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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A445

BY

CONVERSE CONSULTANTS, INC.
EARTH SCIENCES ASSOCIATES
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May 31, 1984

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

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A445 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 A445.

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 Mike Traubenick.

Respectfully submitted,

for James A. [illegible]

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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 our geotechnical investigations and engineering analyses for the A445 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A445 Design Unit consists of the North Hollywood Station and about 450 feet of tail track structure. The Station and tail track will be constructed by cut-and-cover methods and require excavations as deep as 58 feet below the existing ground surface. Construction will occur in alluvial soils. Available data for Design Unit A445 suggest that groundwater levels are well below the bottom of the proposed excavations. The report defines the subsurface conditions and provides recommendations for design and construction purposes. Although this report may be used for construction purposes, it is not intended to provide all of the information that may be required.

1.1 STATION AND TAIL TRACK STRUCTURES

The subsurface conditions at the Station site and along the alignment of the tail track structure consist of coarse-grained Alluvium which are primarily sands and gravels. Some of the materials encountered in the borings drilled at the site also contain cobbles and boulders. Groundwater was encountered within the Alluvium at depths of about 140 feet below the existing ground surface.

Station construction will consist of an excavation approximately 560 feet long, 60 feet wide, and up to 58 feet deep. The proposed tail track structure will be about 450 feet long and will consist of twin reinforced concrete box structures which are about 21 feet wide and 21 feet high. The depth of the excavation for the tail track will be about 55 to 56 feet below the existing ground surface. The Station and tail track structures will be bearing on the Alluvium and retaining alluvial deposits. Since the excavations will not extend through the groundwater table, dewatering should not be required.

Temporary support of the Station and tail track excavations will be provided by either a conventional or a conservative type shoring system with internal bracing or external tieback systems. Successful installation of soldier piles and tiebacks will require certain precautions to maintain the stability of such borings, especially when drilling within the cobble and boulder zones. Lateral pressures and other guidelines for design of temporary support systems are provided in this report.

The undisturbed natural Alluvium will adequately support the permanent reinforced concrete Station and tail track structures. Design lateral pressures for permanent structures for various loading conditions are outlined in the text of the report.

1.2 UNDERPINNING

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

1.3 SEISMIC CONSIDERATIONS

Since the available data suggest that groundwater levels are about 140 feet below the existing ground surface and well below the bottom of the proposed excavations, liquefaction should not be a hazard at this site.

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD report entitled "Guidelines for Seismic Design of Underground Structures" dated March 1984. Seismological conditions which may impact the project and the operating and maximum design earthquakes which may be anticipated in the Los Angeles area are described in the SCRTD report entitled "Seismological Investigations and Design Criteria" dated May 1983. The 1984 report complements and supplements the 1983 report. Site specific static and dynamic properties for materials in Design Unit A445 are given in the report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A445. The subject design unit includes the proposed North Hollywood Station and a 450-foