$	17.990	0.040	
8:13	33	$2.29x10^{-2}$	$5.94x10^{-6}$	17.990	0.040	
8:16	36	$2.50x10^{-2}$	$6.48x10^{-6}$	18.005	0.055	
8:20	40	$2.78x10^{-2}$	$7.21x10^{-6}$	18.010	0.060	
8:24	44	$3.06x10^{-2}$	$7.93x10^{-6}$	18.010	0.060	
8:28	48	$3.33x10^{-2}$	$8.63x10^{-6}$	18.020	0.070	
8:32	52	$3.61x10^{-2}$	$9.36x10^{-6}$	18.030	0.080	
8:36	56	$3.89x10^{-2}$	$1.01x10^{-5}$	18.035	0.085	
8:40	60	$4.17x10^{-2}$	$1.08x10^{-5}$	18.050	0.100	
8:46	66	$4.58x10^{-2}$	$1.19x10^{-5}$	18.060	0.110	
8:52	72	$5.00x10^{-2}$	$1.30x10^{-5}$	18.070	0.120	
8:58	78	$5.42x10^{-2}$	$1.41x10^{-5}$	18.080	0.130	
9:04	84	$5.83x10^{-2}$	$1.51x10^{-5}$	18.090	0.140	
9:11	91	$6.32x10^{-2}$	$1.64x10^{-5}$	18.100	0.150	
9:20	100	$6.94x10^{-2}$	$1.80x10^{-5}$	18.125	0.176	
9:33	113	$7.85x10^{-2}$	$2.04x10^{-5}$	18.150	0.200	

Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
9:40	120	8.33×10^{-2}	2.16×10^{-5}	18.170	0.220	
9:50	130	9.03×10^{-2}	2.34×10^{-5}	18.190	0.240	
10:00	140	9.72×10^{-2}	2.52×10^{-5}	18.220	0.270	
10:20	160	1.11×10^{-1}	2.88×10^{-5}	18.250	0.300	
10:40	180	1.25×10^{-1}	3.24×10^{-5}	18.290	0.340	
11:00	200	1.39×10^{-1}	3.60×10^{-5}	18.330	0.380	
11:20	220	1.53×10^{-1}	3.97×10^{-5}	18.370	0.420	
11:43	243	1.69×10^{-1}	4.38×10^{-5}	18.410	0.460	
12:00	260	1.81×10^{-1}	4.69×10^{-5}	18.450	0.500	
12:30	290	2.01×10^{-1}	5.21×10^{-5}	18.490	0.540	
1:00	320	2.22×10^{-1}	5.76×10^{-5}	18.550	0.600	
1:30	350	2.43×10^{-1}	6.30×10^{-5}	18.610	0.660	
2:00	380	2.64×10^{-1}	6.85×10^{-5}	18.650	0.700	
2:30	410	2.85×10^{-1}	7.39×10^{-5}	18.690	0.740	
3:00	440	3.06×10^{-1}	7.93×10^{-5}	18.740	0.790	
4:00	500	3.47×10^{-1}	9.00×10^{-5}	18.830	0.880	
4:30	530	3.68×10^{-1}	9.54×10^{-5}	18.860	0.910	
5:15	575	3.99×10^{-1}	1.03×10^{-4}	18.920	0.970	
6:00	620	4.31×10^{-1}	1.12×10^{-4}	18.980	1.030	
7:00	680	4.72×10^{-1}	1.22×10^{-4}	19.060	1.110	
7:15	695	4.83×10^{-1}	1.25×10^{-4}	19.080	1.130	

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Universal City Station

Date of Test 04/14/83

Static Water Level 15.61

Observed By TDH

Radius from Pumped Well 161.9

Average Discharge 30 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
7:40	0	--	--	15.610	0.0	
10:49	9	6.25×10^{-3}	2.38×10^{-7}	15.615	0.005	
10:51	11	7.63×10^{-3}	2.91×10^{-7}	15.627	0.017	
8:00	20	1.39×10^{-2}	5.30×10^{-7}	15.629	0.019	
8:10	30	2.08×10^{-2}	7.95×10^{-7}	15.632	0.022	
8:20	40	2.78×10^{-2}	1.06×10^{-6}	15.640	0.030	
8:30	50	3.47×10^{-2}	1.32×10^{-6}	15.652	0.042	
8:35	55	3.81×10^{-2}	1.45×10^{-6}	15.660	0.050	
8:40	60	4.17×10^{-2}	1.59×10^{-6}	15.664	0.054	
8:50	70	4.86×10^{-2}	1.85×10^{-6}	15.680	0.070	
8:55	75	5.20×10^{-2}	1.98×10^{-6}	15.685	0.075	
9:00	80	5.55×10^{-2}	2.12×10^{-6}	15.693	0.083	
9:10	90	6.25×10^{-2}	2.38×10^{-6}	15.705	0.095	
9:20	100	6.94×10^{-2}	2.65×10^{-6}	15.711	0.101	
9:30	110	7.63×10^{-2}	2.91×10^{-6}	15.719	0.109	
9:40	120	8.33×10^{-2}	3.18×10^{-6}	15.725	0.115	
9:50	130	9.03×10^{-2}	3.45×10^{-6}	15.733	0.123	
10:00	140	9.72×10^{-2}	3.71×10^{-6}	15.743	0.133	
10:20	160	1.11×10^{-1}	4.23×10^{-6}	15.759	0.149	
10:40	180	1.25×10^{-1}	4.77×10^{-6}	15.771	0.161	
11:00	200	1.39×10^{-1}	5.30×10^{-6}	15.789	0.179	
11:22	222	1.54×10^{-1}	5.88×10^{-6}	15.809	0.199	

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Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
11:45	245	1.70×10^{-1}	6.49×10^{-6}	15.829	0.219	
12:00	260	1.81×10^{-1}	6.91×10^{-6}	15.836	0.226	
12:30	290	2.01×10^{-1}	7.67×10^{-6}	15.860	0.250	
1:00	320	2.22×10^{-1}	8.47×10^{-6}	15.868	0.258	
1:30	350	2.43×10^{-1}	9.27×10^{-6}	15.871	0.261	
2:00	380	2.64×10^{-1}	1.01×10^{-5}	15.871	0.261	
2:30	410	2.85×10^{-1}	1.09×10^{-5}	15.889	0.279	
3:00	440	3.06×10^{-1}	1.17×10^{-5}	15.911	0.301	
4:00	500	3.47×10^{-1}	1.32×10^{-5}	15.956	0.346	
4:30	530	3.68×10^{-1}	1.40×10^{-5}	16.010	0.400	
5:15	575	3.99×10^{-1}	1.52×10^{-5}	16.070	0.460	
6:00	620	4.31×10^{-1}	1.64×10^{-5}	16.120	0.510	
7:00	680	4.72×10^{-1}	1.80×10^{-5}	16.190	0.580	
7:15	695	4.83×10^{-1}	1.84×10^{-5}	16.210	0.600	

AQUIFER TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Universal City Station

Date of Test 04/16/83

Static Water Level 18.04

Observed By TDH

Radius from Pumped Well 62.1

Average Discharge 30 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
8:40 a	0			18.040	0.0	
8:51	11	$7.64\mathrm{x}1\bar{0}^{3}$	$1.98\mathrm{x}10^{-6}$	18.050	0.010	
8:59	19	$1.32\mathrm{x}10^{-2}$	$3.42\mathrm{x}10^{-6}$	18.060	0.020	
9:06	26	$1.81\mathrm{x}1\bar{0}^{2}$	$4.69\mathrm{x}10^{-6}$	18.065	0.025	
9:13	33	$2.29\mathrm{x}1\bar{0}^{2}$	$5.94\mathrm{x}10^{-6}$	18.070	0.030	
9:17	37	$2.57\mathrm{x}1\bar{0}^{2}$	$6.66\mathrm{x}10^{-6}$	18.075	0.035	
9:21	41	$2.85\mathrm{x}1\bar{0}^{2}$	$7.39\mathrm{x}10^{-6}$	18.080	0.040	
9:28	48	$3.33\mathrm{x}1\bar{0}^{2}$	$8.63\mathrm{x}10^{-6}$	18.085	0.045	
9:39	59	$4.10\mathrm{x}1\bar{0}^{2}$	$1.06\mathrm{x}10^{-5}$	18.090	0.050	
9:51	71	$4.93\mathrm{x}1\bar{0}^{2}$	$1.28\mathrm{x}10^{-5}$	18.100	0.060	
9:59	79	$5.49\mathrm{x}1\bar{0}^{2}$	$1.42\mathrm{x}10^{-5}$	18.110	0.070	
10:04	84	$5.83\mathrm{x}1\bar{0}^{2}$	$1.51\mathrm{x}10^{-5}$	18.120	0.080	
10:10	90	$6.25\mathrm{x}1\bar{0}^{2}$	$1.62\mathrm{x}10^{-5}$	18.130	0.090	
10:15	95	$6.60\mathrm{x}1\bar{0}^{2}$	$1.71\mathrm{x}10^{-5}$	18.135	0.095	
10:20	100	$6.94\mathrm{x}1\bar{0}^{2}$	$1.80\mathrm{x}10^{-5}$	18.145	0.105	
10:30	110	$7.64\mathrm{x}1\bar{0}^{2}$	$1.98\mathrm{x}10^{-5}$	18.160	0.120	
10:45	125	$8.68\mathrm{x}1\bar{0}^{2}$	$2.25\mathrm{x}10^{-5}$	18.180	0.140	
11:00	140	$9.72\mathrm{x}1\bar{0}^{2}$	$2.52\mathrm{x}10^{-5}$	18.190	0.150	
11:20	160	$1.11\mathrm{x}1\bar{0}^{1}$	$2.88\mathrm{x}10^{-5}$	18.210	0.170	
11:40	180	$1.25\mathrm{x}1\bar{0}^{1}$	$3.24\mathrm{x}10^{-5}$	18.235	0.195	
12:00	200	$1.39\mathrm{x}1\bar{0}^{1}$	$3.60\mathrm{x}10^{-5}$	18.260	0.220	
12:30	230	$1.60\mathrm{x}1\bar{0}^{1}$	$4.15\mathrm{x}10^{-5}$	18.290	0.250	

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Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
1:00	260	$1.81x10^{-1}$	$4.69x10^{-5}$	18.540	0.300	
1:30	290	$2.01x10^{-1}$	$5.21x10^{-5}$	18.370	0.330	
2:00	320	$2.22x10^{-1}$	$5.76x10^{-5}$	18.400	0.360	
2:35	355	$2.47x10^{-1}$	$6.40x10^{-5}$	18.450	0.410	
3:00	380	$2.64x10^{-1}$	$6.85x10^{-5}$	18.490	0.450	
4:05	445	$3.09x10^{-1}$	$8.01x10^{-5}$	18.560	0.520	
4:30	470	$3.26x10^{-1}$	$8.45x10^{-5}$	18.580	0.540	

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Universal City Station

Date of Test 04/16/83

Static Water Level 15.52

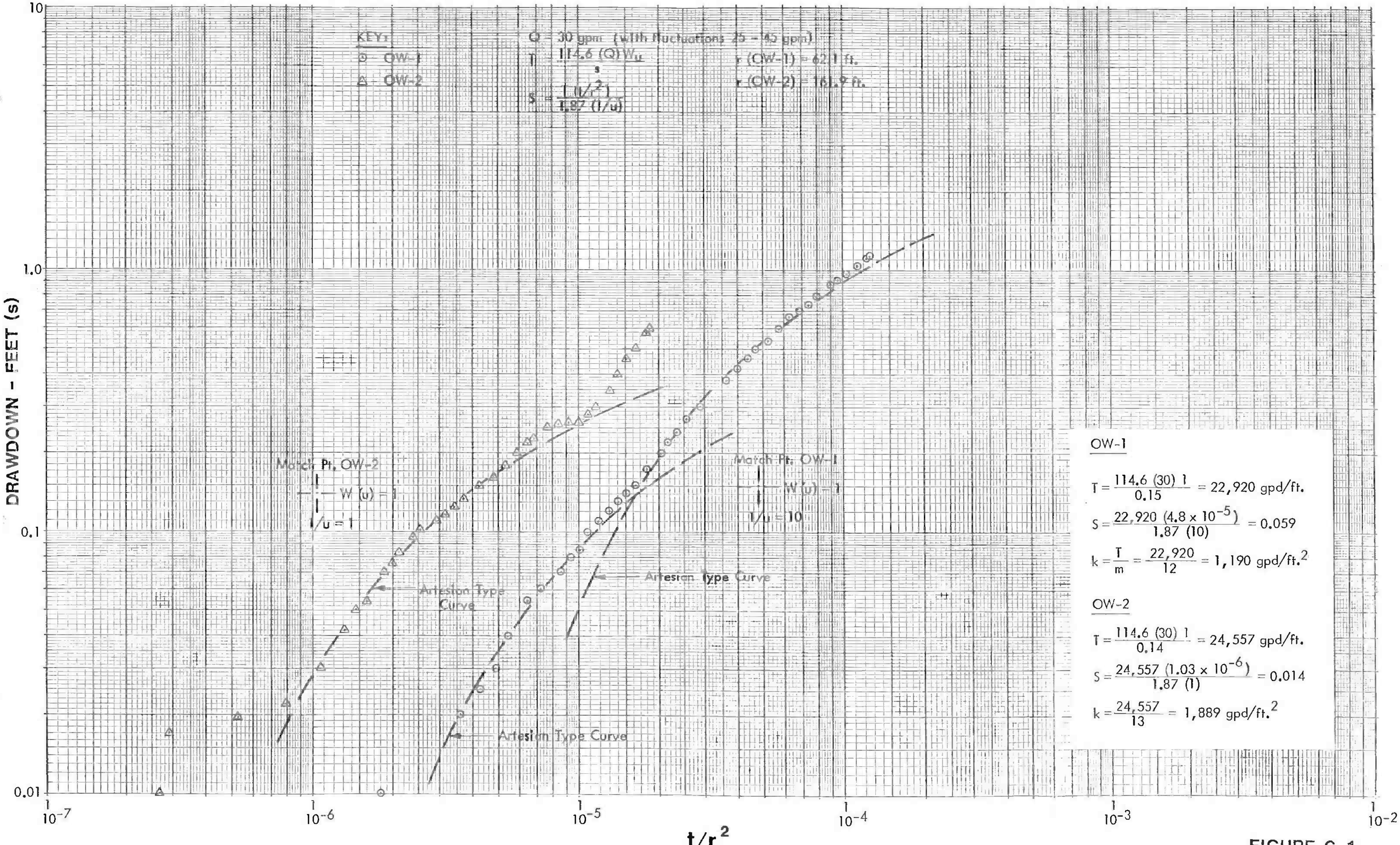
Observed By TDH

Radius from Pumped Well 161.9

Average Discharge 30 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
8:40	0			15.520		
8:41	1	6.94x10^{-4}	2.65x10^{-8}	15.525	0.005	
8:44	4	2.78x10^{-3}	1.06x10^{-7}	15.530	0.010	
8:46	6	4.17x10^{-3}	1.59x10^{-7}	15.535	0.015	
8:49	9	6.25x10^{-3}	2.38x10^{-7}	15.540	0.020	
8:52	12	8.33x10^{-3}	3.18x10^{-7}	15.547	0.027	
8:55	15	1.04x10^{-2}	3.97x10^{-7}	15.556	0.036	
8:57	17	1.18x10^{-2}	4.50x10^{-7}	15.559	0.039	
9:00	20	1.39x10^{-2}	5.30x10^{-7}	15.564	0.044	
9:04	24	1.67x10^{-2}	6.37x10^{-7}	15.568	0.048	
9:07	27	1.88x10^{-2}	7.17x10^{-7}	15.575	0.055	
9:12	32	2.22x10^{-2}	8.47x10^{-7}	15.583	0.063	
9:17	37	2.57x10^{-2}	9.80x10^{-7}	15.591	0.071	
9:22	42	2.92x10^{-2}	1.11x10^{-6}	15.600	0.080	
9:27	47	3.26x10^{-2}	1.24x10^{-6}	15.610	0.090	
9:30	50	3.47x10^{-2}	1.32x10^{-6}	15.612	0.092	
9:35	55	3.82x10^{-2}	1.46x10^{-6}	15.615	0.095	
9:40	60	4.17x10^{-2}	1.59x10^{-6}	15.621	0.101	
9:50	70	4.86x10^{-2}	1.84x10^{-6}	15.629	0.109	
10:00	80	5.56x10^{-2}	2.12x10^{-6}	15.640	0.120	
10:10	90	6.25x10^{-2}	2.38x10^{-6}	15.645	0.125	
10:20	100	6.94x10^{-2}	2.65x10^{-6}	15.655	0.135	

Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
10:30	110	7.64x10^{-2}	2.91x10^{-6}	15.665	0.145	
10:45	125	8.68x10^{-2}	3.31x10^{-6}	15.685	0.165	
11:00	140	9.72x10^{-2}	3.71x10^{-6}	15.695	0.175	
11:20	160	1.11x10^{-1}	4.23x10^{-6}	15.710	0.190	
11:40	180	1.25x10^{-1}	4.77x10^{-6}	15.720	0.200	
12:00	200	1.39x10^{-1}	5.30x10^{-6}	15.735	0.215	
12:30	230	1.60x10^{-1}	6.10x10^{-6}	15.750	0.230	
1:00	260	1.81x10^{-1}	6.91x10^{-6}	15.760	0.240	
1:30	290	2.01x10^{-1}	7.67x10^{-6}	15.775	0.255	
2:00	320	2.22x10^{-1}	8.47x10^{-6}	15.785	0.265	
2:35	355	2.47x10^{-1}	9.42x10^{-6}	15.800	0.280	
3:00	380	2.64x10^{-1}	1.01x10^{-5}	15.810	0.290	
4:05	445	3.09x10^{-1}	1.18x10^{-5}	15.835	0.315	
4:30	470	3.26x10^{-1}	1.24x10^{-5}	15.840	0.320	



Q = 30 gpm (with fluctuations 25 - 45 gpm)

$T = \frac{114.6\ (Q)\ W_u}{s}$

$S = \frac{T\ (t/r^2)}{1.87\ (1/u)}$

r (OW-1) = 62.1 ft.

r (OW-2) = 161.9 ft.

OW-1

$T = \frac{114.6\ (30)\ 1}{0.15} = 22,920$ gpd/ft.

$S = \frac{22,920\ (4.8 \times 10^{-5})}{1.87\ (10)} = 0.059$

$k = \frac{T}{m} = \frac{22,920}{12} = 1,190$ gpd/ft.^{2}

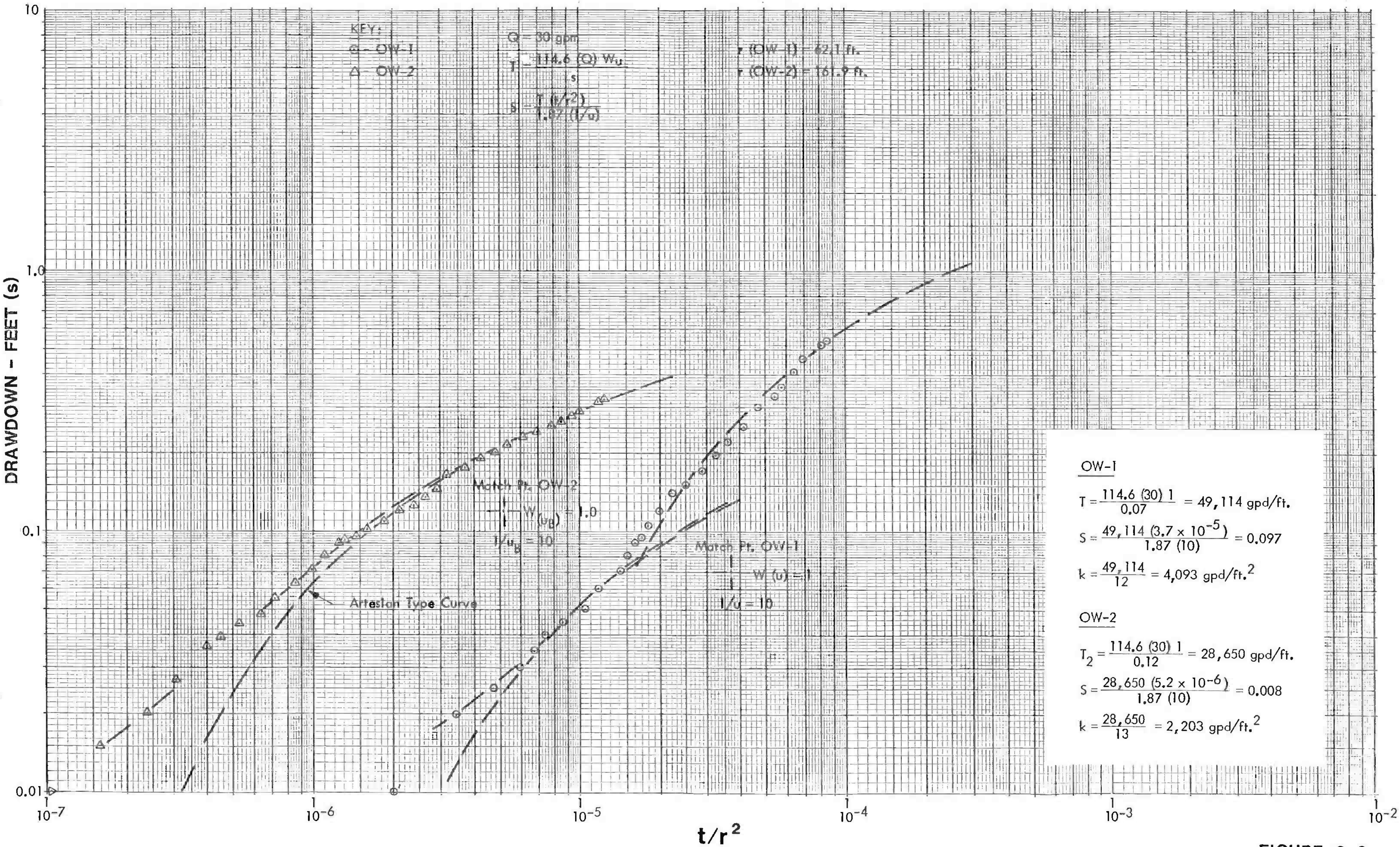
OW-2

$T = \frac{114.6\ (30)\ 1}{0.14} = 24,557$ gpd/ft.

$S = \frac{24,557\ (1.03 \times 10^{-6})}{1.87\ (1)} = 0.014$

$k = \frac{24,557}{13} = 1,889$ gpd/ft.^{2}

FIGURE C-1



OW-1

$$T = \frac{114.6\ (30)\ 1}{0.07} = 49,114 \text{ gpd/ft.}$$

$$S = \frac{49,114\ (3.7 \times 10^{-5})}{1.87\ (10)} = 0.097$$

$$k = \frac{49,114}{12} = 4,093 \text{ gpd/ft.}^2$$

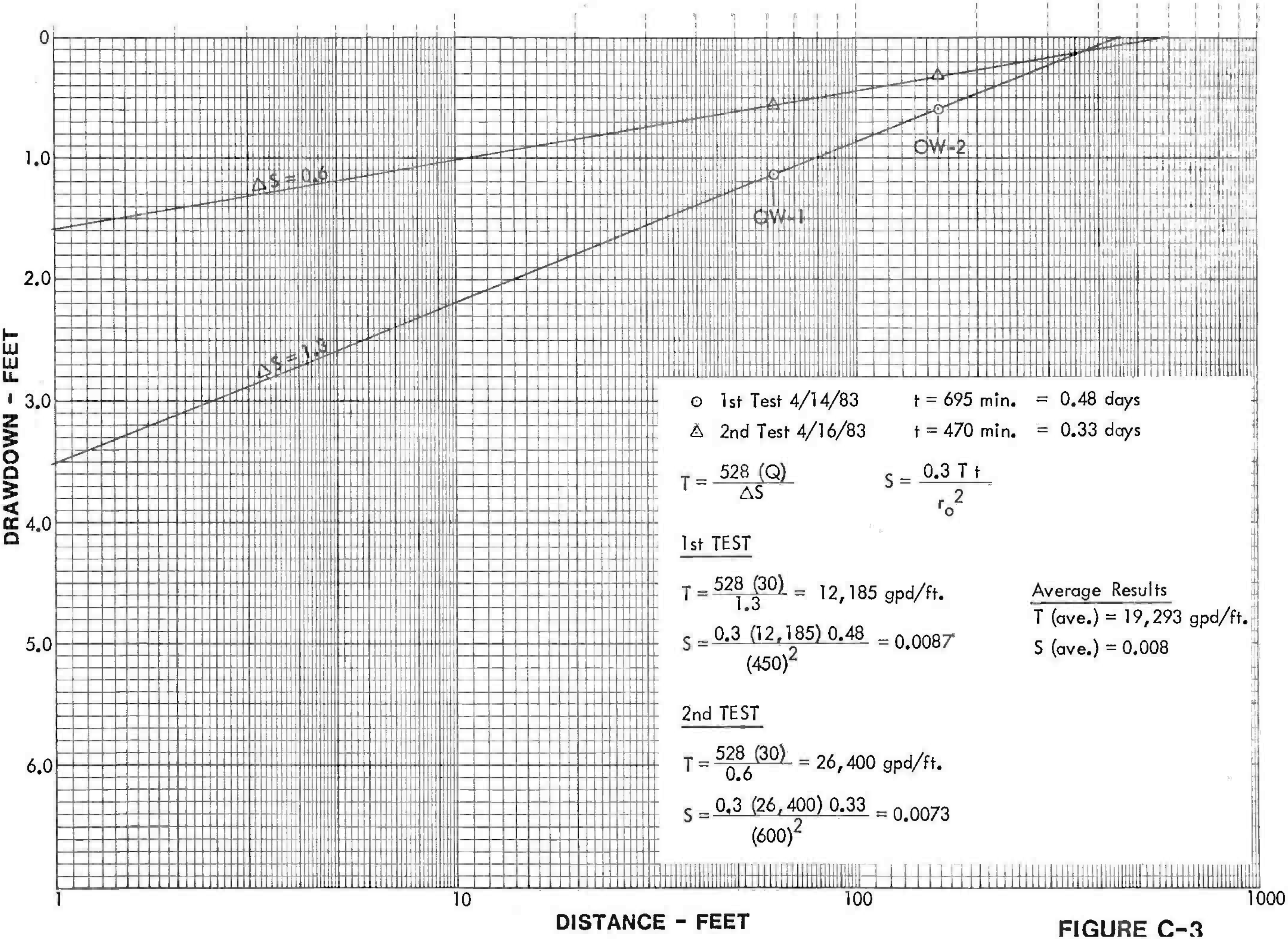
OW-2

$$T_2 = \frac{114.6\ (30)\ 1}{0.12} = 28,650 \text{ gpd/ft.}$$

$$S = \frac{28,650\ (5.2 \times 10^{-6})}{1.87\ (10)} = 0.008$$

$$k = \frac{28,650}{13} = 2,203 \text{ gpd/ft.}^2$$

FIGURE C-2



○ 1st Test 4/14/83 t = 695 min. = 0.48 days

△ 2nd Test 4/16/83 t = 470 min. = 0.33 days

$$T = \frac{528\ (Q)}{\Delta S} \qquad S = \frac{0.3\ T\ t}{r_o^2}$$

1st TEST

$$T = \frac{528\ (30)}{1.3} = 12,185 \text{ gpd/ft.}$$

$$S = \frac{0.3\ (12,185)\ 0.48}{(450)^2} = 0.0087$$

2nd TEST

$$T = \frac{528\ (30)}{0.6} = 26,400 \text{ gpd/ft.}$$

$$S = \frac{0.3\ (26,400)\ 0.33}{(600)^2} = 0.0073$$

Average Results

T (ave.) = 19,293 gpd/ft.

S (ave.) = 0.008

FIGURE C-3

Appendix D

Water Quality Analyses

APPENDIX D WATER QUALITY ANALYSIS

D.1 INTRODUCTION

Chemical analyses have not been performed on any groundwater samples obtained directly from the Universal City Station site. During the 1981 geotechnical investigation, a total of three water samples taken from Boreholes CEG-33 and CEG-35 were subjected to chemical analyses by Jacobs Laboratories (formerly PJB Laboratories in Pasadena, California). Boring CEG-33 is located about 2000 feet southeast of the proposed Station site while Boring CEG-35 is located about 3000 feet northwest of the Station site (please refer to Drawing No. 1 of the 1981 geotechnical report for the locations of these holes). Results of the chemical analyses performed during the 1981 investigation are summarized in this appendix. The primary purposes of obtaining and testing the water samples were as follows:

- Develop a current chemical constituent baseline for the groundwater along the subject Metro Rail Project alignment.
- Evaluate water chemicals that could have significant influence on design requirements.
- Identify chemical constituents for compliance with EPA requirements for future tunneling activities.

Chemical constituents tested by PJB Laboratories include:

- Major cations.
- Major anions.
- pH special test for boron.
- Conductivity.
- TDS.

D.2 ANALYSIS AND RESULTS

In our opinion, neither a complicated chemical analysis nor interpretation were required for the purpose of the 1981 geotechnical study. Therefore, standard water chemical analysis tests were performed by PJB Laboratories, the results of which are presented herein. The results of the water quality tests are summarized in Table D-1 and the data summary sheets.

TABLE D-1
SELECTED WATER QUALITY PARAMETERS

Boring No.	PVC Diam. (in.)	Depth Water Sampled (ft)	Date Sampled	pH @ 25° C	Total Dissolved Solids (ppm)	Sulfate SO_4 (ppm)	Boron, B (ppm)	Possible Water Type & Comments
33	1	21.8	02-12-81	7.2	1,504	693	0.66	Na/SO_4
33	2	23.3	02-11-81	7.5	1,154	538	0.38	Na/SO_4
35	1	95.0	02-12-81	7.6	2,605	19	3.2	Na/Cl

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Jacobs Laboratories

Water Quality

April 6, 1981

Converse Ward Davis Dixon
126 W. Del Mar Blvd.
P.O. Box 2268D
Pasadena, CA 91105

Lab No. P81-02-123
P81-02-142
P81-02-159
P81-02-186
P81-03-017

Attention: Buzz Spellman

Report of Chemical Analysis

The enclosed analytical results are for thirty (30) samples of ground water received by this laboratory on February 12, 17, 18, 20 and March 3, 1981. The samples were collected and delivered by Converse, Ward, Davis, Dixon personnel.

Cation/Anion balance was not acheived on many of the samples due to the presence of an unmeasured cation, probably aluminum or barium. This fact is reflected in the large difference between the milliequivalents of total hardness, (Milligrams $CaCO_3$/1 ÷ 50 = milliequivalents) and the summed milliequivalents of calcium and magnesium. These samples balance electrically using the total hardness in place of the calcium and magnesium. This indicates a cation (or cations) was not measured. The most common ions are aluminum and barium. If you so desired, we may analyze these samples for the missing element(s).

Respectfully submitted,

William, R. Ray
Manager, Water Laboratory

asl

Converse Ward Davis Dixon

Lab No. P81-03-017-3

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

Sample labeled: HOLE 33-1"

Conductivity: 2,130 μ mhos/cm

Turbidity: NTU

pH 7.2@25°C
pHs @60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	198	9.88
Magnesium, Mg	98	8.06
Sodium, Na	145	6.31
Potassium, K	5.8	0.15
		Total 24.40
Anions determined:		
Bicarbonate, as HCO_3	474	7.77
Chloride, Cl	94	2.66
Sulfate, SO_4	693	14.44
Fluoride, F	0.6	0.03
Nitrate, as N	0.3	0.00
		Total 24.90
Carbon dioxide, CO_2, Calc.	43	
Hardness, as $CaCO_3$	898	
Silica, SiO_2	31	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.66	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	1,504	

Converse Ward Davis Dixon

Lab No. P81-02-123-5

No. Samples : 6
Sampled By : Client
Brought By : Client
Date Received: 2-12-81

Sample labeled: HOLE 33-2"

Conductivity: 1,710 µ mhos/cm

Turbidity: NTU

pH 7.5 @ 25°C
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	94	4.69
Magnesium, Mg	68	5.59
Sodium, Na	186	8.09
Potassium, K	5.3	0.14
	Total	18.51
Anions determined:		
Bicarbonate, as HCO_3	329	5.39
Chloride, Cl	60	1.70
Sulfate, SO_4	538	11.21
Fluoride, F	0.7	0.04
Nitrate, as N	2.7	0.19
	Total	18.53
Carbon dioxide, CO_2, Calc.	15	
Hardness, as $CaCO_3$	515	
Silica, SiO_2	27	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.38	
Total Dissolved Minerals, (by addition: $HCO_3 \rightarrow CO_3$)	1,154	

Converse Ward Davis Dixon

Lab No. P81-02-142-7

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-17-81

Sample labeled: HOLE 35-1", 175'

Conductivity: 4,640 µ mhos/cm

Turbidity: NTU

pH 7.6 @ 25°C
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)		Milli-equivalents per liter
Cations determined:			
Calcium, Ca	56		2.79
Magnesium, Mg	67		5.51
Sodium, Na	795		34.58
Potassium, K	12		0.31
		Total	43.19
Anions determined:			
Bicarbonate, as HCO_3	343		5.62
Chloride, Cl	1,423		40.12
Sulfate, SO_4	19		0.40
Fluoride, F	0.3		0.02
Nitrate, as N	5.7		0.41
		Total	46.57
Carbon dioxide, CO_2, Calc.	12		
Hardness, as $CaCO_3$	560		
Silica, SiO_2	34		
Iron, Fe	< 0.01		
Manganese, Mn	< 0.01		
Boron, B	3.2		
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	2,605		

Appendix E

Geotechnical Laboratory Testing

APPENDIX E GEOTECHNICAL LABORATORY TESTING

E.1 INTRODUCTION

Laboratory geotechnical tests were performed on selected soil and bedrock samples obtained from the borings.

The tests performed may be classified into two broad categories:

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

The laboratory test data from the present investigation are presented in Table E-1, while data from the 1981 geotechnical investigation are presented in Table E-2. Results of laboratory tests that were performed during an investigation conducted by Converse Consultants in early to mid 1983 are presented in Table E-3. The soils and bedrock listed in these tables are described in Section 5.0 of the report.

E.1.1 Data Analysis

The summary of laboratory test results is presented in Tables E-1, E-2, and E-3. Figures E-1 through E-4 summarize strength and modulus data for fine-grained Alluvium. Figure E-5 summarizes the effective strength data for coarse-grained Alluvium. Figure E-6 is a compilation of modulus data from laboratory tests performed on coarse-grained Alluvium and Figures E-7 through E-10 summarize strength and modulus data for the Topanga Formation bedrock. It should be noted that test results from this investigation and from other design units have been combined when, in our judgment, it was considered appropriate to do so.

E.2 INDEX AND IDENTIFICATION

E.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-2487-69 test method. When necessary to substantiate visual classifications, tests were conducted in accordance with the ASTM D-2487-69 test method.

E.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 E-11 through E-16.

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.

E.2.3 Atterberg Limits

Atterberg Limit Tests were performed on selected soil and bedrock samples to evaluate their plasticity and to aid in their classification. The testing procedure was in accordance with ASTM D-423-66 and D-424-59 test methods. Test results are presented on Figures E-17 and E-18 and Tables E-1 and E-2.

E.2.4 Moisture Content

Moisture content determinations were performed on selected soil and bedrock samples to assist in their classification and to evaluate groundwater location. The testing procedure was a modified version of the ASTM D-2216 test method. Test results are presented on Tables E-1, E-2, and E-3.

E.2.5 Unit Weight

Unit weight determinations were performed on selected undisturbed soil and bedrock 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 then 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 E.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables E-1, E-2, and E-3.

E.3 ENGINEERING PROPERTIES: STATIC

E.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-2166-66 test method. Results of the unconfined compression tests are presented in Tables E-1, E-2, and E-3.

E.3.2 Triaxial Compression

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

E.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 desired maximum strain 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 sample was loaded until failure occurred. Results of the triaxial compression tests are presented in Figures E-19 through E-31.

E.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.

Progressive direct shear tests were performed on selected undisturbed samples. 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 direct shear tests are summarized on Tables E-1 and E-2 and Figure E-1.

E.3.4 Free Swell

Free swell tests were performed on selected undistured samples of cohesive, potentially expansive soils. The test procedure entailed placing the undisturbed soil sample in a consolidometer, applying a vertical confining load, and inundating the sample with tap water. The resulting one-dimensional swell of the sample was measured and recorded. Results of these tests are presented on Table E-1.

E.3.5 Consolidation

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

Apparatus used for the consolidation test is designed to receive the 1-inch high brass rings directly. Porous stones were placed in contact with both sides of the specimens to permit ready addition or release of water. Loads are 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 E-32 through E-36.

E.3.6 Permeability

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

E.3.7 Porosity

Porosity, or void ratio, of selected undisturbed samples was determined by measuring the dry unit weight and specific gravity, then calculating the void ratio, e, and porosity, n, using the following formula:

$$e = (1 - Vs)/Vs, \text{ where } Vs = (\gamma_d)/(G \times \gamma_w) \text{ and } n = e/(1 + e)$$

γ_w = unit weight of water

γ_d = unit dry weight of the soil

G = specific gravity of soil solids.

In some cases, an assumed average value for the specific gravity, based on the measured values for other specimens, was used for the porosity calculation.

TABLE E-1
LABORATORY TEST DATA

BORING NO.	SAMPLE NO.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
							LL	PI			ø, deg	c, ksf						
34-1	C-1	8.0	Sandy Clay	A	105	17					25	0.45						
	PB-1	14.5	Clayey Sand	A	110	17	28	10		2.37					X	X		X
	C-2	18.0	Sandy Clay	A	102	24				1.28								
	C-3	28.0	Sandy Clay	A	96	27											X	
	C-4	38.0	Clayey Sand	A	107	20					(1)							
	C-5	53.0	Silty Sand	A	97	29												
	PB-5	63.8	Silty Sand /Sand with gravel	A	113	15			$2.99\text{x}10^{-6}$						X			X
	C-6	72.9	Claystone	Tt	106	21					(1)							
	C-7	88.7	Claystone/Sandstone	Tt	105	22												
	PB-7	94.5	Claystone	Tt	107	21	38	14							X	X		X
	C-8	98.9	Claystone/Sandstone	Tt	113	18					(1)		0.6(0.5)				X	
34-2	C-1	8.0	Sandy Clay	A	106	16				3.62								
	C-2	18.0	Clayey Sand	A	100	23					29	0.25						
	PB-2	24.5	Sandy Clay/Clayey Sand	A	99	25				0.32					X			X
	C-3	28.0	Sandy Clay/Clayey Sand	A	98	28				0.44								

NOTE: (1) One point direct shear test

TABLE E-1 (Cont.)
LABORATORY TEST DATA

BORING NO.	SAMPLE NO.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
							LL	PI			ø, deg	c, ksf						
34-2	C-4	38.0	Sandy Clay	A	102	23											X	
	C-5	53.0	Silty Sand/Clayey Sand	A	93	35									X	X		X
	PB-6	81.0	Claystone	Tt	80	39				1.81								
	C-6	84.5	Claystone	Tt	85(3)	35					(1)							
34-3	C-2	18.0	Silty Clay	A	97	29											X	
	C-3	28.0	Sandy Clay/Clayey Sand	A	105	19					30	0.40						
	PB-3	34.5	Sandy Clay	A	95	26	34	13		0.72(2)					X	X		X
	C-4	38.0	Clayey Sand	A	103	24												
	C-6	68.3	Claystone/Siltstone	Tt	108	19					19	2.5						
	PB-6	76.6	Claystone/Siltstone	Tt	112	18												X
	PB-9	105.4	Claystone/Siltstone	Tt	106	20									X			X
	PB-10	115.5	Claystone/Siltstone	Tt	114	15				9.98								
34-4	C-1	8.0	Sandy Clay	A	106	17					34	0.0						
	PB-1	14.5	Clayey Sand	A	104	23	28	10							X	X		X
	C-2	18.0	Silty Clay	A	98	26				0.98								
	PB-4	44.5	Silty Clay	A	85	35				2.12								

NOTES: (2) Unconfined test result low due to presence of sand lenses.
(3) Sample was disturbed.

TAB[illegible]-1 (Cont.)
LABORATORY TEST DATA

BORING NO.	SAMPLE NO.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
							LL	PI			ø, deg	c, ksf						
34-4	C-5	53.0	Sand	A	100	22					32	0.0					X	
	PB-5	64.5	Silty Sand/Sand with gravel	A	101	24			1.49 x 10^{-6}						X			X
	C-6	72.5	Sandstone	Tt	112	16					(1)							
	PB-6	84.0	Claystone	Tt	99	25				5.72								
	PB-8	104.2	Claystone	Tt	105	20												
	PB-9	114.0	Claystone/Sandstone	Tt	109	18									X			X
34-5	PB-1	17.5	Silty Clay	A	100	24	42	22		1.63								
	C-1	46.0	Sandy Silt	A	93	32					37	0.0						
	PB-5	72.5	Claystone	Tt	90	24			2.6 x 10^{-7}						X	X		X
	PB-7	86.5	Claystone	Tt	111	18												X
	C-5	95.9	Claystone	Tt	113	17					(1)		3.0(0.5)				X	
	C-6	105.4	Sandstone	Tt	110	18												
	C-7	115.4	Sandstone	Tt	106	19											X	

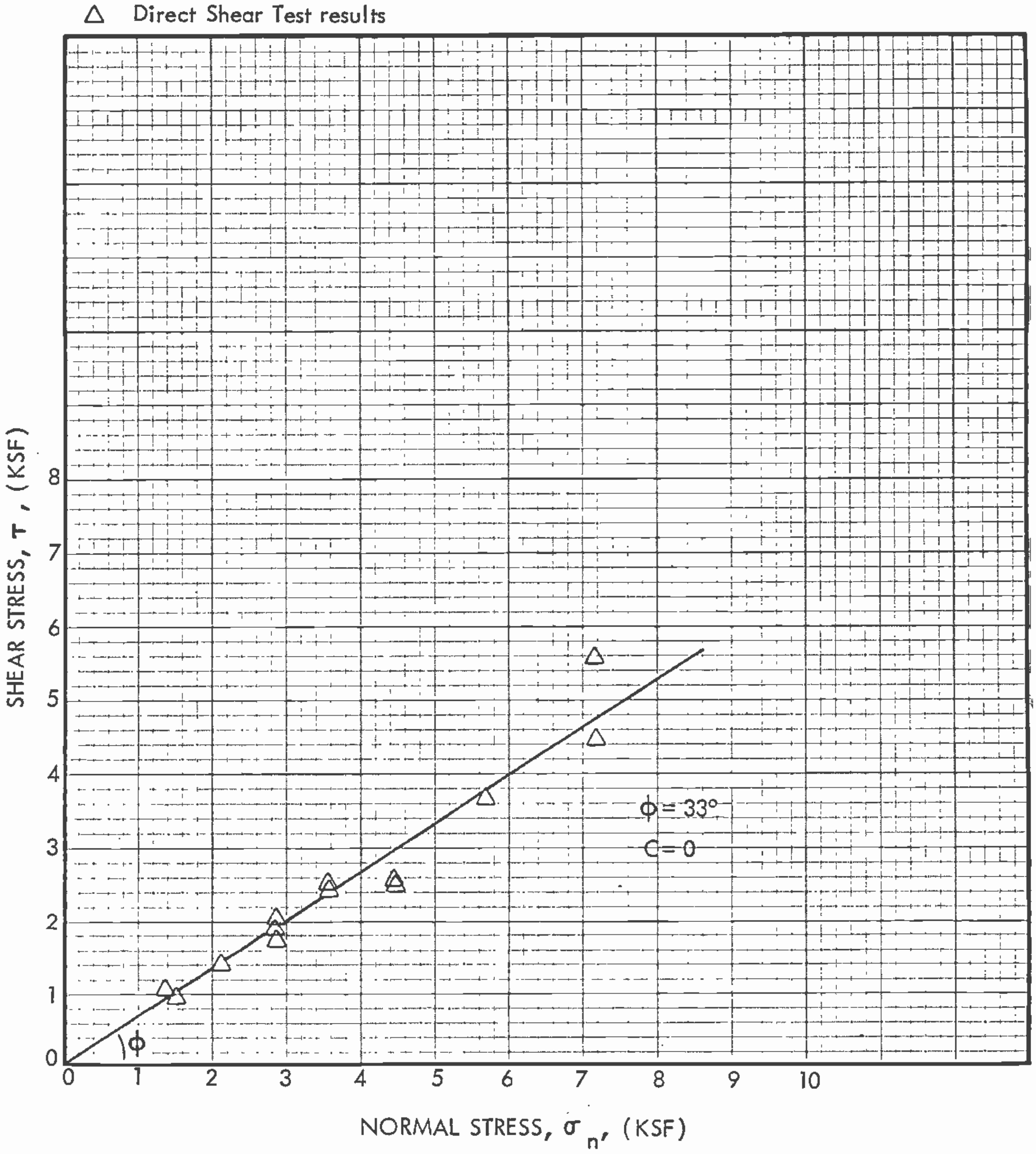
TABLE E-2 COMPREHENSIVE LIST OF SOILS ENGINEERING PROPERTIES FROM LABORATORY TESTS

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Particle Size Cumulative % Passing Sieve No.			Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Coefficient of Consolidation Cc (in/in Per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear		One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
							LL	PI	4	40	200						ø, deg	c, ksf					
34	C1	21	Brown sandy silt	A_2	100	24																	
	J3	21	Fine sandy silt	A_2					100	99	86												
	C2	31	Silt with gravel	A_2	108	20	29	9	100	98	80		4.8E-7			35.8							cup
	J7	40	Silty sand and gravel	A_1					79	32	7												
	S1	71		2(t)			52	24															
	S2	91		2(t)			49	27															
	S3	101		2(t)			49	27															
	S8	121		2(t)			64	34															
	S11	129		2(t)			44	15															
	S34	179		2(t)			87	63															

TABLE E-3
LABORATORY TEST RESULTS
(CCI; 1983)

Boring No.	Sample No.	Soil Classification	Depth (ft)	Moisture Content (%)	Dry Density (pcf)	Unconfined Compressive Strength (ksf)
34A	C1	CL	10	16	113	5.36
	C2	SC	30	20	109	2.28
	C3	Sandstone	50	12	127	6.23
	C5	Sandstone	80	10	128	-----
	PB2	Sandstone	90	9	131	16.7
	C6	Sandstone	100	9	129	-----
34B	C1	CL	20	24	101	1.27
	C2	ML/CL	40	34	89	1.06
	C3	SM	70	14	130	-----
	PB1	Siltstone	80	22	101	3.58
	C4	Siltstone	90	20	109	3.62
	C5	Siltstone	120	16	116	-----
34D	C1	SP	32	23	102	-----
	PB1	Siltstone	50	23	95	1.44
	PB2	Siltstone	60	20	107	.46*
	C2	Siltstone	61	17	110	12.0
	PB3	Siltstone	70	19	105	1.20
	C3	Siltstone	80	22	96	2.33
	C4	Siltstone	100	26	102	5.31

*Failed on sandy bedding plane.



SUMMARY OF DIRECT SHEAR TEST RESULTS - ALLUVIUM

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

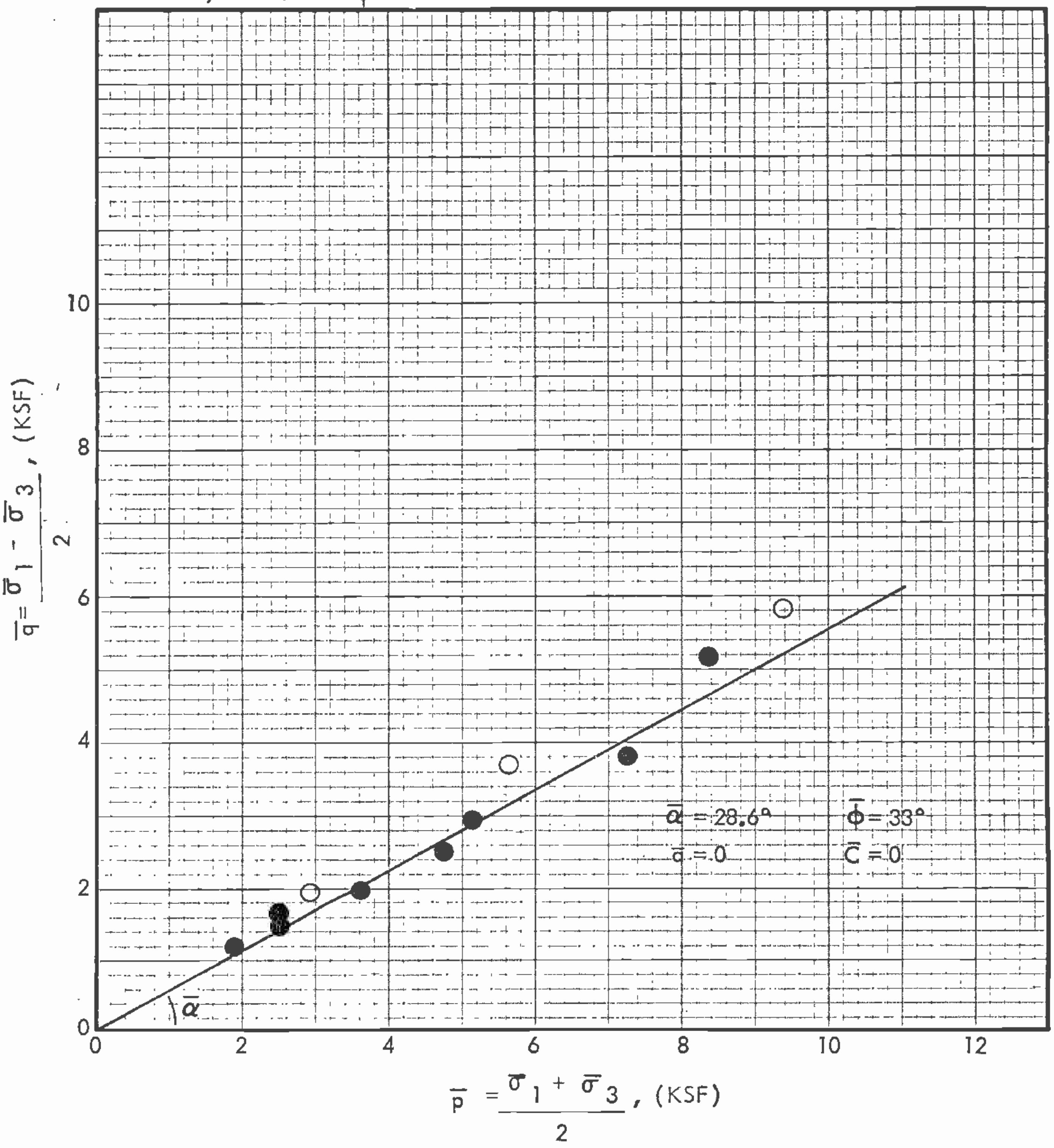
Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
E-1

Approved for publication ________ by ________

● Triaxial Test results

NOTES: 1) Solid symbols are from this investigation
2) Open symbols are from 1981 investigation
3) $\tan \alpha = \sin \phi$



SUMMARY OF EFFECTIVE STRENGTH DATA - FINE-GRAINED ALLUVIUM

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

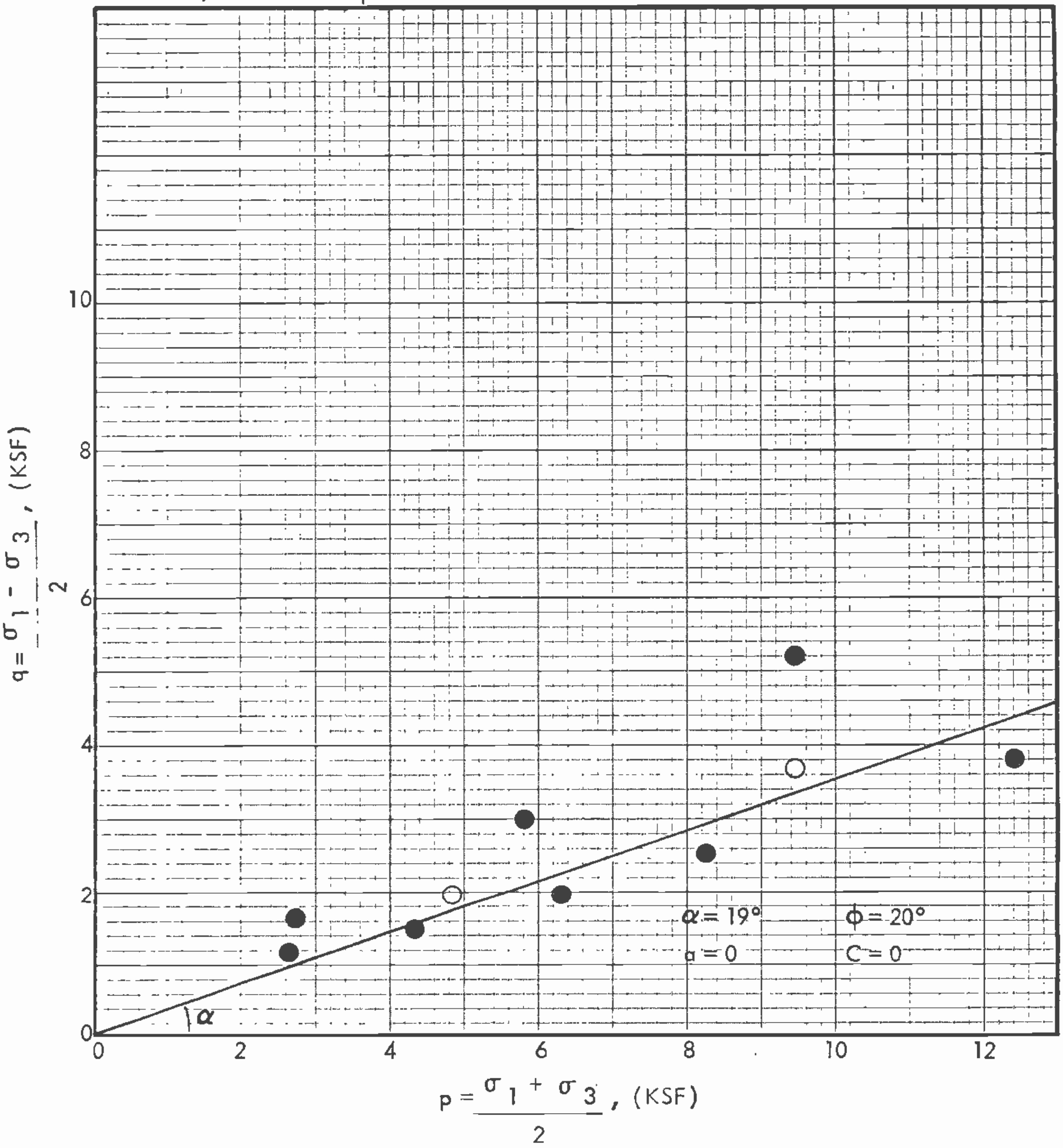
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
E-2

● Triaxial Test results

NOTES: 1) Solid symbols are from this investigation
2) Open symbols are from 1981 investigation
3) tan α = sin ϕ



SUMMARY OF TOTAL STRENGTH DATA - FINE-GRAINED ALLUVIUM

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

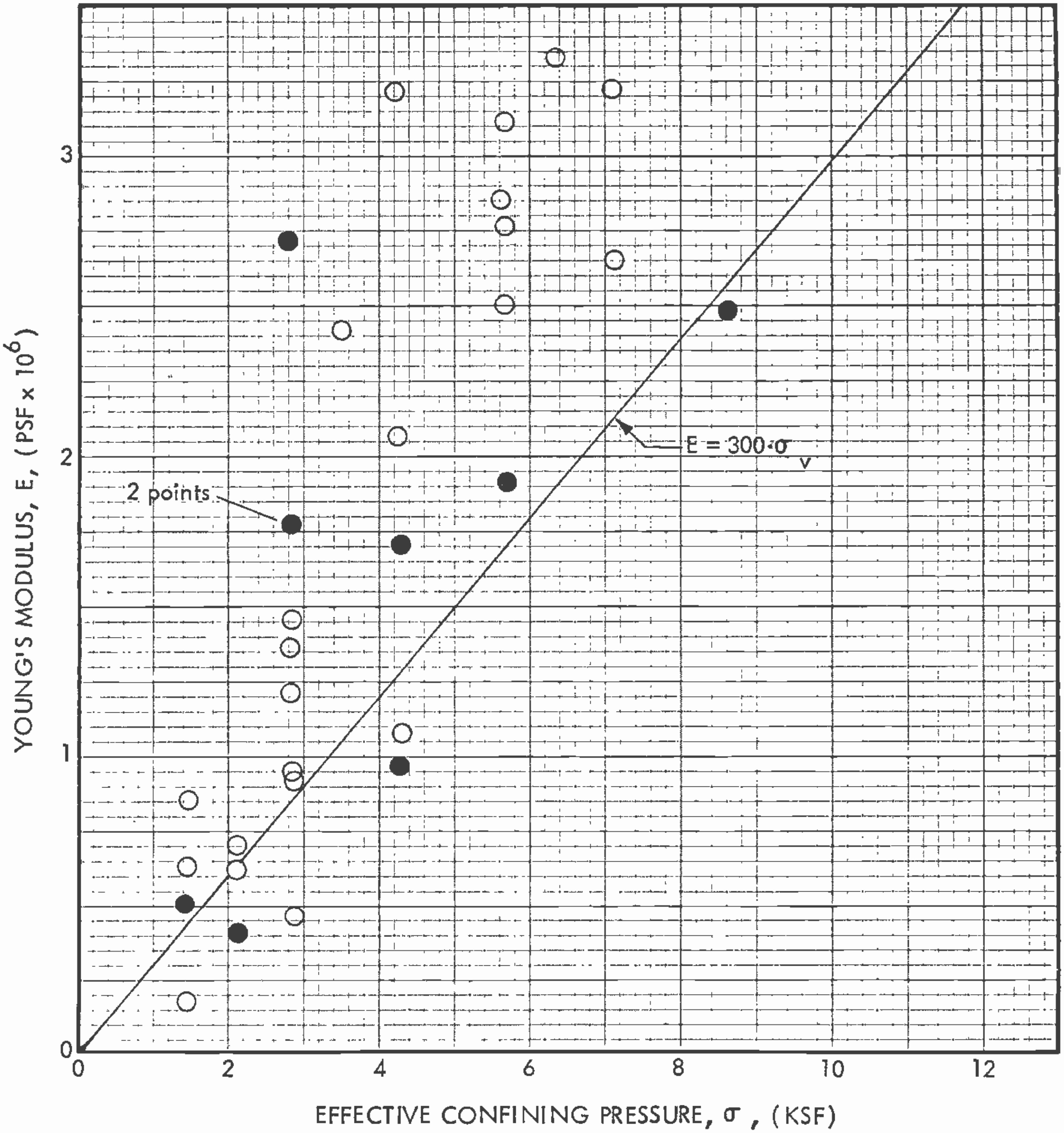
Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
E-3

Approved for publication ______ by ______

● Triaxial Test results

NOTES:
1) Solid symbols are from this investigation
2) Open symbols are from 1981 investigation and other design units
3) Modulus calculated from unload/reload stage of Triaxial Tests when performed



SUMMARY OF MODULUS DATA - FINE-GRAINED ALLUVIUM

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

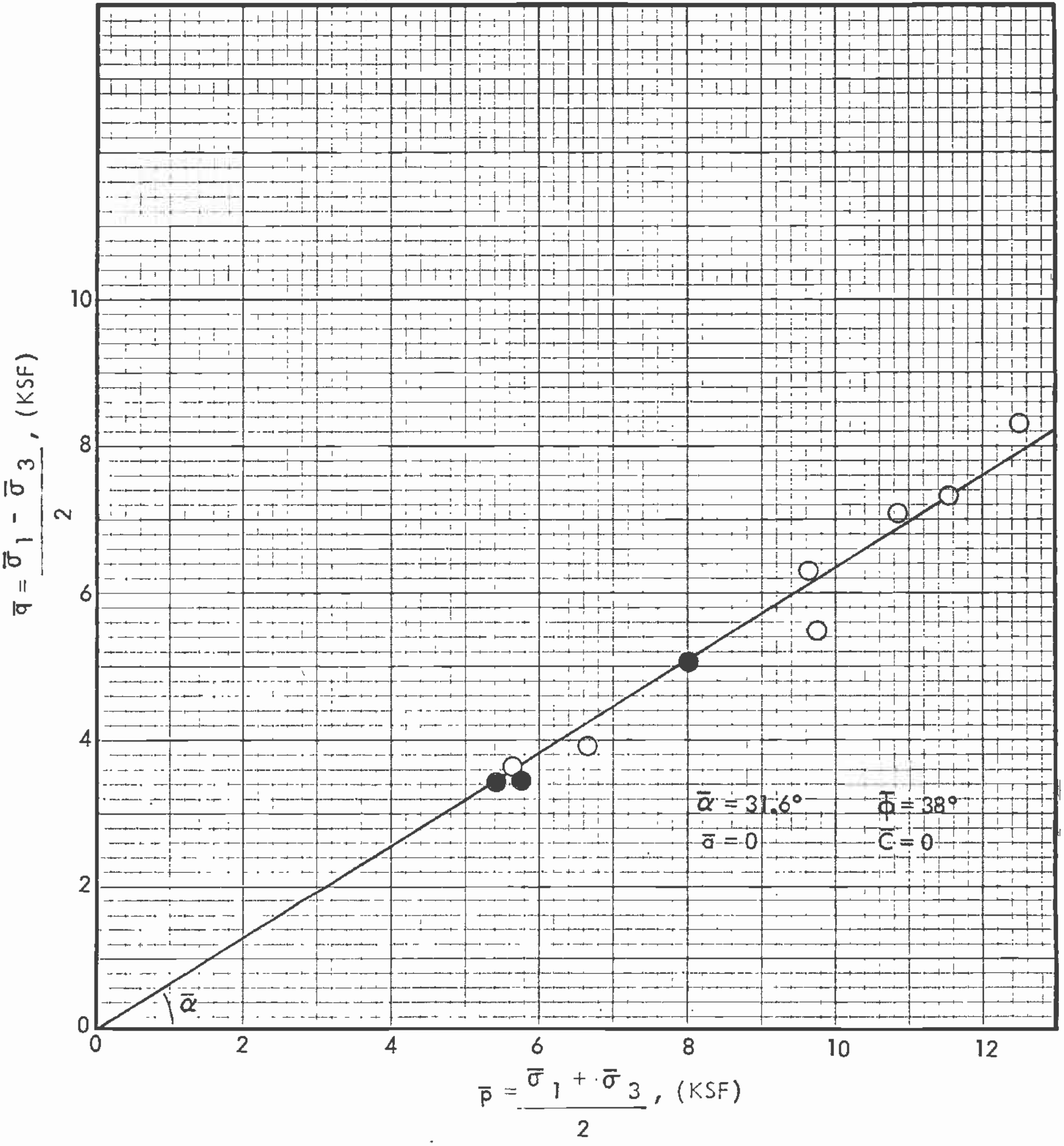
Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
E-4

Approved for publication ______ by ______

● Triaxial Test results

NOTES: 1) Solid symbols are from this investigation
2) Open symbols are from other design units
3) $\tan \alpha = \sin \phi$



SUMMARY OF EFFECTIVE STRENGTH DATA - COARSE-GRAINED ALLUVIUM

DESIGN UNIT A425
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

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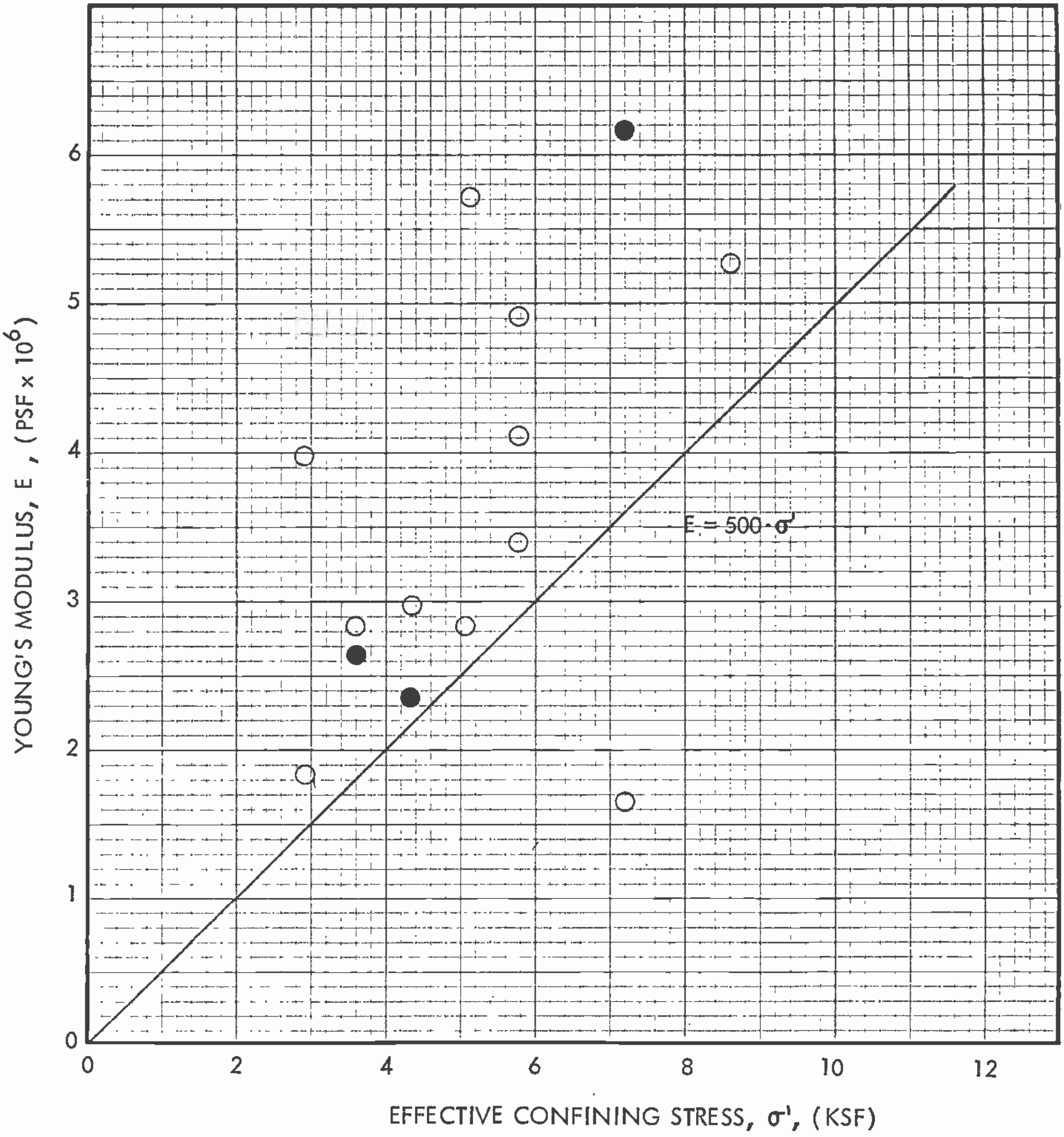
Figure No.
E-5

● Triaxial Test results

NOTES: 1) Solid symbols are data from this investigation
2) Open symbols are data from other design units
3) Modulus calculated from unload/reload stage of Triaxial Tests when performed



SUMMARY OF MODULUS DATA - COARSE-GRAINED ALLUVIUM

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Figure No.
E-6

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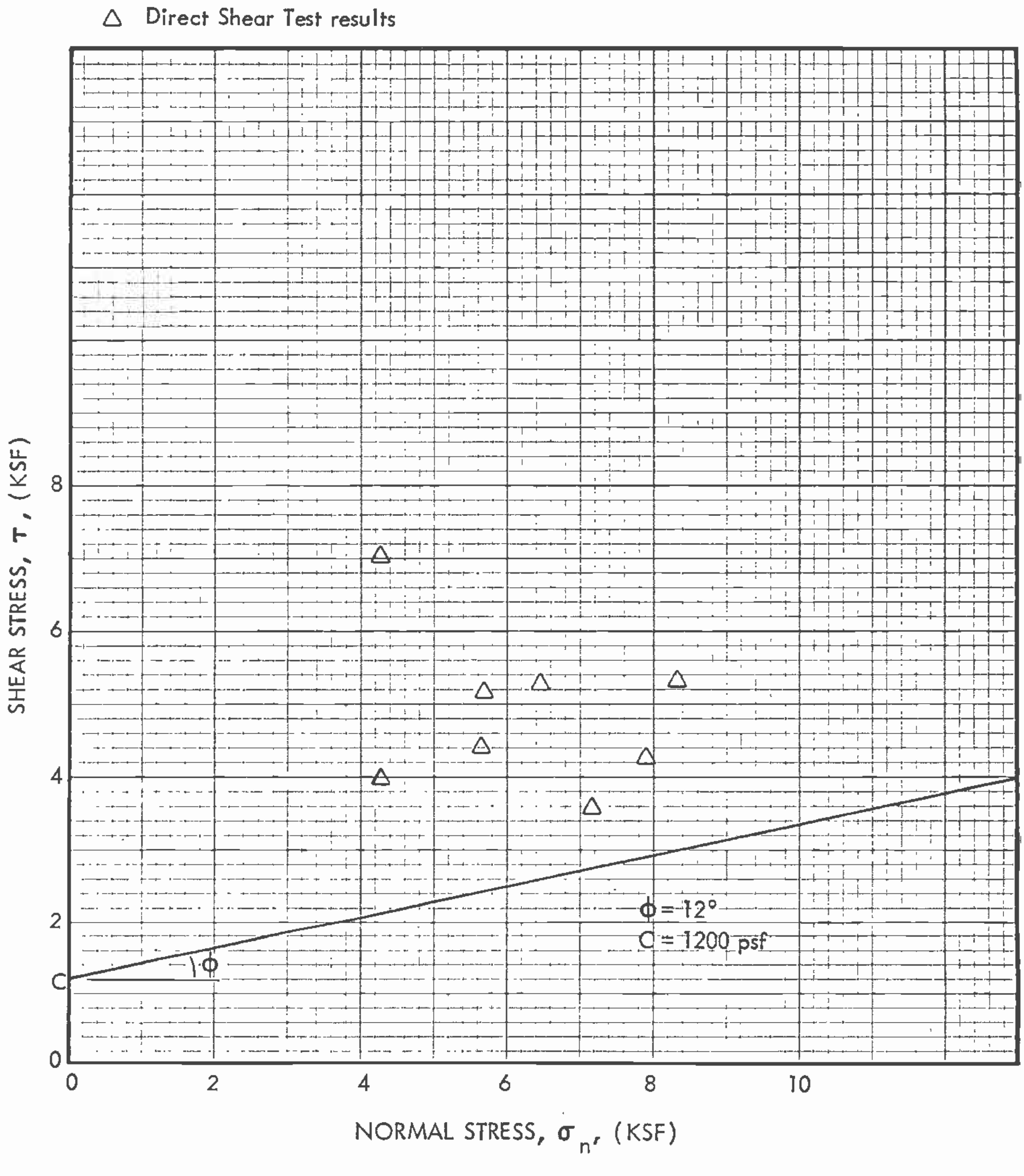

SUMMARY OF DIRECT SHEAR TEST RESULTS - TOPANGA FORMATION

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Figure No.
E-7

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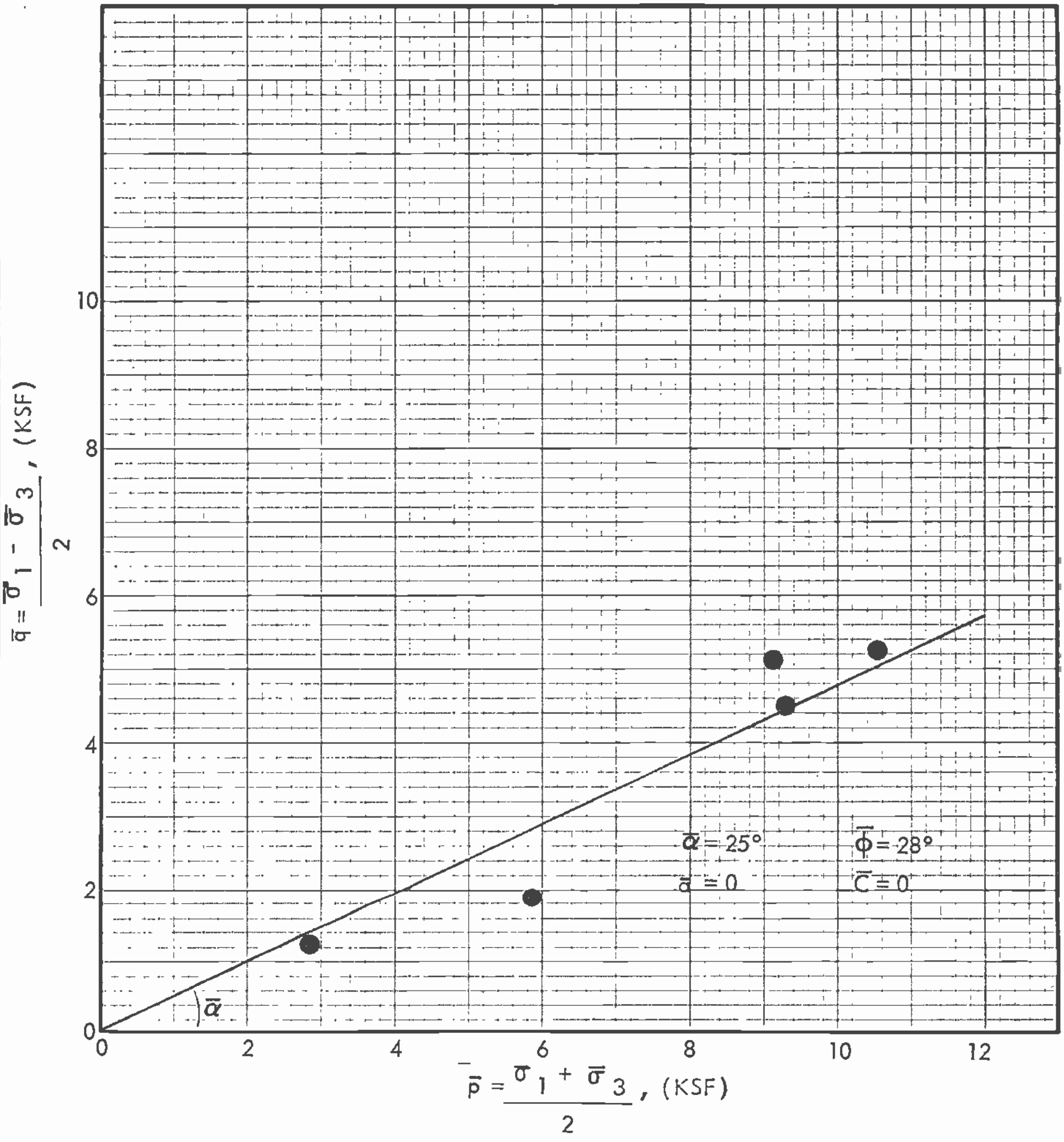
● Triaxial Test results

NOTE : 1) $\tan \alpha = \sin \phi$



SUMMARY OF EFFECTIVE STRENGTH DATA - TOPANGA FORMATION

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Figure No.
E-8

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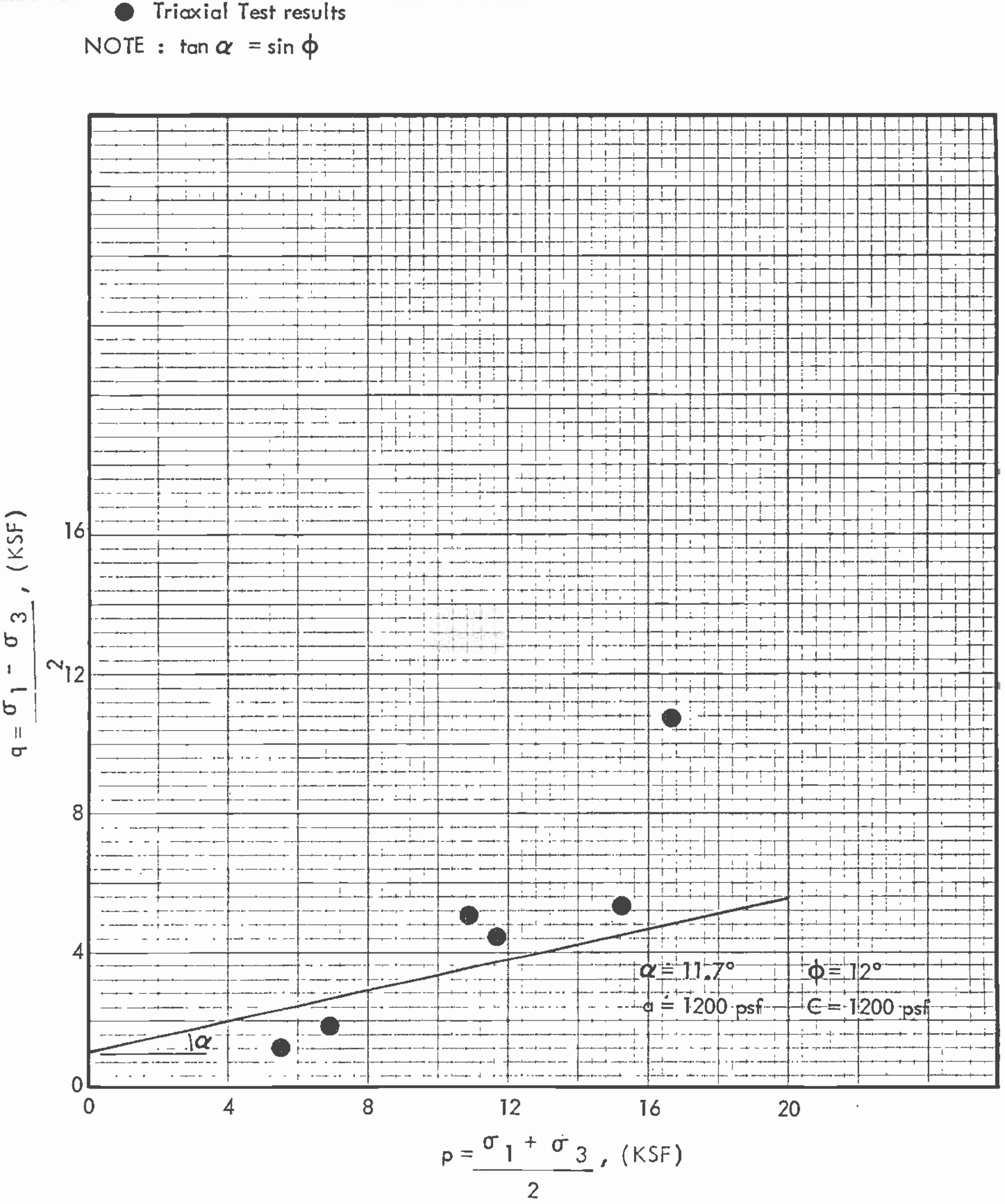

SUMMARY OF TOTAL STRENGTH DATA - TOPANGA FORMATION

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Figure No.
E-9

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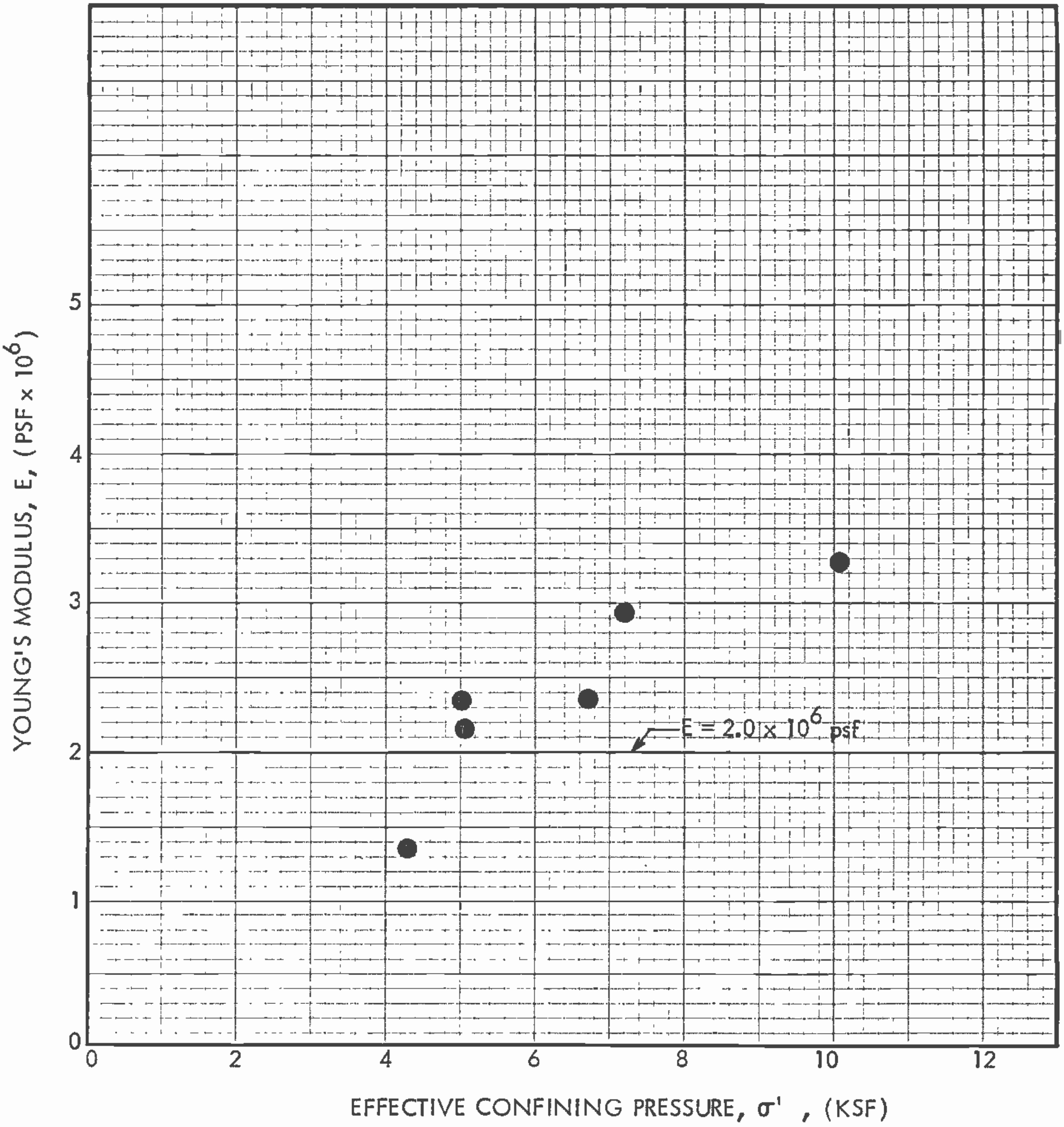
● Triaxial Test results

NOTE : 1)Modulus calculated from unload/reload stage of Triaxial Test



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SUMMARY OF MODULUS DATA - TOPANGA FORMATION

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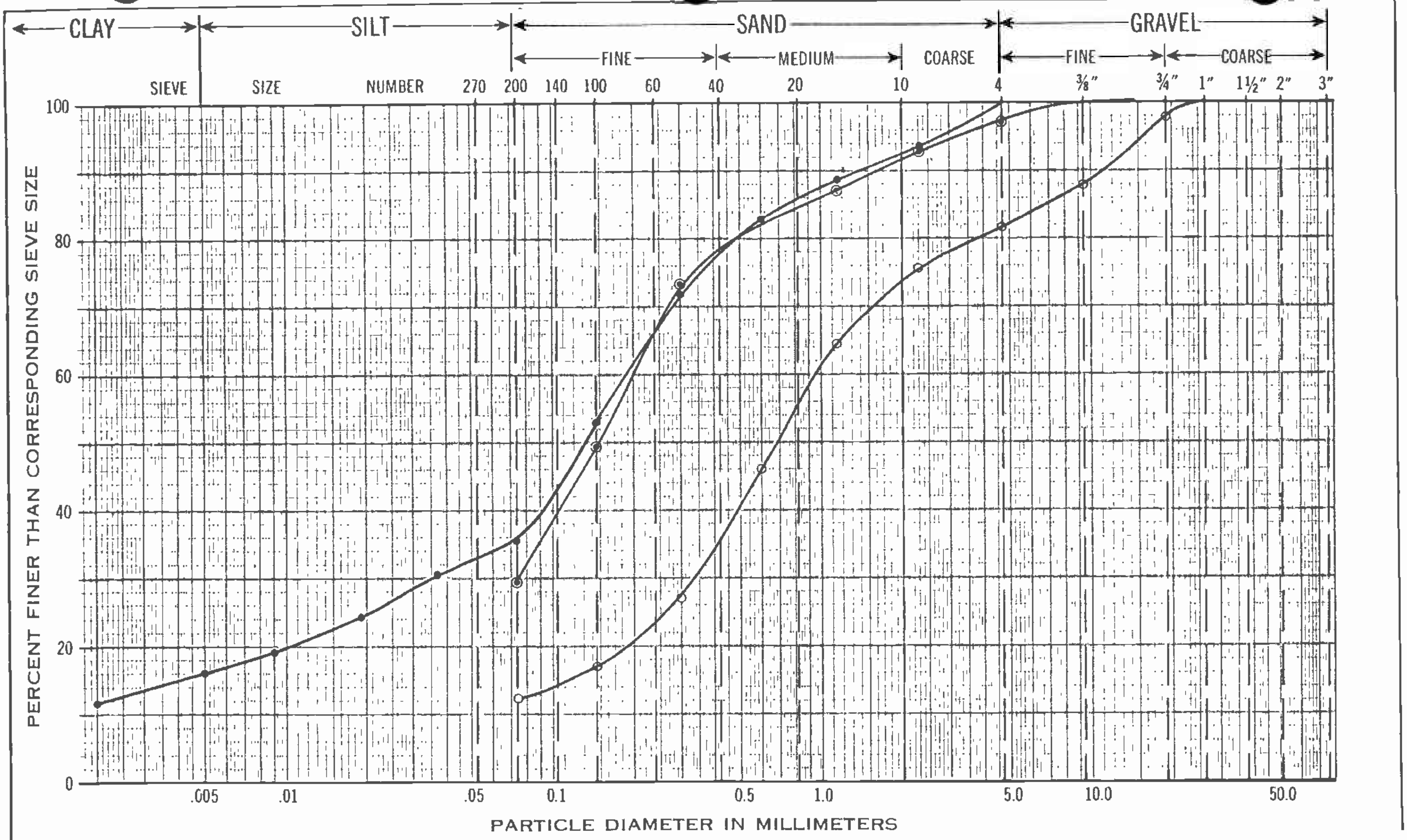
Figure No.
E-10

FORM NO. D-3-1 Approved for publication ________ by ________



SYMBOL	BORING	SAMPLE	DEPTH, feet
●	34-1	PB-1	12.0-14.5
○	34-1	PB-5	62.0-63.8
◉	34-1	PB-7 *	92.0-94.5

*Coarse fraction predominated by weathered stone which did not break-down in washing process.

GRAIN-SIZE DISTRIBUTION CHART

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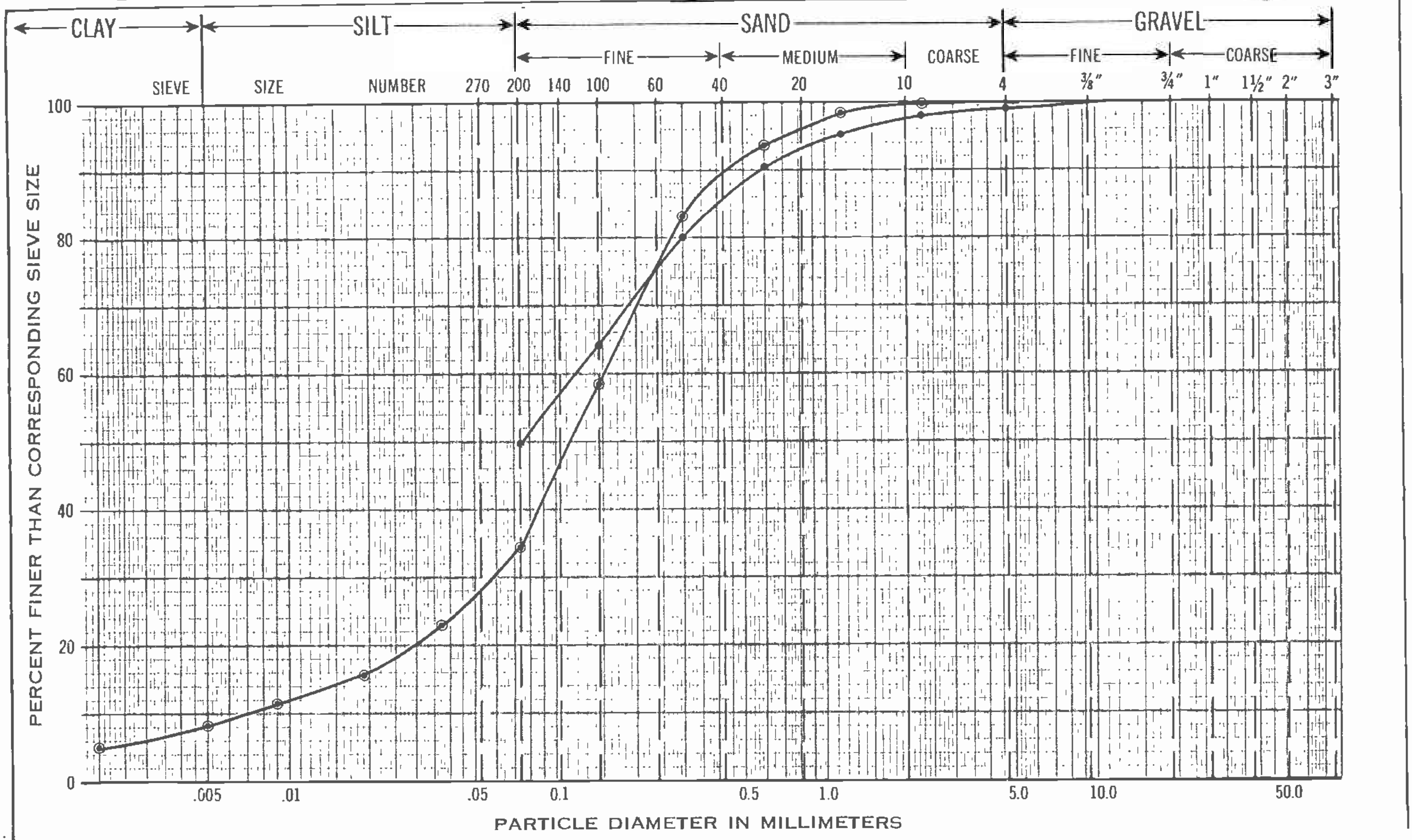
Drawing No.
E-11

FORM NO. D-3 Approved for publication ____________ by ____________



SYMBOL	BORING	SAMPLE	DEPTH, feet
●	34-2	PB-2	22.0-24.5
◉	34-2	C-5	52.5-53.0

GRAIN-SIZE DISTRIBUTION CHART

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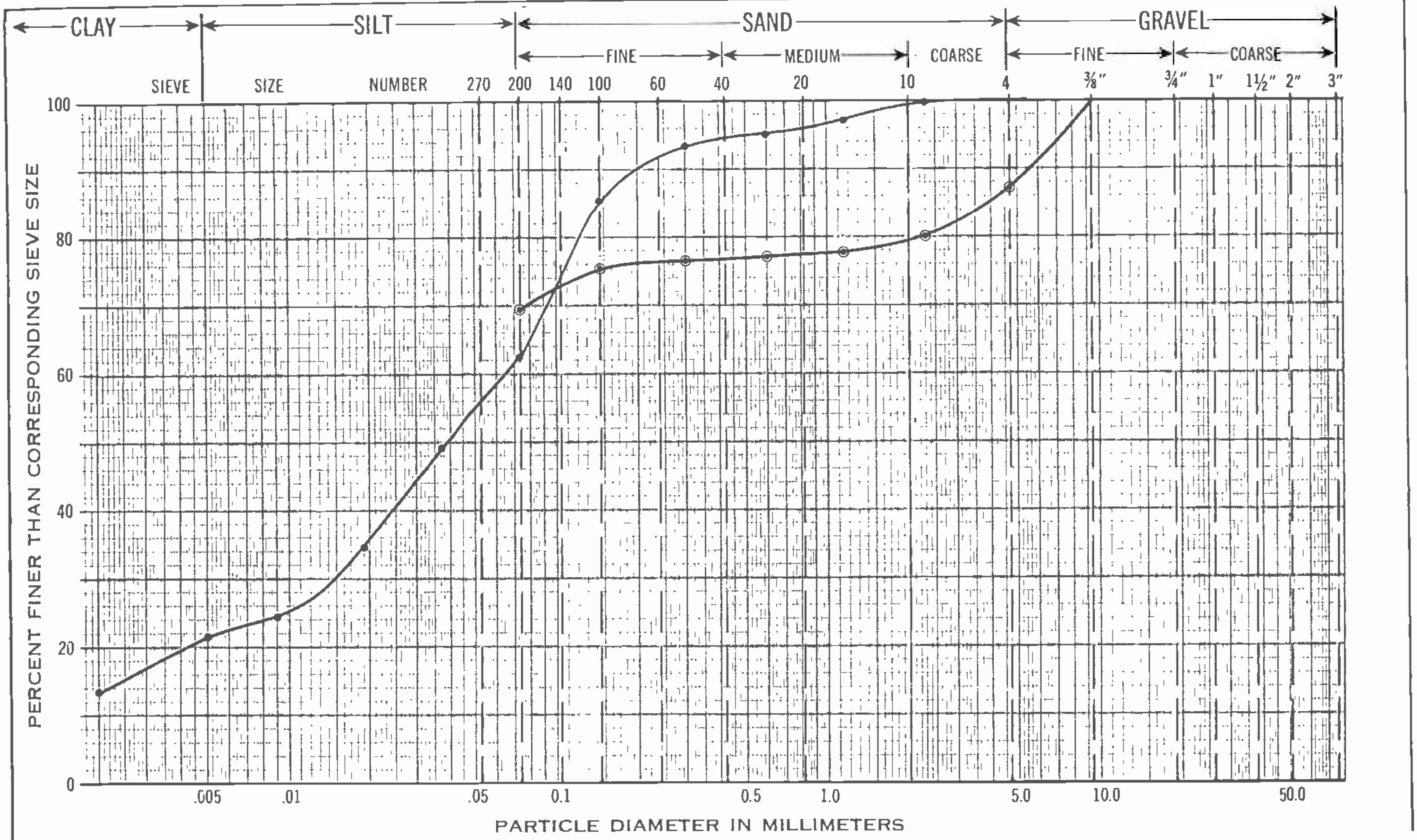
Drawing No.
E-12

FORM NO. D-3- Approved for publication ______ by ______



SYMBOL	BORING	SAMPLE	DEPTH, feet
●	34-3	PB-3	32.0-34.5
◉	34-3	PB-9	114.0

GRAIN-SIZE DISTRIBUTION CHART

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Project No.
83-1140

Drawing No.
E-13

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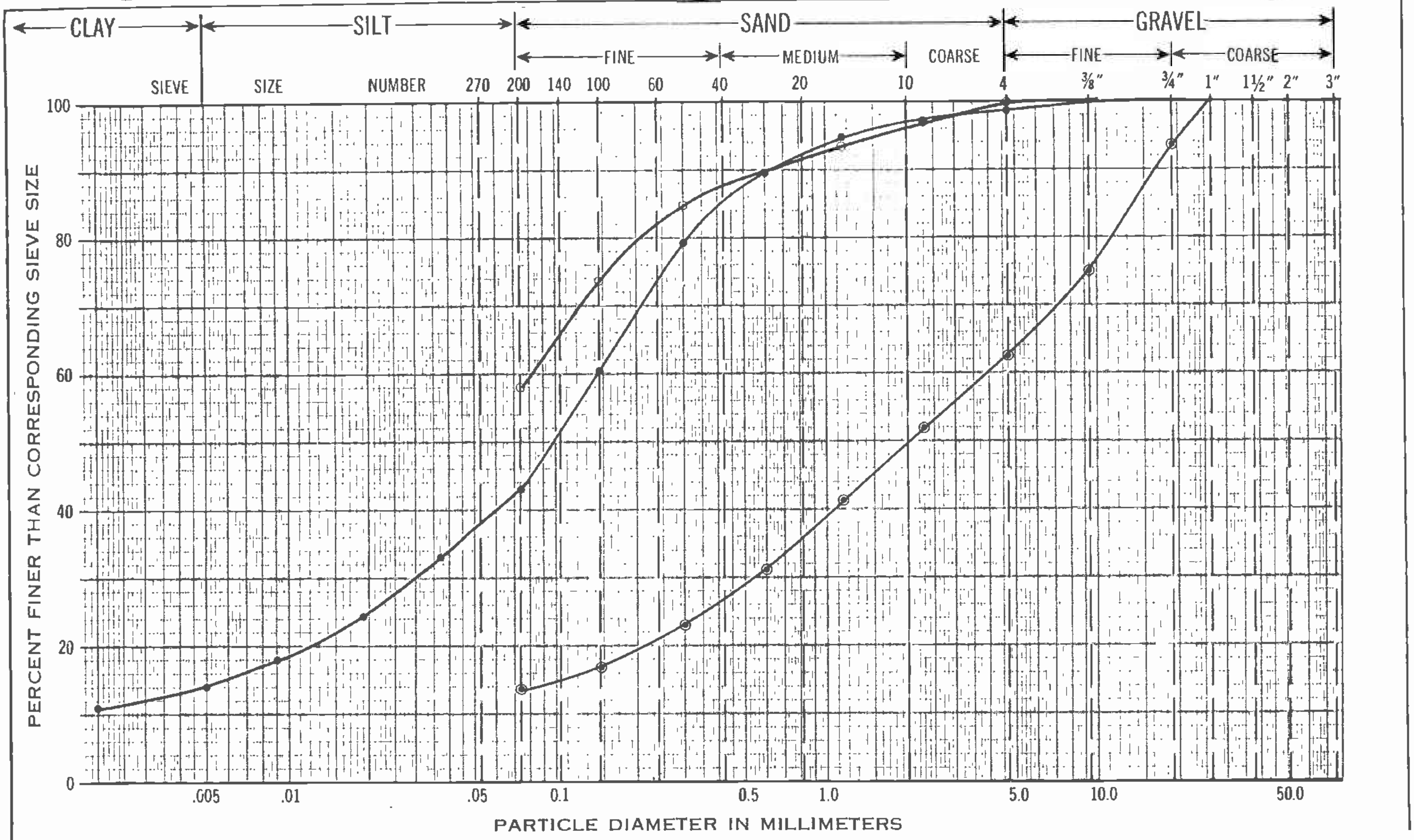

SYMBOL	BORING	SAMPLE	DEPTH, feet
●	34-4	PB-1	12.0-14.5
◉	34-4	PB-5	62.0-64.5
○	34-4	PB-9	111.5-114.0

GRAIN-SIZE DISTRIBUTION CHART

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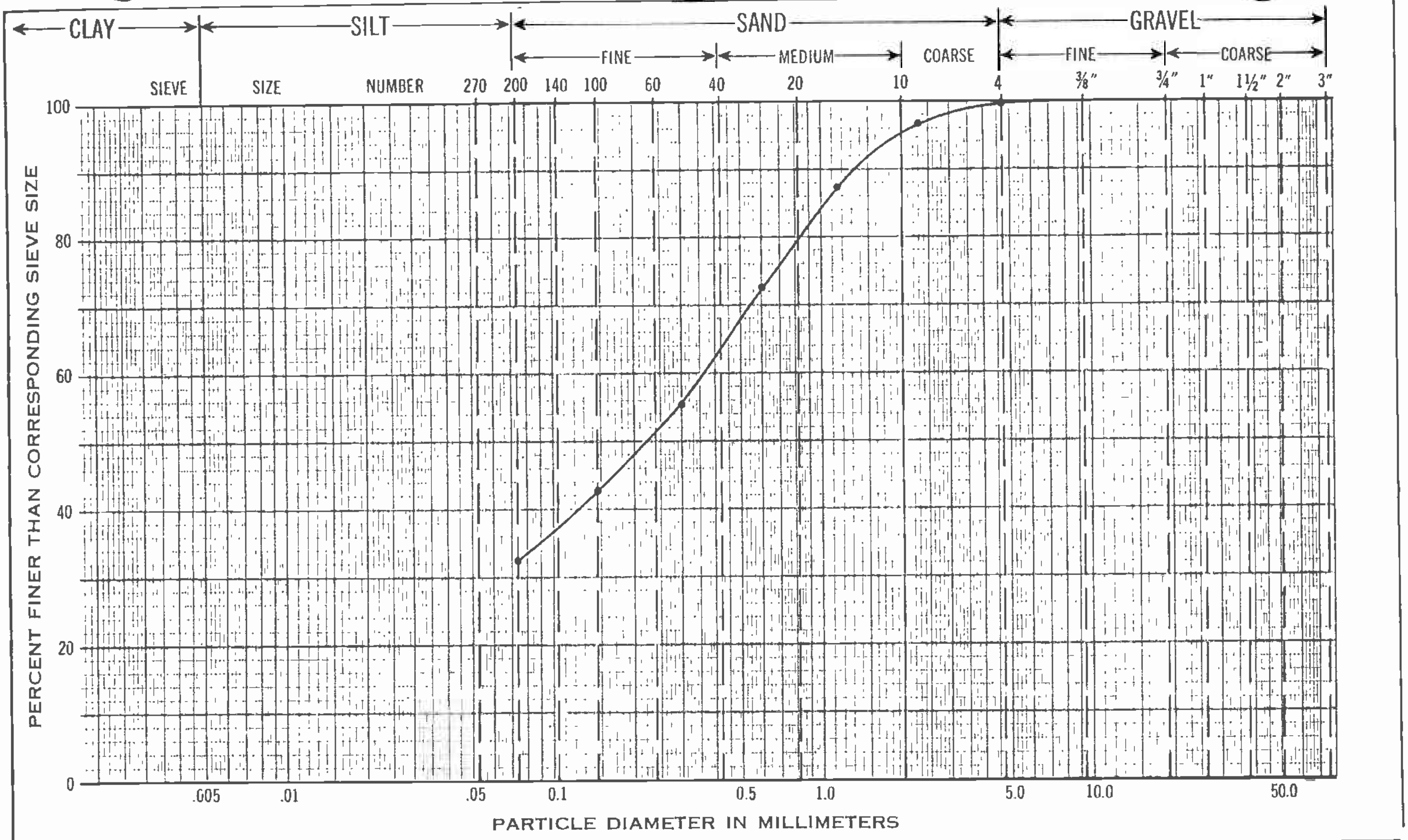
Drawing No.
E-14

FORM NO. D-3-1 Approved for publication ______ by ______



SYMBOL	BORING	SAMPLE	DEPTH, feet
●	34-5	PB-5 *	70.0-72.5

* Coarse fraction predominated by weathered stone which did not break-down in washing process.

GRAIN-SIZE DISTRIBUTION CHART

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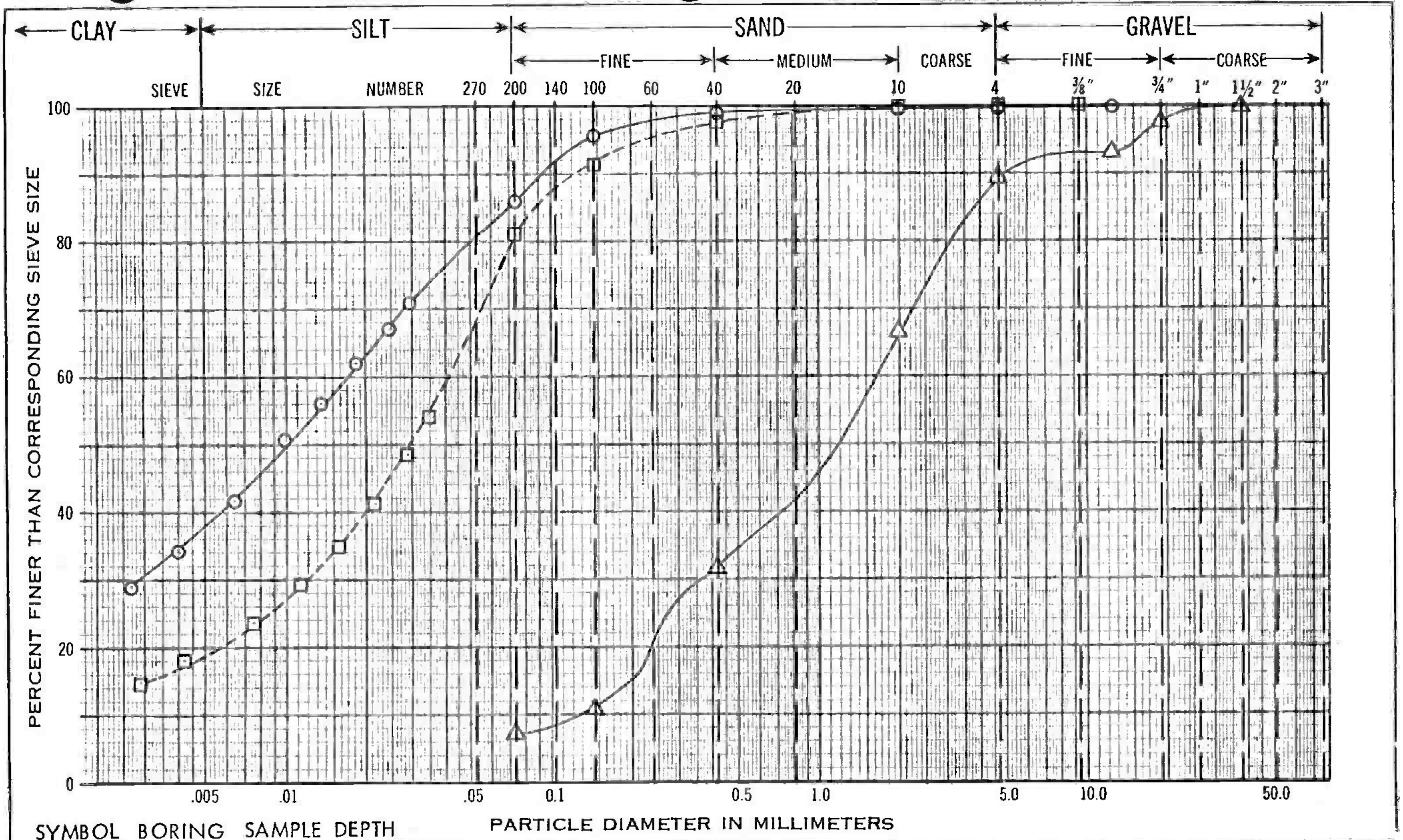
Drawing No.
E-15

FORM NO. D22 Approved for publication ________ by ________

GRAIN-SIZE ANALYSIS OF SOILS ASTM [illegible]IGNATION: D422



SYMBOL	BORING	SAMPLE	DEPTH
—○—	34	J-3	21'
- -□- - -	34	C-2	30'
—△—	34	J-7	40'

GRAIN-SIZE DISTRIBUTION CHART

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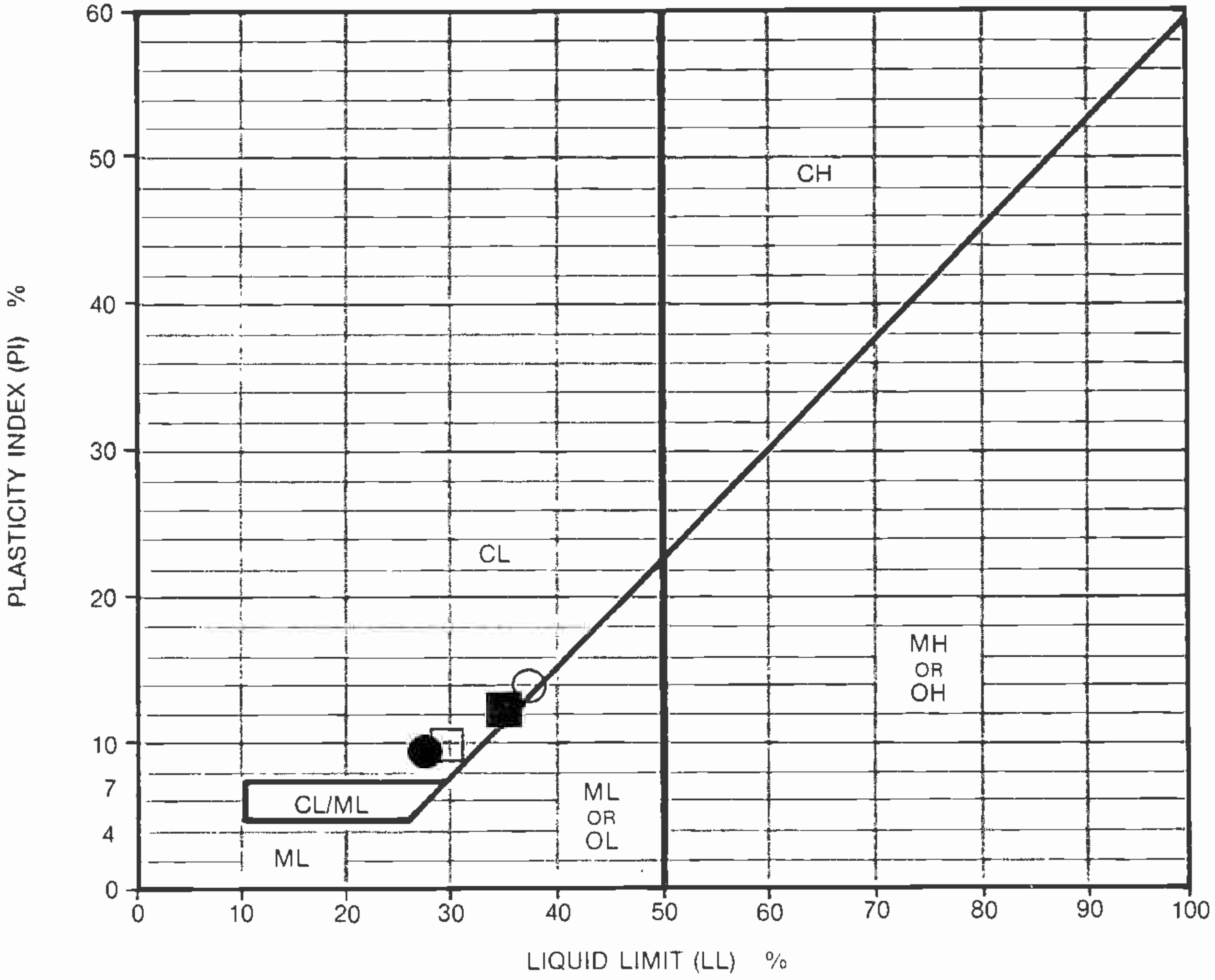
Figure No
E-16



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
●	BH 34-1, 12.0'-14.5' (CL)	28.0	18.3	9.7	35.6
○	BH 34-1, 92.0'-94.5' (CL-claystone)	37.5	24.0	13.5	29.7
■	BH 34-3, 32.0'-34.5' (CL)	34.3	21.7	12.6	62.3
□	BH 34-4, 12.0'-14.5' (SC)	28.0	18.0	10.0	43.0

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PLASTICITY CHART

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Project No
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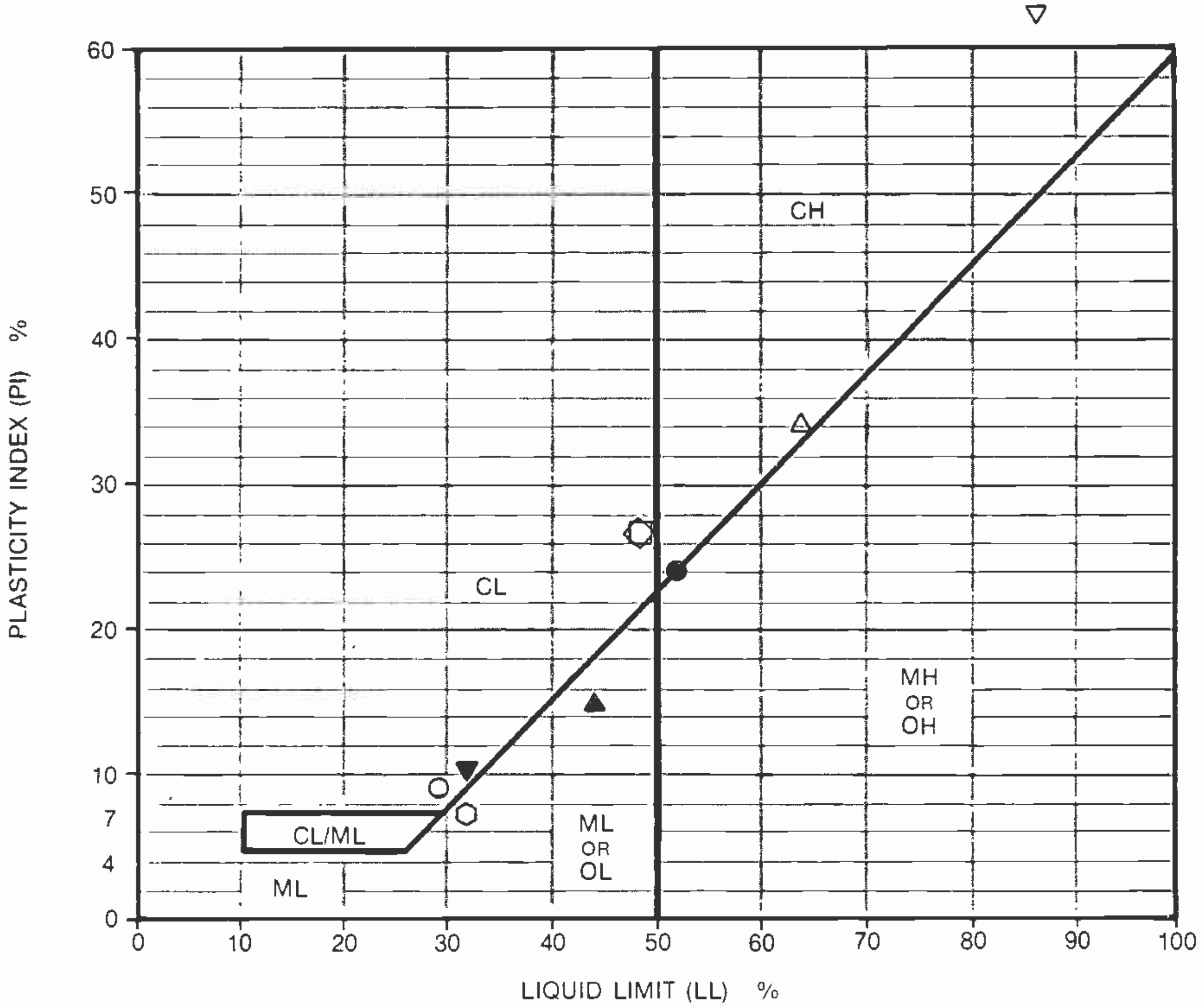
Figure No.
E-17



Symbol	Classification and Source			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
○	BH 34,	30'	(CL)	29	20	9	80
●	BH 34,	70'	(CH/MH)	52	28	24	-
□	BH 34,	90'	(CL)	49	22	27	-
◇	BH 34,	100'	(CL)	49	22	27	-
△	BH 34,	120'	(CH)	64	30	34	-
▲	BH 34,	128'	(ML)	44	29	15	-
▽	BH 34,	178'	(CH)	87	24	63	-
▼	BH 36,	31'	(CL)	32	22	10	-
⬡	BH 37,	110'	(ML)	32	25	7	86

PLASTICITY CHART

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Figure No
E-18

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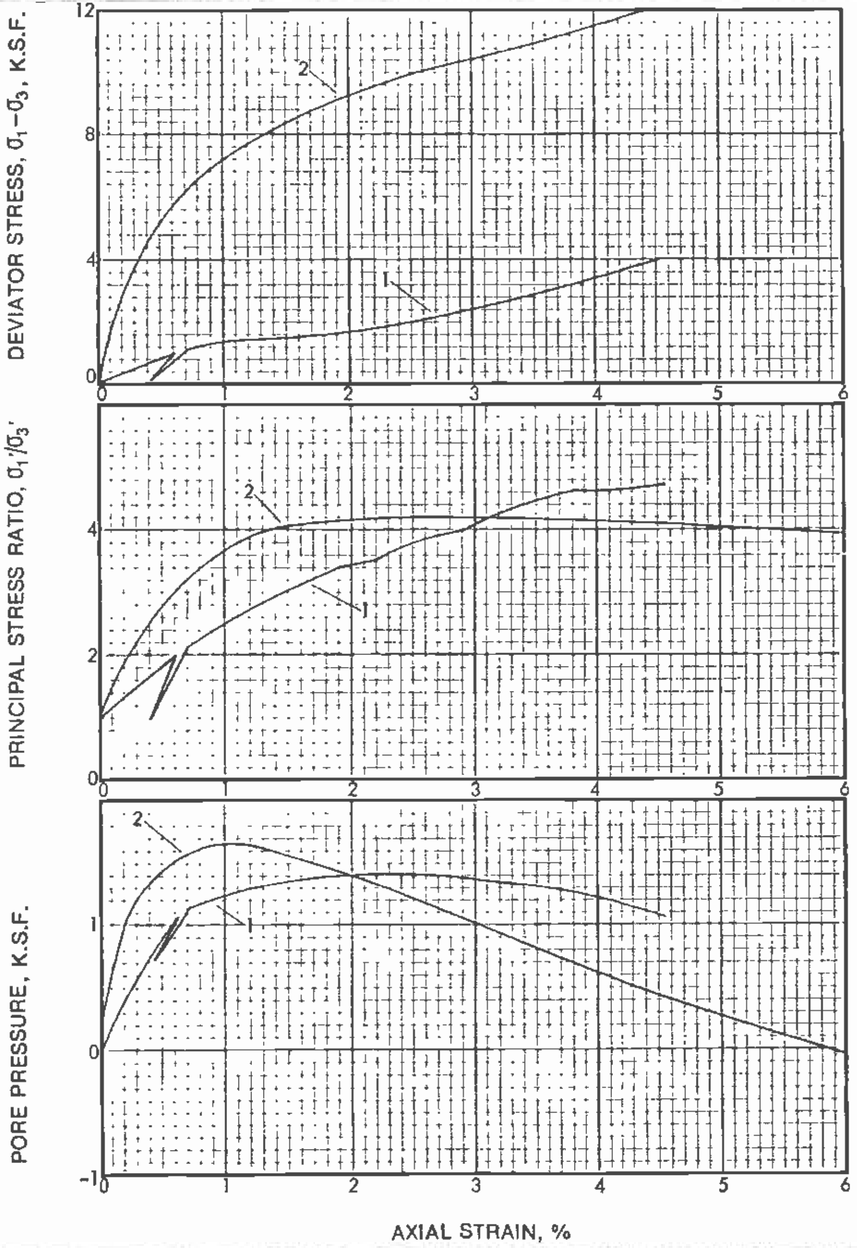

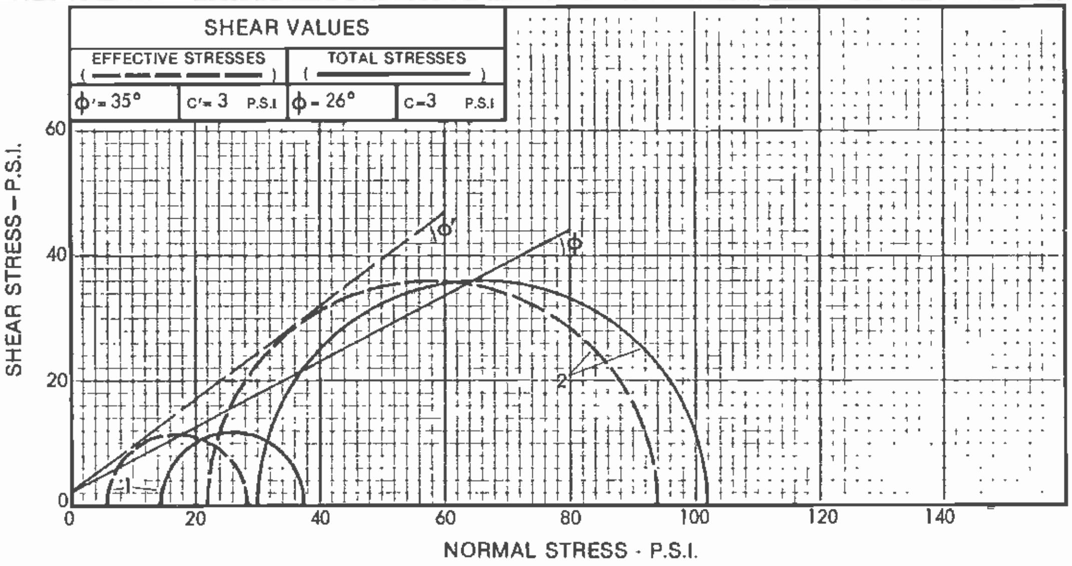

SHEAR VALUES			
EFFECTIVE STRESSES (— — — — —)		TOTAL STRESSES (————)	
φ' = 35°	c' = 3 P.S.I	φ = 26°	c = 3 P.S.I

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-1	34-1	PB-1	12.0-14.5	SC	5.866	2.86	111.1	-	Stage 1
PB-1	34-1	PB-1	12.0-14.5	SC	5.620	2.90	113.0	16.0	Stage 2
				Initial	5.990	2.84	110.2	17.4	Pitcher undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-1	Stage 1	15	22.3	8.9	6.1	28.4	Two Stage ICU with
PB-1	Stage 2	30	71.7	7.6	22.4	94.1	Pore Pressure
							Measurements

TRIAXIAL COMPRESSION TESTS

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Project No
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Figure No
E-19

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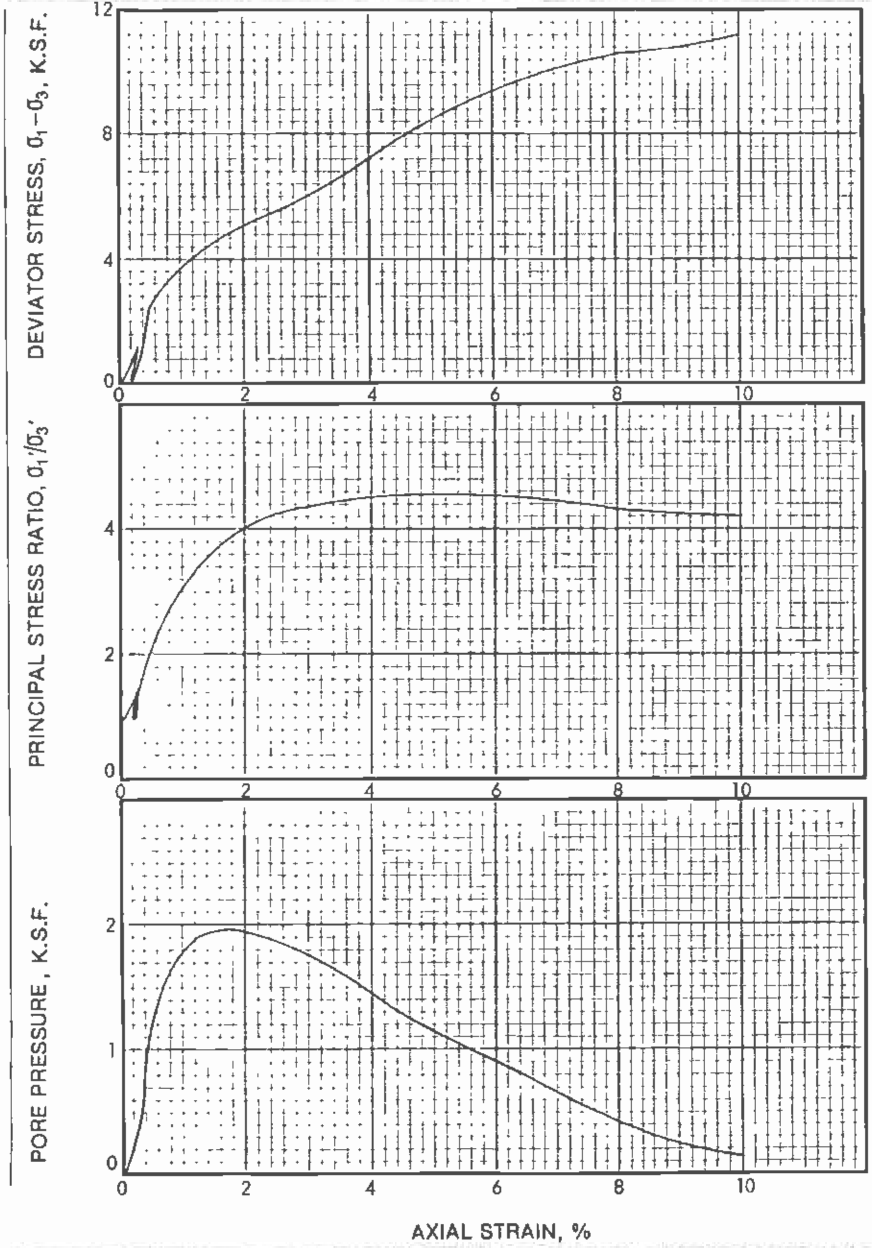

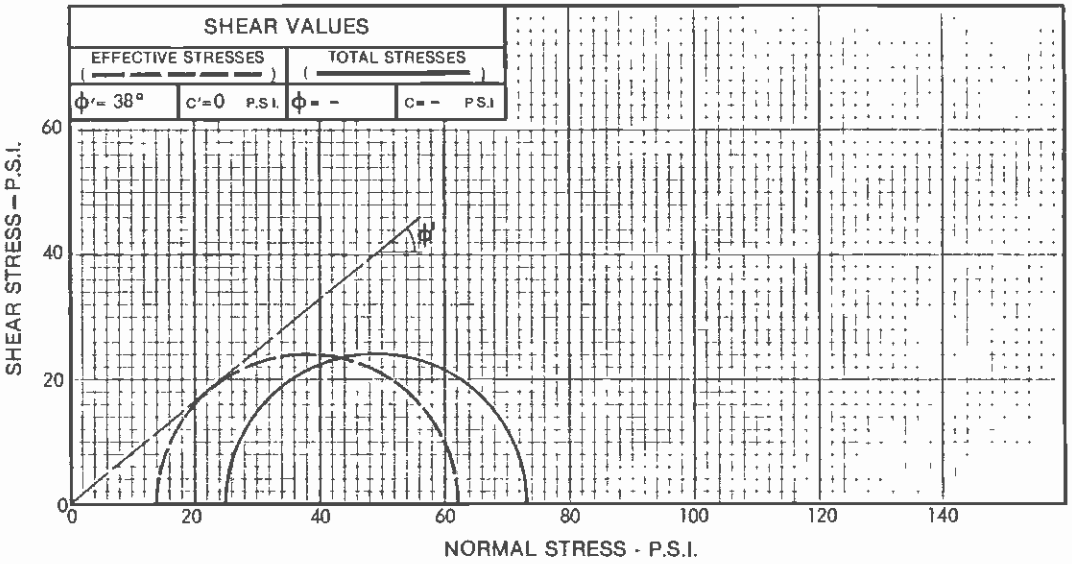

SHEAR VALUES			
EFFECTIVE STRESSES (— — — —)		TOTAL STRESSES (———)	
φ'= 38°	c'=0 P.S.I.	φ= -	c= - P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-5	34-1	PB-5	62.0-63.8	SM/SP	6.142	2.85	113.6	18.4	
				Initial	6.188	2.84	113.3	15.2	Pitcher undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3c} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-5	–	25	47.3	11.0	14.0	61.3	ICU with Pore Pressure measure-ments

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

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Figure No
E-20

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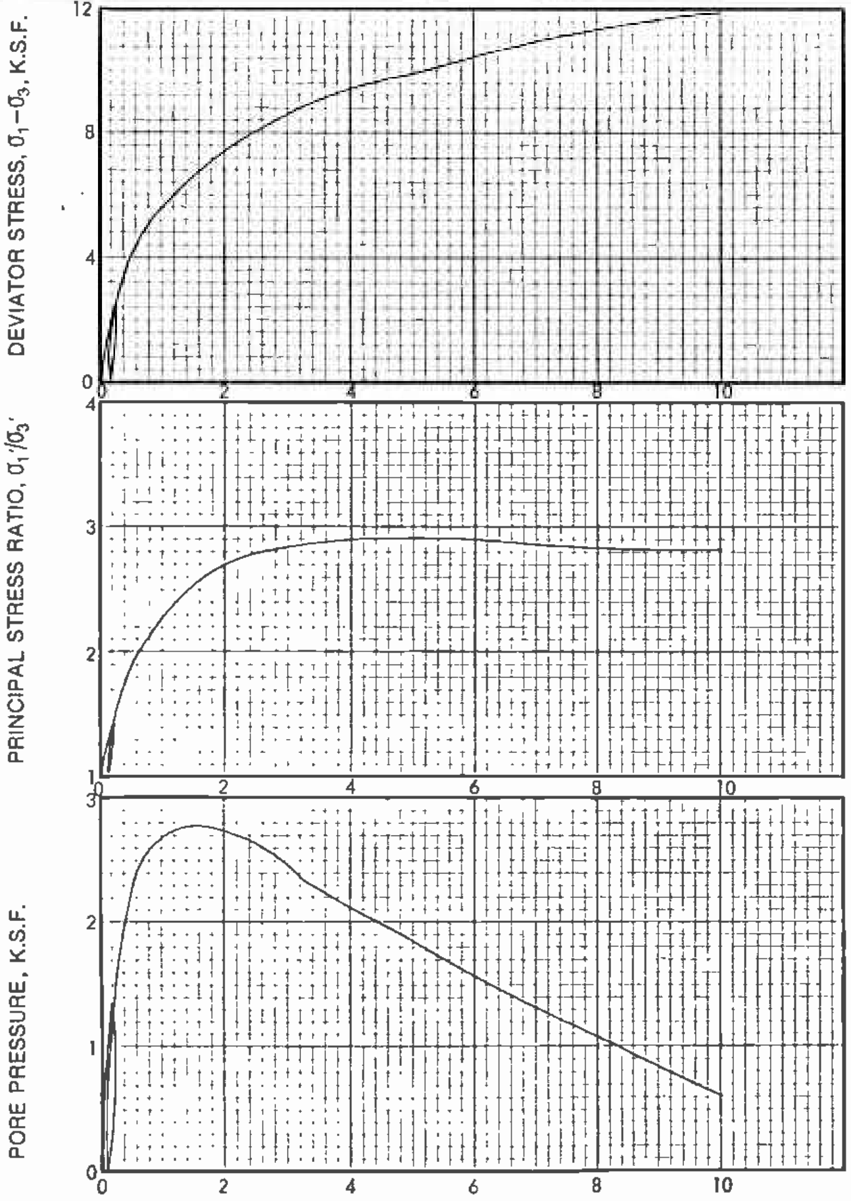

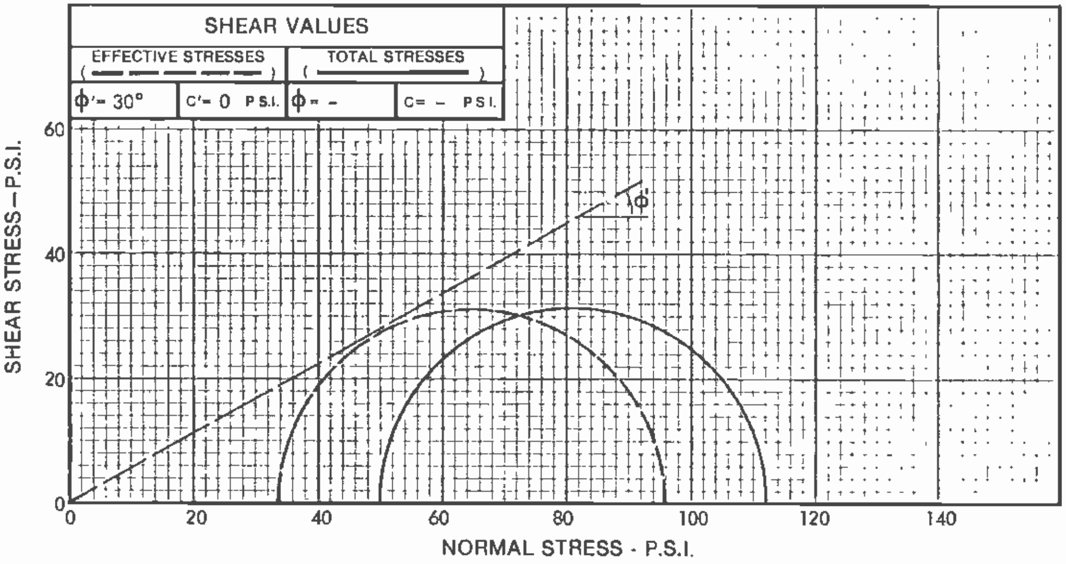

SHEAR VALUES			
EFFECTIVE STRESSES (— — — — —)		TOTAL STRESSES (———)	
ϕ'= 30°	C'= 0 P.S.I.	ϕ = -	C= - P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P C F)	MOISTURE CONTENT (PERCENT)	
PB-7	34-1	PB-7	92.0-94.5	Topanga	5.910	2.89	108.0	18.9	
				INITIAL	6.000	2.88	107.3	20.7	Pitcher Undisturbed

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.)	
PB-7	-	50	62.7	16.5	33.5	96.2	ICU with Pore Pressure measurements

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

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Figure No
E-21

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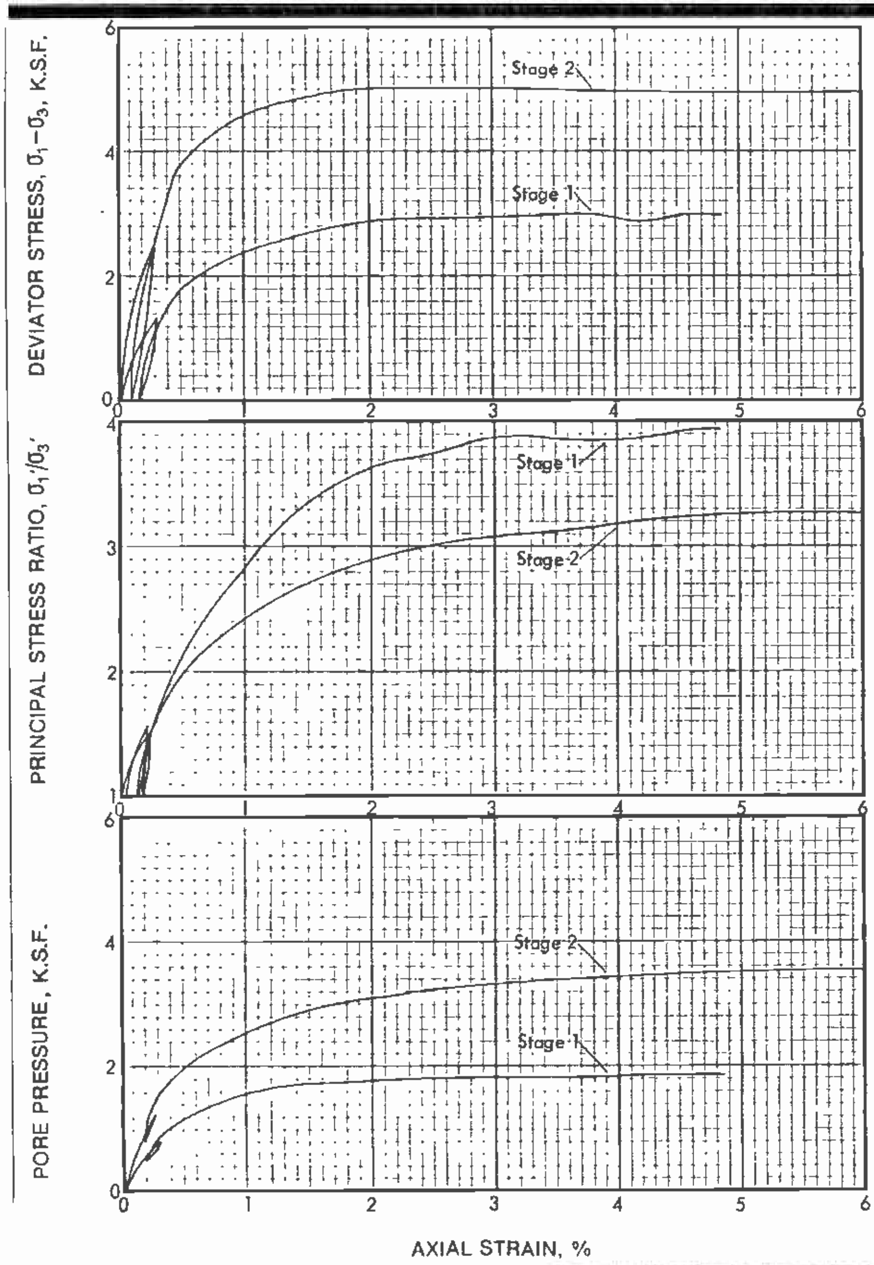

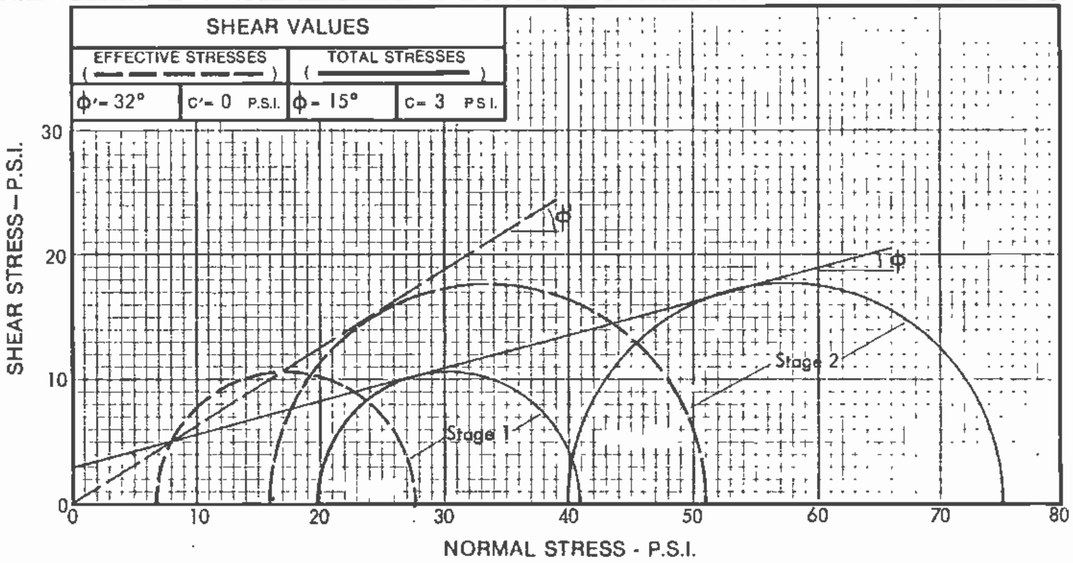

SHEAR VALUES			
EFFECTIVE STRESSES (— — — —)		TOTAL STRESSES (————)	
ϕ'- 32°	c'- 0 P.S.I.	ϕ- 15°	c- 3 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-2	34-2	PB-2	22.0-24.5	CL/SC	5.905	2.85	99.6	-	Stage 1
PB-2	34-2	PB-2	22.0-24.5	CL/SC	5.650	2.89	101.3	22.8	Stage 2
									Pitcher
				Initial	6.000	2.84	98.9	25.0	Undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-2	Stage 1	20	20.5	12.9	7.1	27.6	Two-Stage ICU with Pore Pressure measurements
PB-2	Stage 2	40	34.7	24.1	15.9	50.6	

TRIAXIAL COMPRESSION TESTS

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Figure No
E-22

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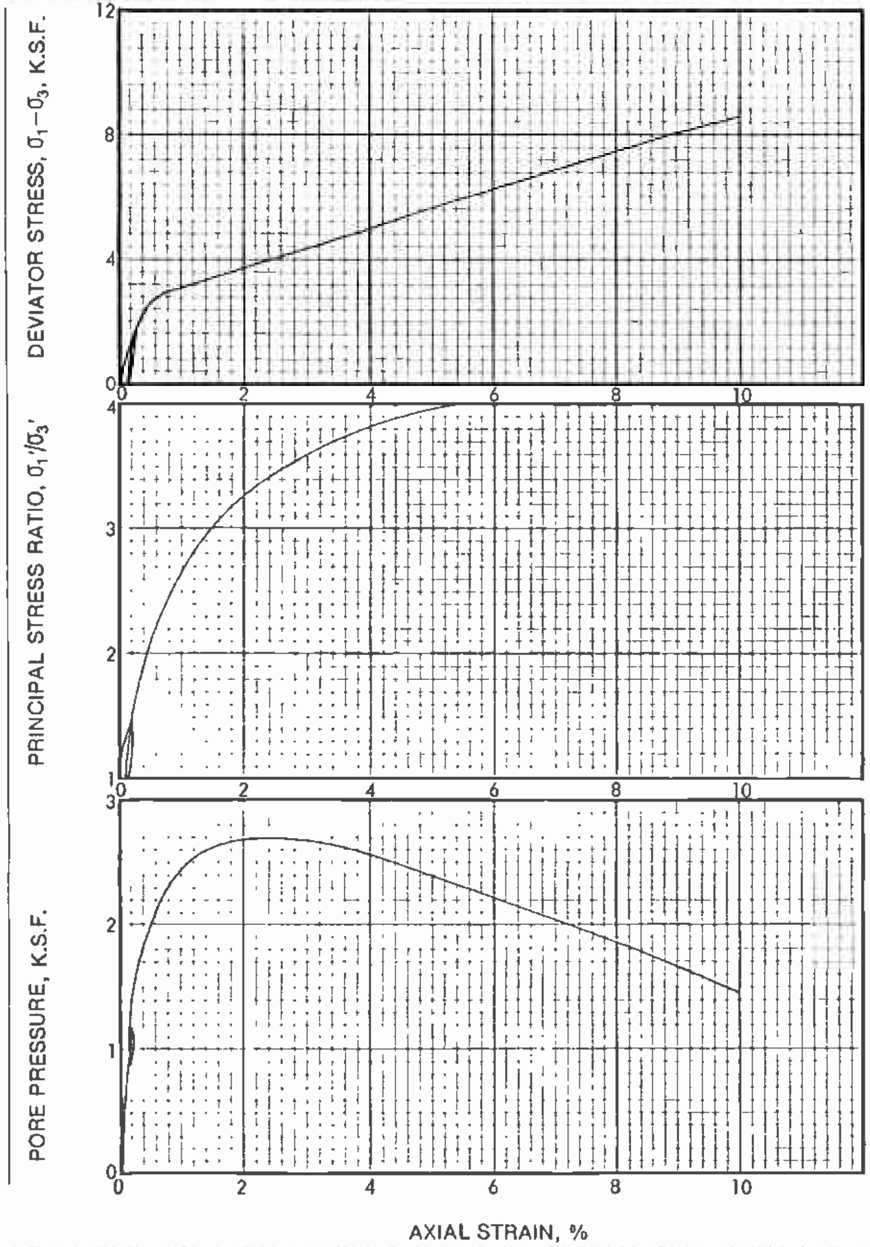

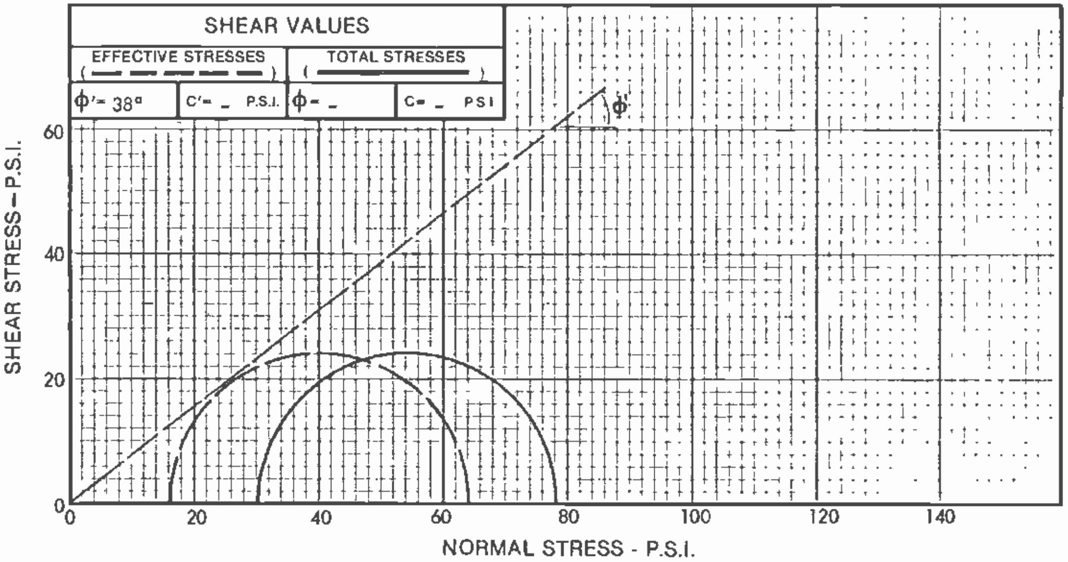

SHEAR VALUES			
EFFECTIVE STRESSES (— — — —)		TOTAL STRESSES (———)	
ϕ'= 38°	C'= _ P.S.I.	ϕ= _	C= _ PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			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	34-2	C-5	52.0-53.0	SM/SC	5.110	2.41	93.1	22.8	
				Initial	5.188	2.40	92.5	35.3	Pitcher Undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-5	-	30	48.1	14.1	15.9	64.0	ICU with Pore Pressure measurements

TRIAXIAL COMPRESSION TESTS

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Figure No
E-23

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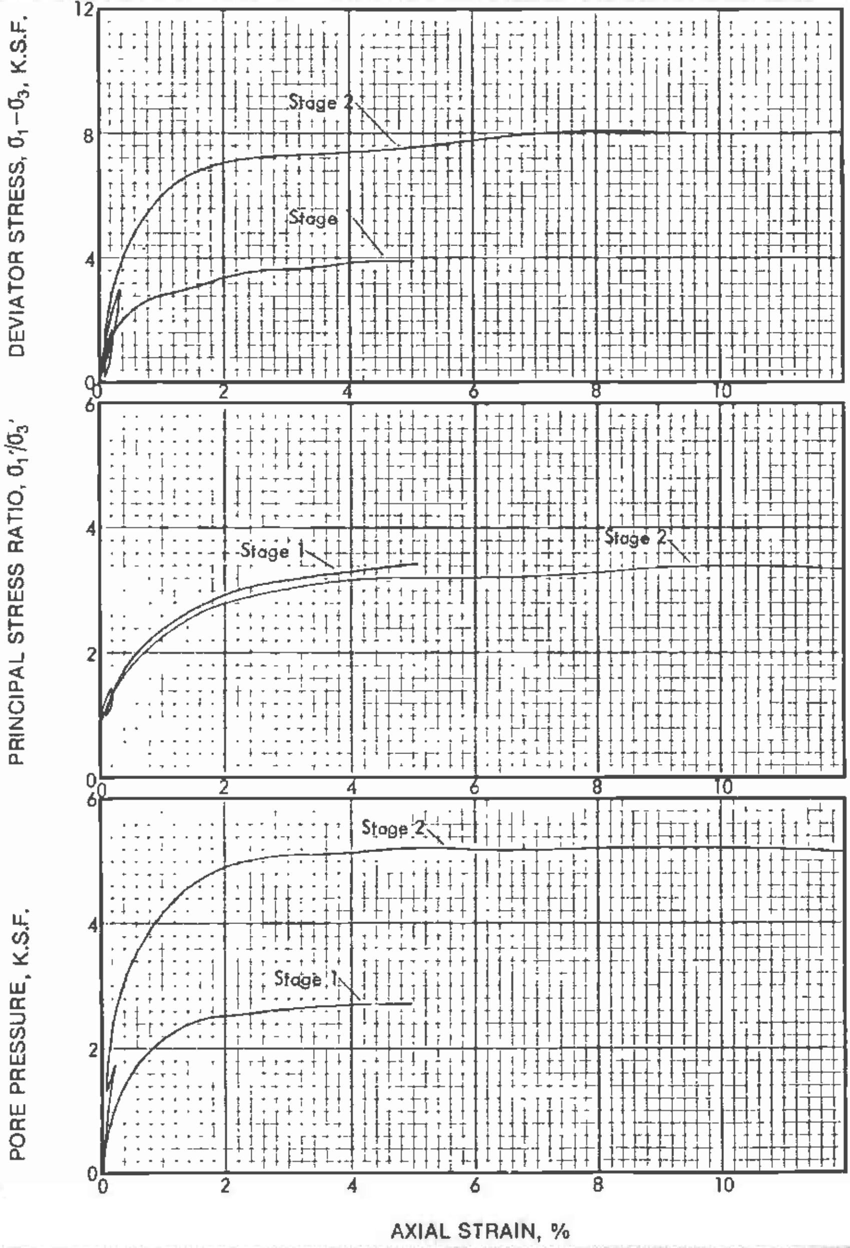

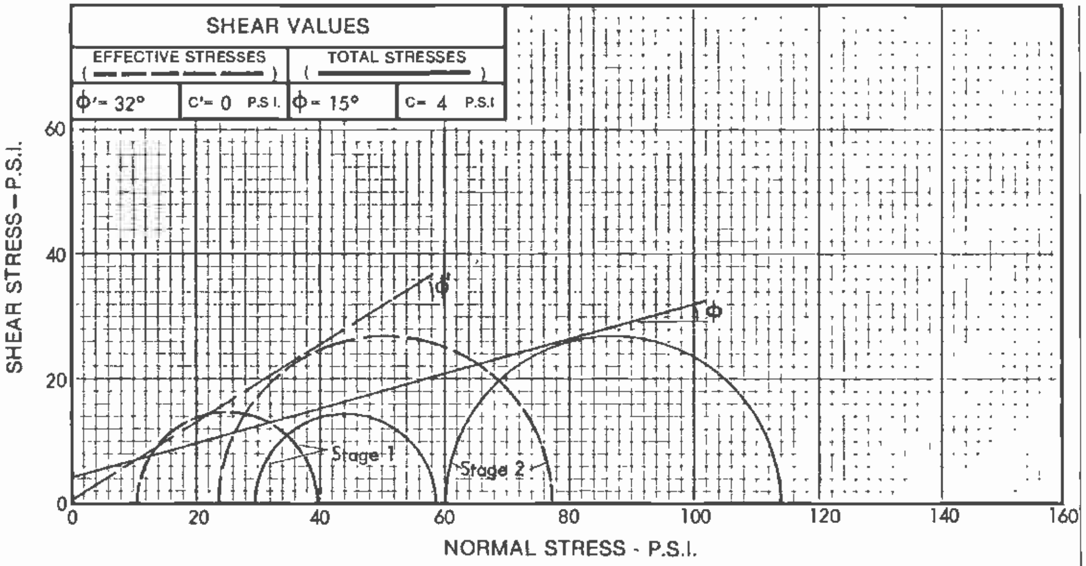

SHEAR VALUES			
EFFECTIVE STRESSES (— — — —)		TOTAL STRESSES (———)	
φ'= 32°	c'= 0 P.S.I.	φ= 15°	c= 4 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-3	34-3	PB-3	32.0-34.5	CL	5.895	2.86	96.1	-	Stage 1
PB-3	34-3	PB-3	32.0-34.5	CL	5.641	2.90	97.8	23.7	Stage 2
				Initial	6.000	2.85	95.4	25.8	

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL PRESSURE σ_{3c} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-3	Stage 1	30	27.5	18.5	11.5	39.0	Two-Stage ICU with Pore Pressure measurements
PB-3	Stage 2	60	52.8	36.1	23.9	76.7	

TRIAXIAL COMPRESSION TESTS

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Figure No
E-24

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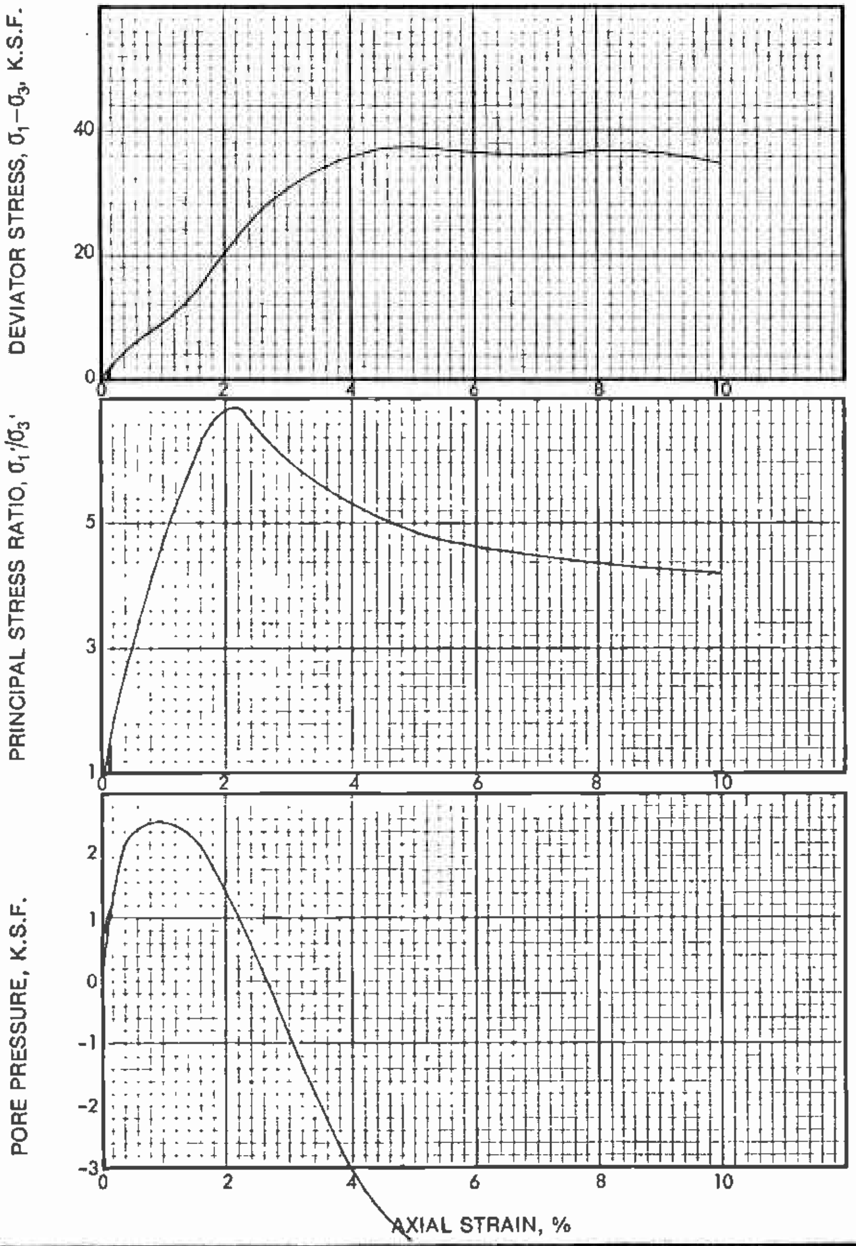
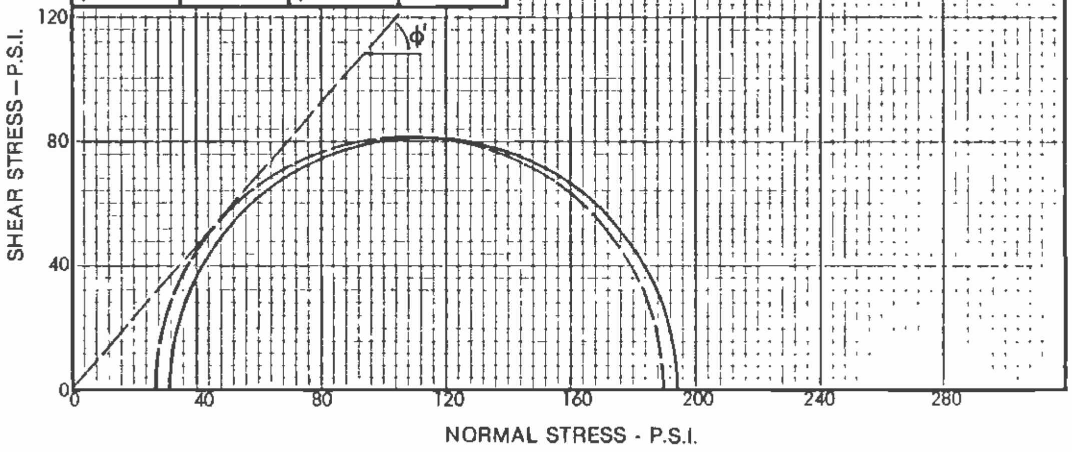


SHEAR VALUES			
EFFECTIVE STRESSES (— — —)		TOTAL STRESSES (———)	
ϕ'= 49°	c'= 0 PSI	ϕ= -	C= - PSI



SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P C F)	MOISTURE CONTENT (PERCENT)	
PB-6	34-3	PB-6	74.1-76.6	Topanga	6.357	2.89	112.0	16.6	
				Initial	6.438	2.88	111.5	17.6	Pitcher undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I)	PORE PRESSURE CHANGE ΔU (P S.I)	MINOR EFFECTIVE STRESS σ_3' (P S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P S I.)	
PB-6		35	162.6	7.3	27.7	190.4	ICU with Pore Pressure measurements

TRIAXIAL COMPRESSION TESTS

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Project No
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Figure No
E-25

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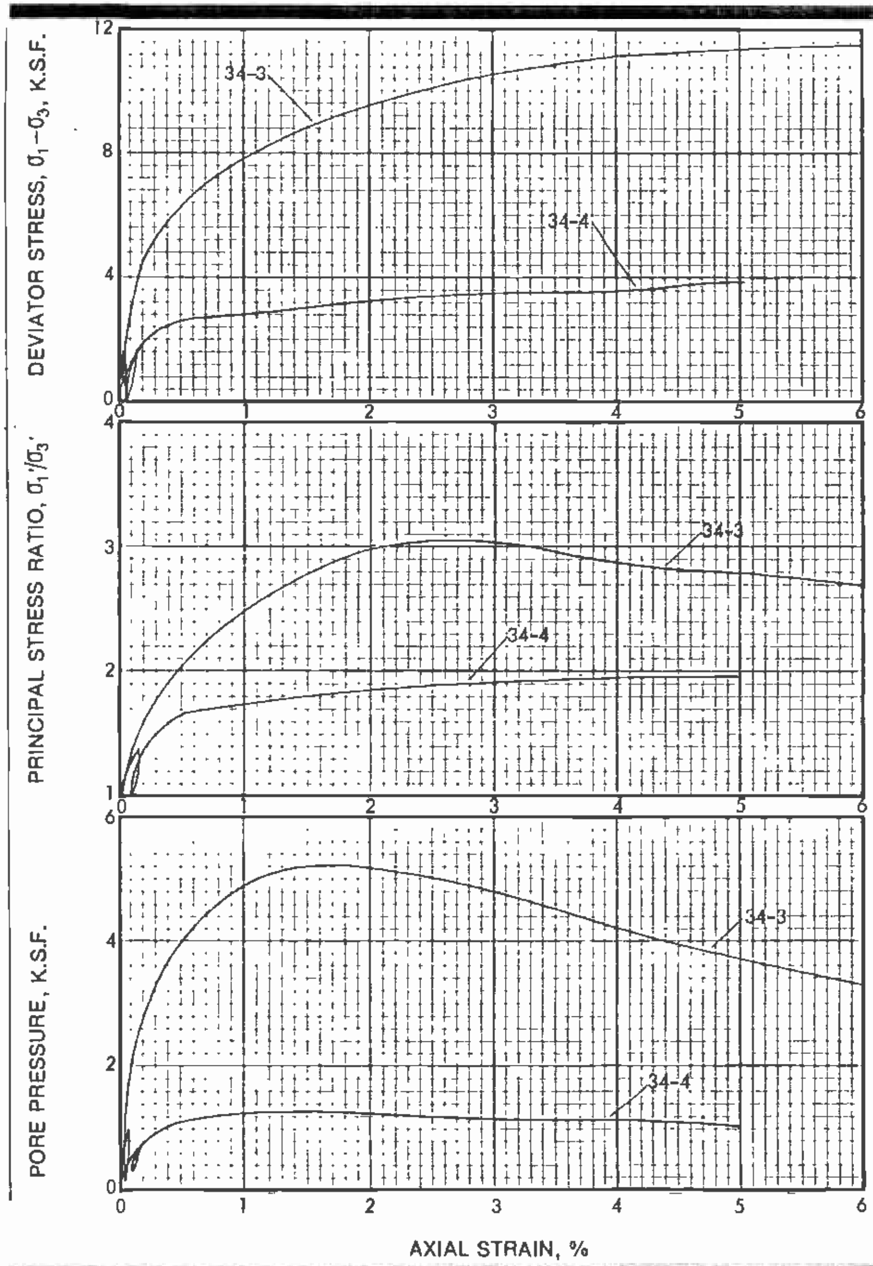

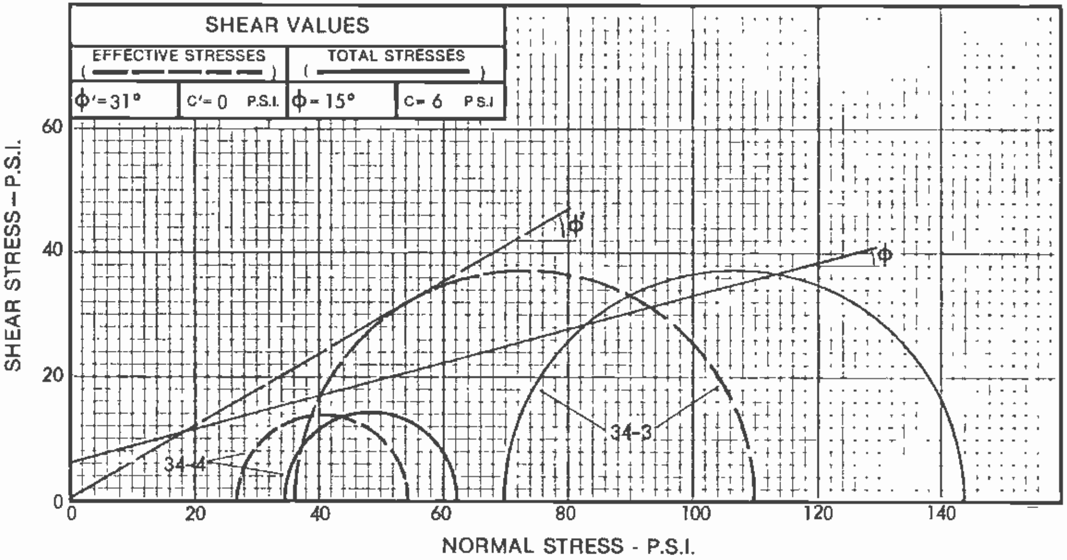

SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'=31°	C'=0 P.S.I.	ϕ= 15°	C= 6 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-9	34-4	PB-9	111.5-114	Topanga	6.232	2.91	109.1	18.8	Pitchers undisturbed
PB-9	34-3	PB-9	103-105.5	Topanga	5.882	2.88	106.4	21.5	
	34-4			Initial	6.281	2.90	108.8	18.3	
	34-3			Initial	6.000	2.87	105.5	19.8	

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.)	
PB-9	34-4	35	26.4	7.3	27.7	54.1	Two single stage ICU's with pore pressure measurements
PB-9	34-3	70	73.3	33.6	36.4	109.7	

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Figure No
E-26

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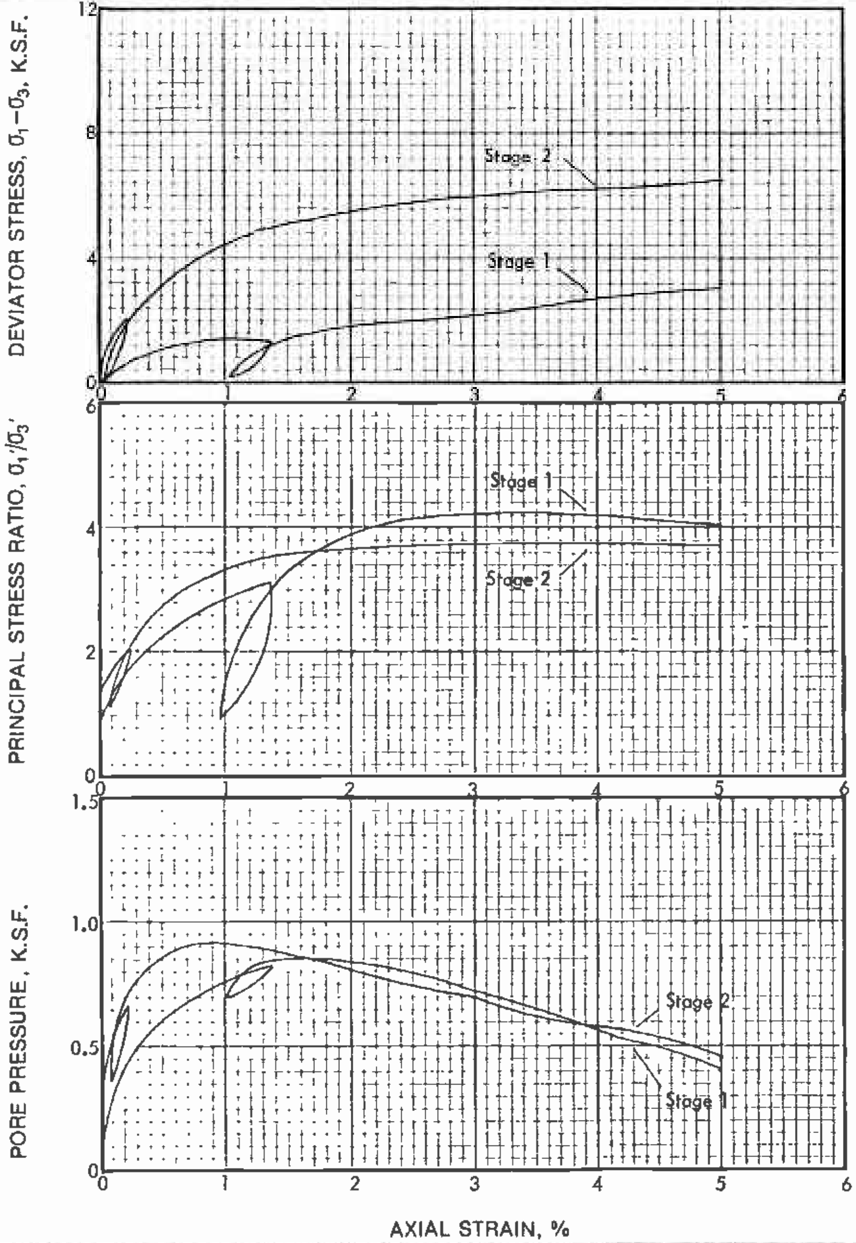

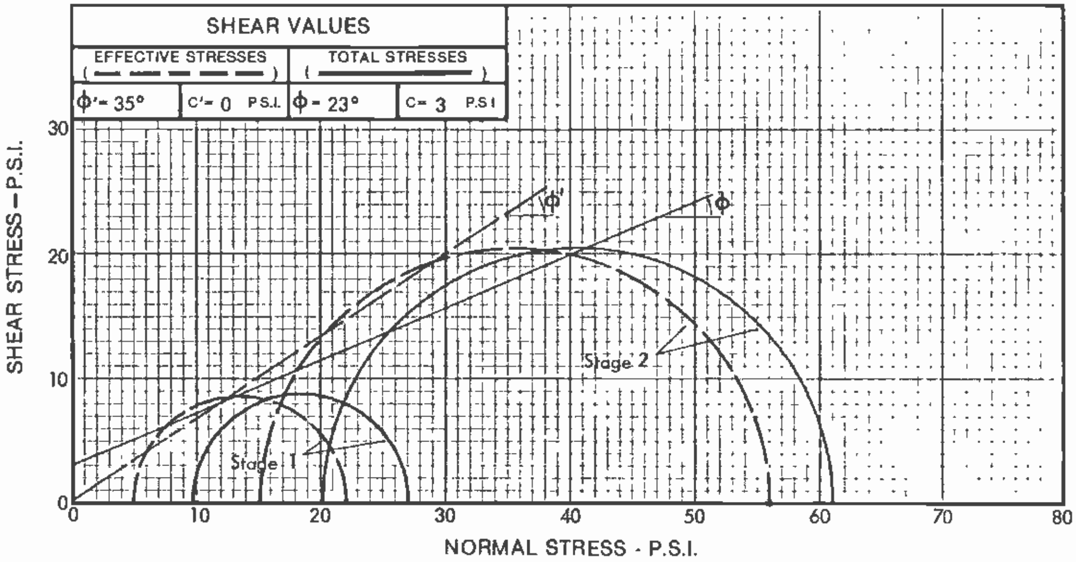

SHEAR VALUES			
EFFECTIVE STRESSES (— — —)		TOTAL STRESSES (———)	
φ'= 35°	c'= 0 P.S.I.	φ= 23°	c= 3 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P C F)	MOISTURE CONTENT (PERCENT)	
PB-1	34-4	PB-1	12.0-14.5	SC	5.937	2.86	104.2	-	Stage 1
PB-1	34-4	PB-1	12.0-14.5	SC	5.753	2.89	105.5	18.2	Stage 2
									Pitcher
				Initial	6.000	2.85	103.8	23.1	Undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL PRESSURE σ_{3C} (P S I)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P S I.)	PORE PRESSURE CHANGE ΔU (P S I)	MINOR EFFECTIVE STRESS σ_3' (P S I)	MAJOR EFFECTIVE STRESS σ_1' (P.S I.)	
PB-1	Stage 1	10	16.6	4.9	5.1	21.7	Two-Stage ICU with Pore Pressure measurements
PB-1	Stage 2	20	41.0	4.9	15.1	56.1	

TRIAXIAL COMPRESSION TESTS

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Figure No
E-27

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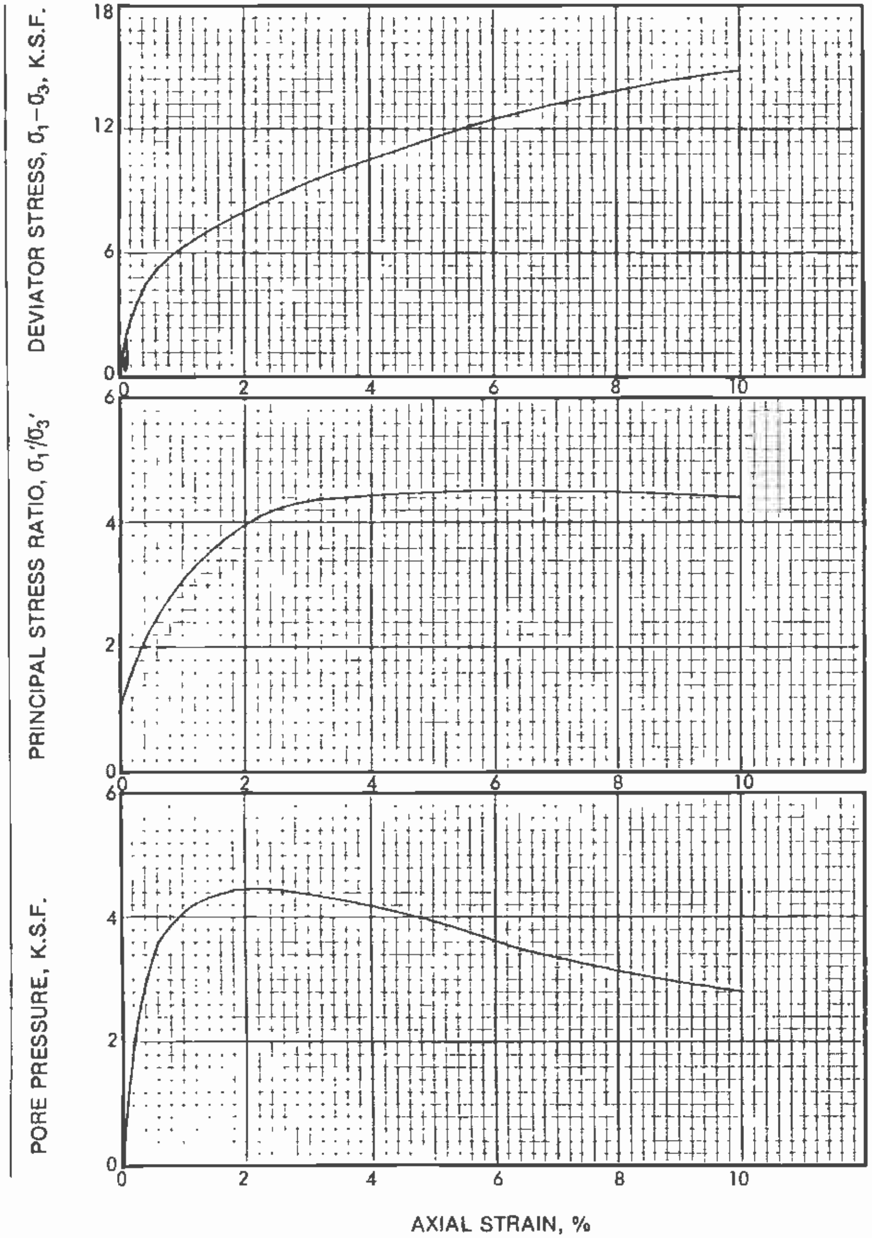

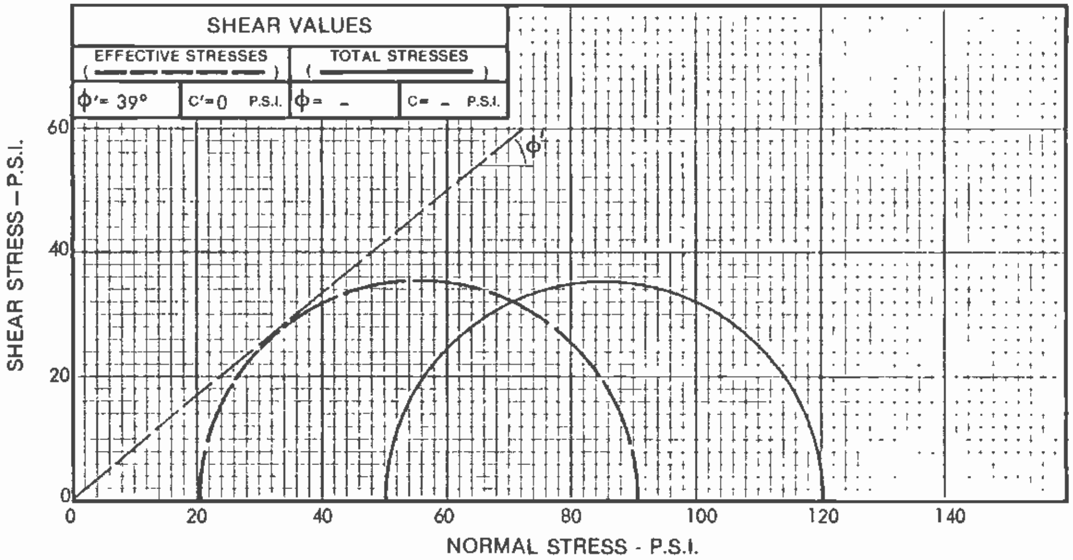

SHEAR VALUES			
EFFECTIVE STRESSES (— — —)		TOTAL STRESSES (———)	
φ'= 39°	C'=0 P.S.I.	φ= –	C= – P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-5	34-4	PB-5	62.0-64.5	SM/SP	6.170	2.86	101.7	16.2	
				Initial	6.250	2.85	101.2	24.5	Pitcher undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-5	–	50	70.3	29.5	20.5	90.8	ICU with pore pressure measurements

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

Project No
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Figure No
E-28

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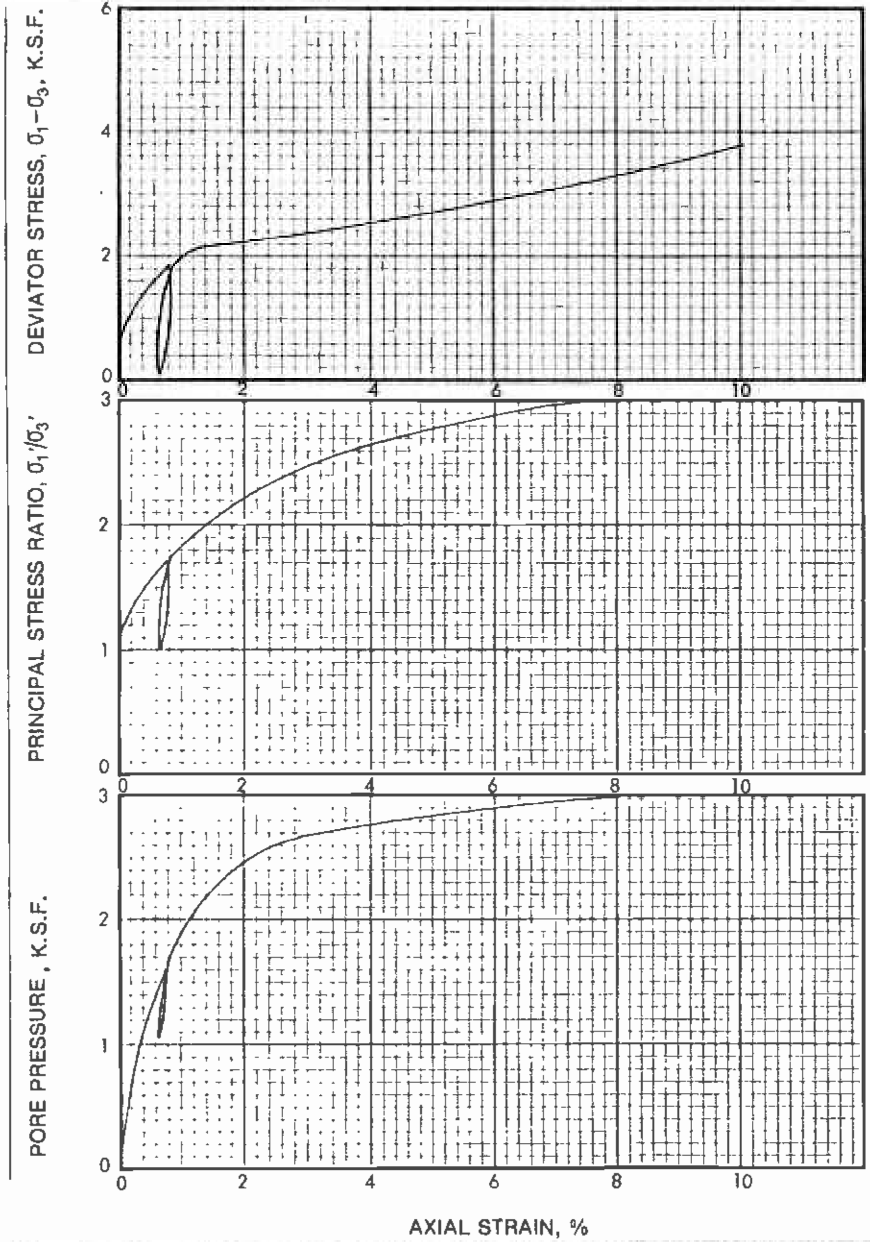

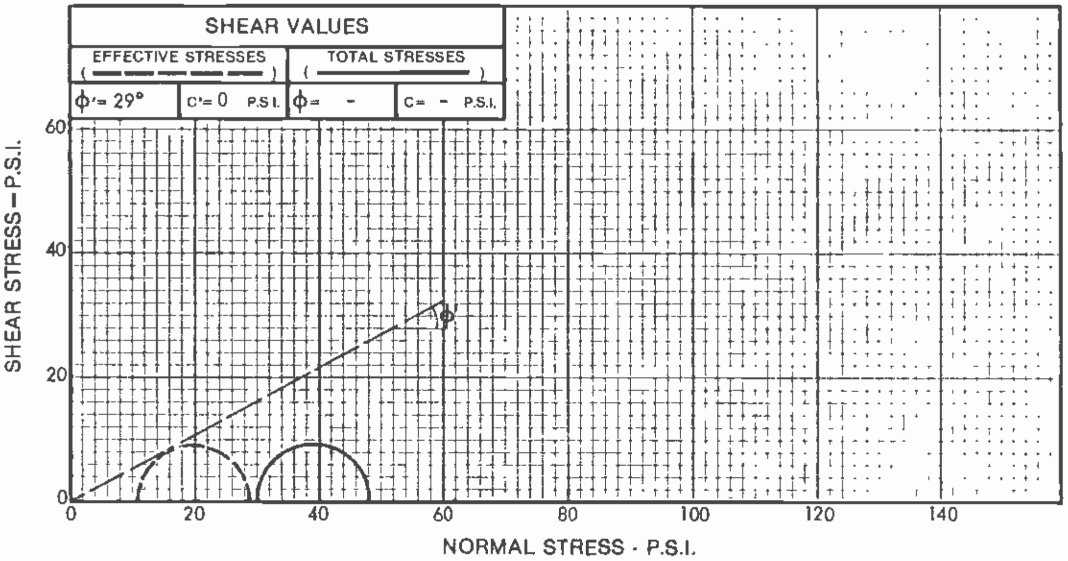

SHEAR VALUES			
EFFECTIVE STRESSES (— — — —)		TOTAL STRESSES (———)	
ϕ'= 29°	C'= 0 P.S.I.	ϕ= -	C= - P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-5	34-5	PB-5	70.0-72.5	Topanga	5.504	2.90	92.6	22.1	
				Initial	5.938	2.83	89.9	24.0	Pitcher undisturbed

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3c} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE Δ U (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-5	-	30	17.0	18.8	11.2	28.2	ICU with pore pressure measurements

TRIAXIAL COMPRESSION TESTS

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Project No
B3-1140

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Figure No
E-29

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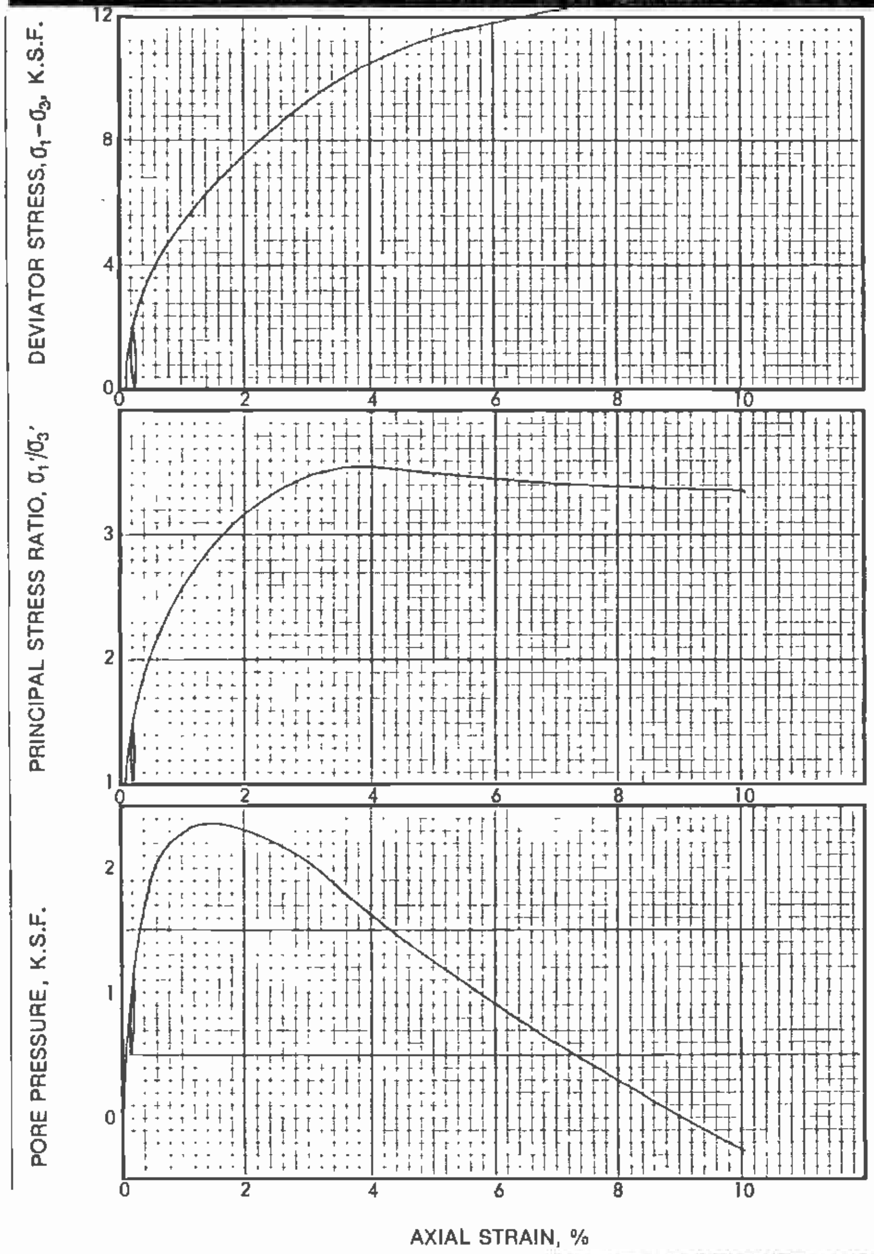

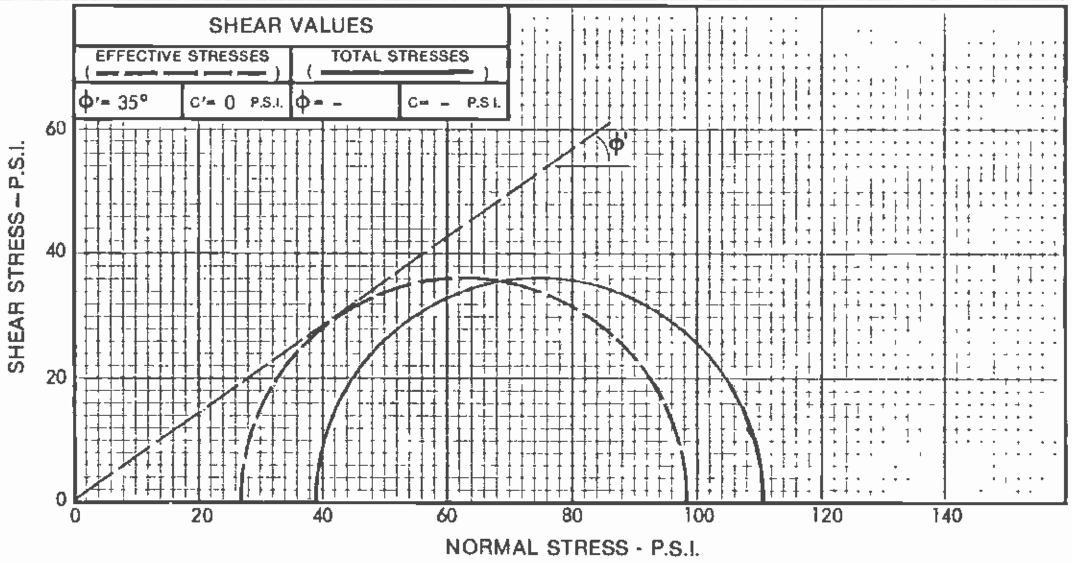

SHEAR VALUES

EFFECTIVE STRESSES (— — —)		TOTAL STRESSES (———)	
φ'= 35°	C'= 0 P.S.I.	φ= -	C= - P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-7	34-5	PB-7	84-86.5	Topanga	6.031	2.86	111.9	17.9	
				Initial	6.125	2.85	111.2	17.7	Pitcher undisturbed

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.)	
PB-7		40	71.7	12.1	27.9	99.6	ICU with Pore Pressure measurements

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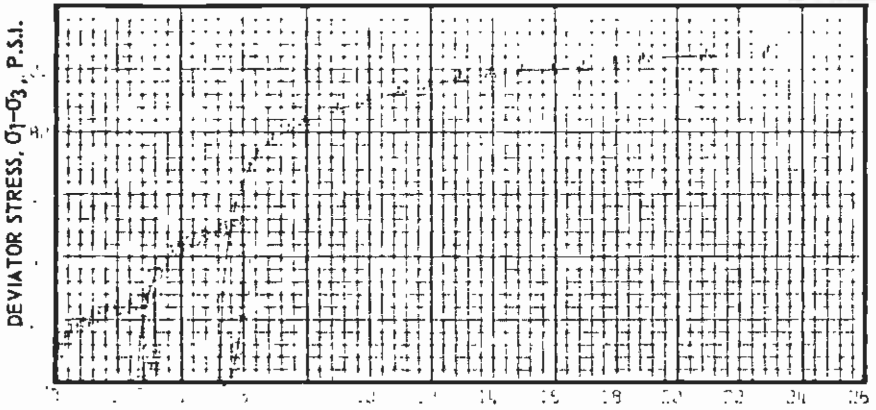
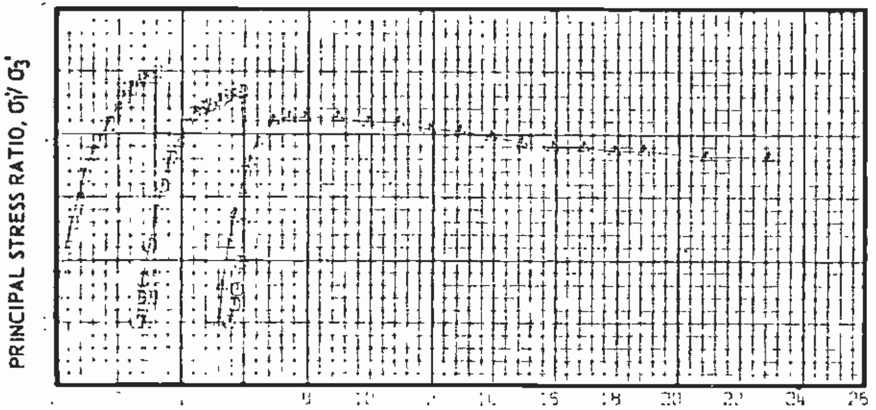
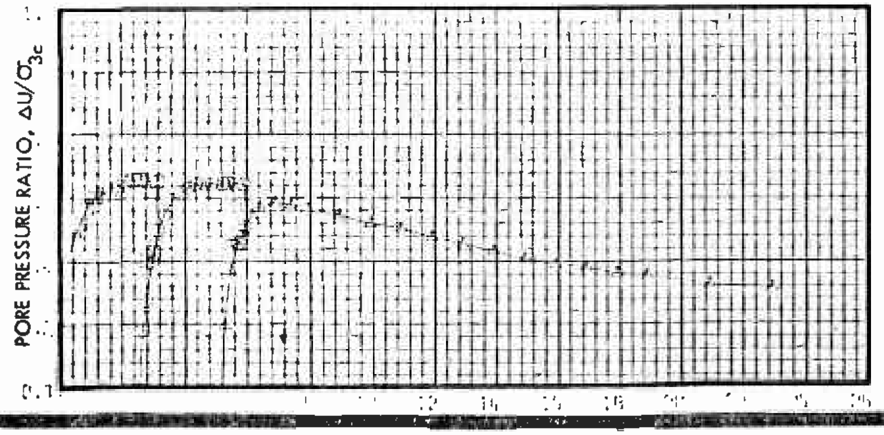
Figure No
E-30







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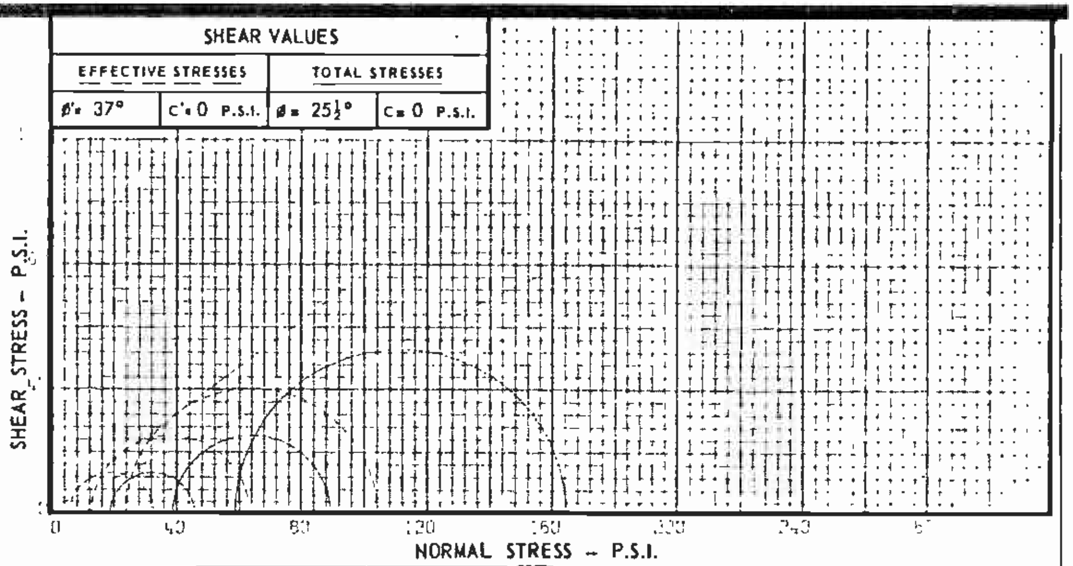

SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 37°	c'= 0 P.S.I.	ϕ = 25½°	c = 0 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSIFICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C2	34	31	ML	5.74	2.42	[illegible]	[illegible]	UNDISTURBED
C2	34	31	ML	5.74	2.42	107.70	20.0	UNDISTURBED
C2	34	31	ML	5.74	2.42	107.70	20.0	UNDISTURBED

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
□	C2	20.0	27.2	6.8	34.[illegible]	60.0	CU PROGRESSIVE
○	C2	40.0	50.8	13.9	64.7	60.0	CU PROGRESSIVE
+	C2	60.0	106.0	24.8	105.6	60.0	CU PROGRESSIVE

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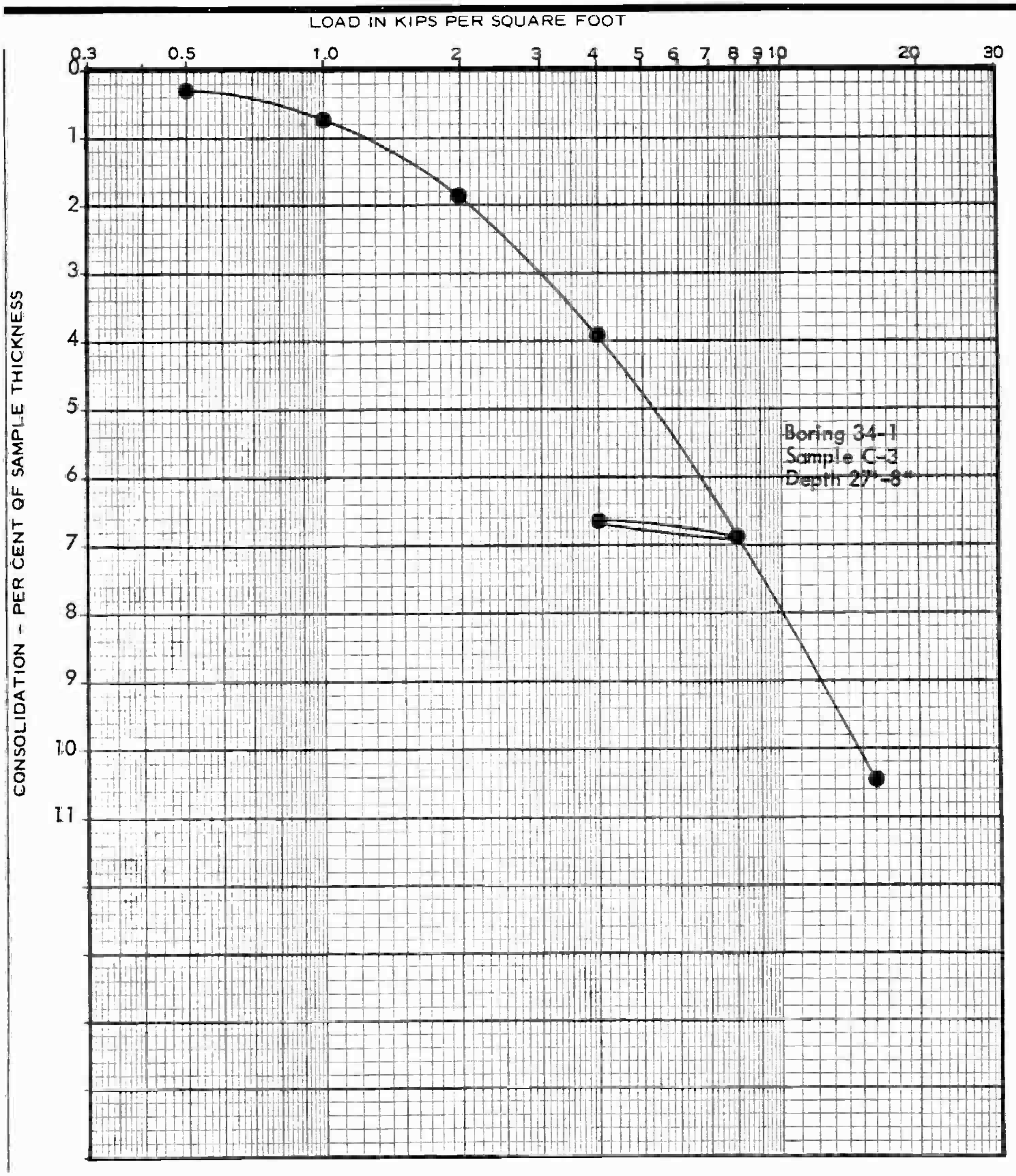
Figure No.
E-31



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Drawing No
E-32

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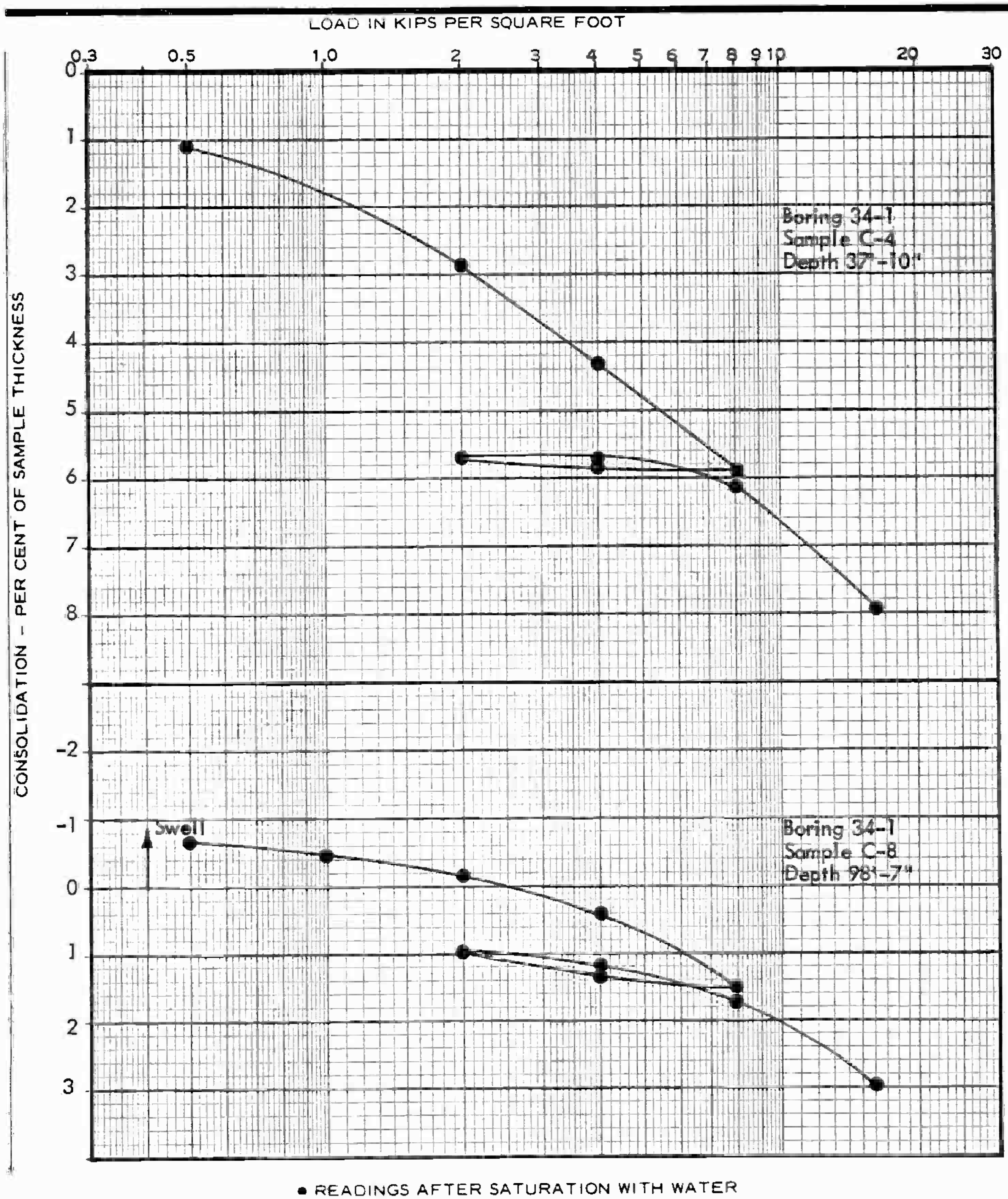

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Drawing No.
E-33

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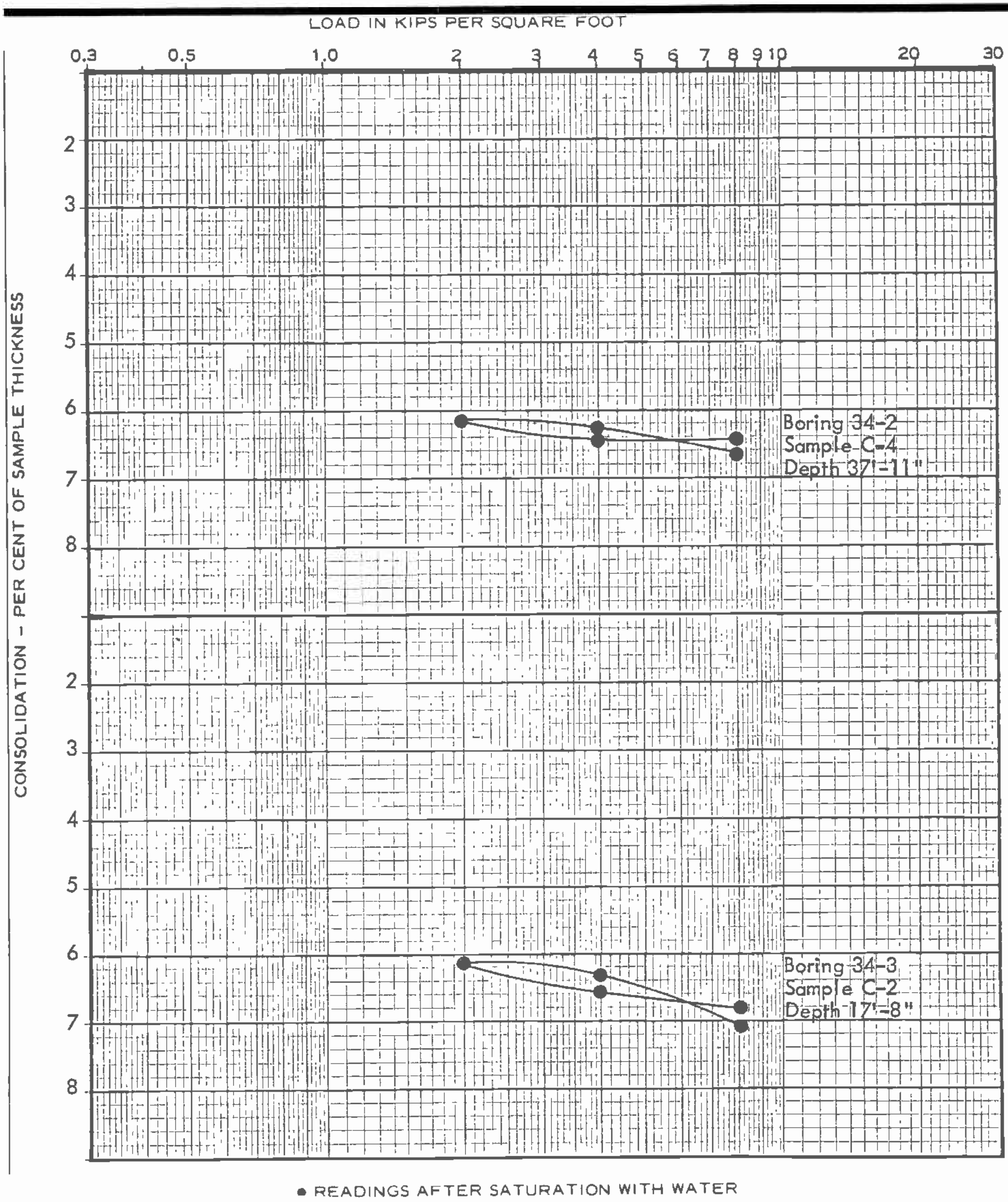

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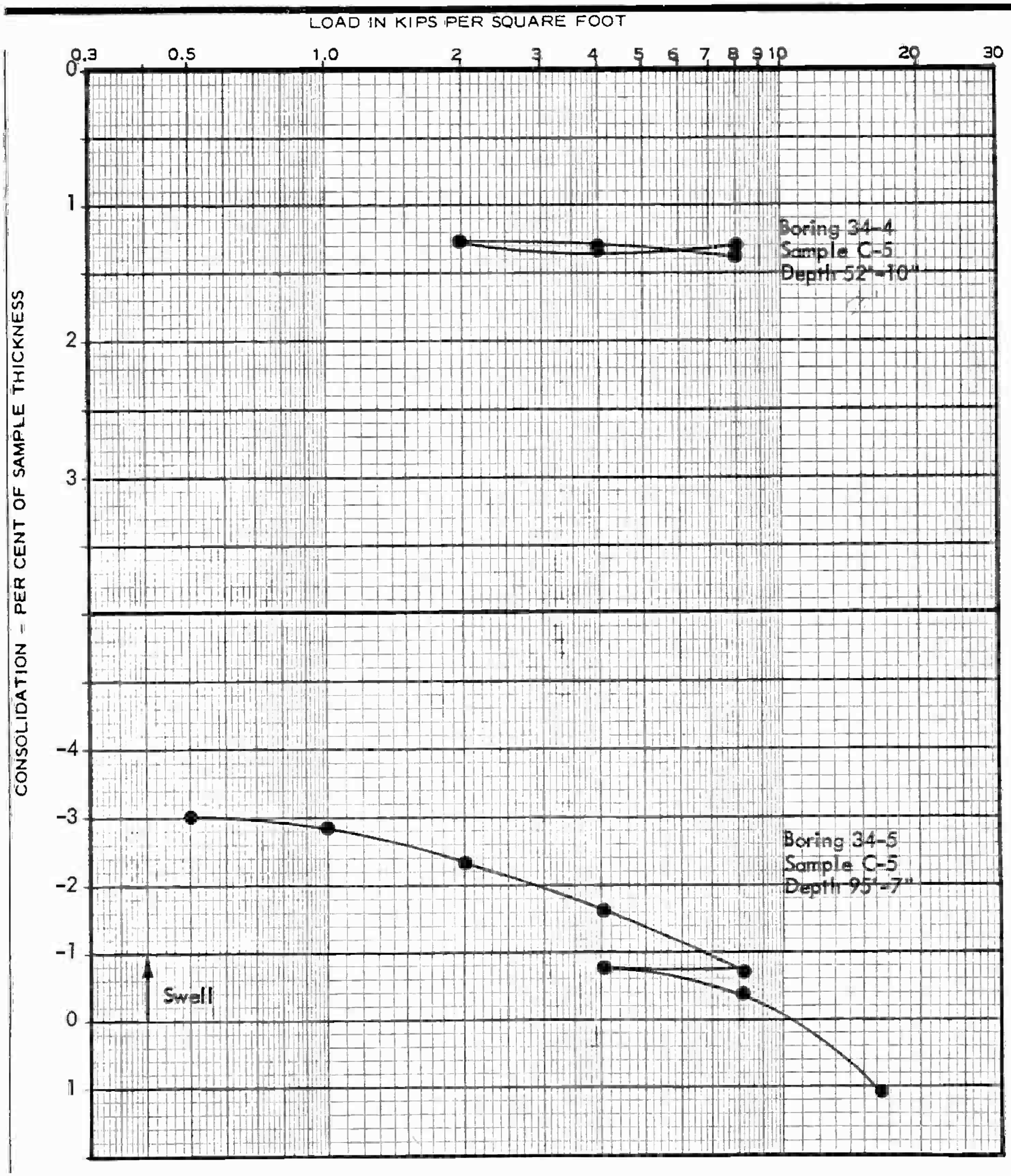
Drawing No
E-34



● READINGS AFTER SATURATION WITH WATER

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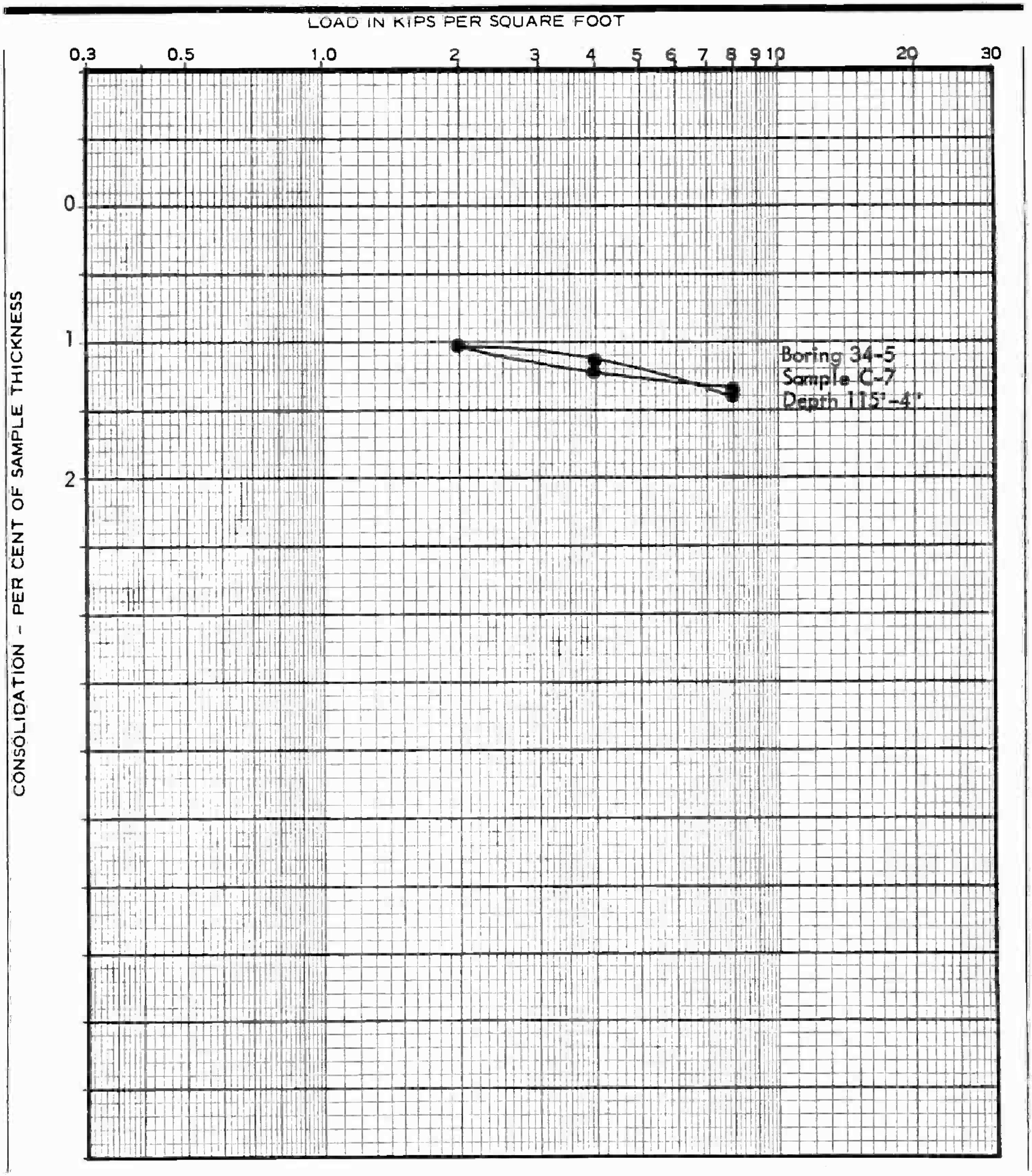
Drawing No.
E-35



● READINGS AFTER SATURATION WITH WATER

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Drawing No
E-36

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Appendix F

Technical Considerations

APPENDIX F: TECHNICAL CONSIDERATIONS

F.1 SHORING PRACTICES IN THE LOS ANGELES AREA

F.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.

F.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 Flower Street 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 0.1 g. 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.

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

This project involved a shored excavation from 70 to 110 feet deep in the Old Alluvium deposit. 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 groundwater table was below the level of excavation, although minor seeps from perched groundwater 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 exiting 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.

F.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 groundwater 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.

F.1.5 Design Lateral Load Practices

Table F-1 summarizes the design lateral loads used for eight 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 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 earthpressure. Assuming a friction angle of 37 degrees, the equivalent design pressure should equal about 22H-psf. For hard clays, the recommended value ranges from 0.15-0.30 (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.

F.2 SEISMICALLY INDUCED EARTHPRESSURES

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.

Table F-1

SHORING LOADS IN LOS ANGELES AREA

Project Location	Excavation Depth (ft)	Soil Conditions	Actual Design Pressure (P)	Equivalent Design Pressure (P')
Broadway Plaza Near 7th/Flower Station	15-30	Fill over Alluvium Sands	19.0H	15.2H
500 S. Hill	25	Fill over Sands and Gravel	22.0H	17.6H
Tishman Building Near CEG-14	25	Alluvium-Clays, Sand, Silt	19.0H	15.2H
Equitable Life Near CEG-14	55	Alluvium Sand/ Siltstone	20.0H	17.5H
Arco Near CEG-9	70-90	Alluvium over Claystone	16.0H	12.0H
Century City Near CEG-20	70-110	Alluvium-Clays and Sands	18.0H	14.4H
St. Vincent's Near 3rd & Lk.	70	Thin Alluvium over Puente	15.0H	12.0H
Oxford Plaza Near 7th/Flower	40	Fill & Alluvium over Siltstone	21.0H	16.8H

Notes: All shoring systems were soldier piles.

All pressure diagrams were trapezoidal.

Equivalent pressure equals a uniform rectangular distribution.

Nevertheless, the pseudo-static method is still used today since it provides a simple means for assessing the additional hazard to stability imposed by earthquake loadings.

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

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

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

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

where:

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

$\theta = \tan^{-1}\ (k_h)/(1-k_v)$

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

γ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes

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

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

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

where:

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

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 as the peak values of horizontal and vertical accelerations do not occur at the same instant of time during an earthquake and are usually at different frequencies. The vertical earthquake component usually contains much higher frequencies than the horizontal component.

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 conservative since the peak acceleration only acts 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 of the face 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 increase 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).

Appendix G

Earthwork Recommendations

APPENDIX G: 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. Soil 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 1.5:1 (horizontal to vertical).

- Structural Fill and Backfill:

  Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite 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 could be increased from the normal 90% to 95% or 100% of the maximum dry density to reduce fill settlement.

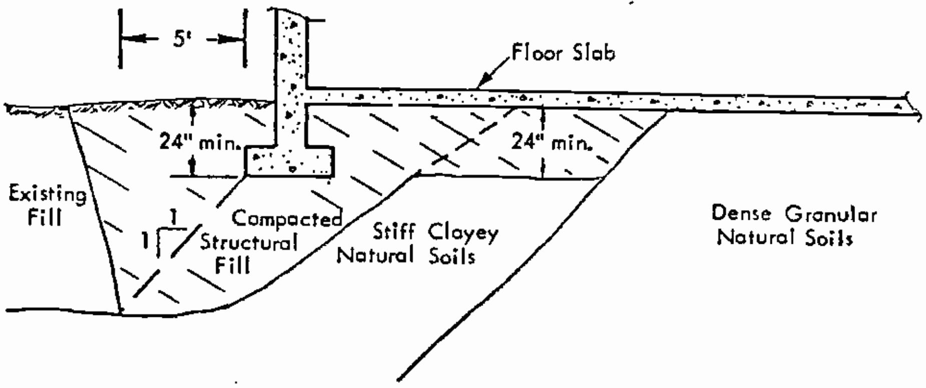
  Where space limitations do not allow for conventional backfill compaction operations, special backfill materials and procedures may be required. Sand-cement slurry, pea gravel or other selected backfill can be used in limited space areas. Sand-cement slurry should contain at least 1-1/2 sacks cement per cubic year. 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., soils containing at least 40 percent passing the No. 200 sieve.

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

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

Geotechnical Reports References

APPENDIX H GEOTECHNICAL REPORT REFERENCES

Report No.	Report Date	Location	Consultant
44	07/27/46	Universal Pictures, Inc.--Sound Stage C	L. T. Evans
45	09/29/61	Revue Studios--Lankershim Boulevard	L. T. Evans
46	10/27/65	Tower No. 2, Universal City Studios--Lankershim Boulevard	L. T. Evans
47	08/06/74	Universal City Studios-- 80 Lankershim Boulevard	L. T. Evans
48	06/03/76	Universal City Studios-- 70 Lankershim Boulevard Office Building and Parking Structure	L. T. Evans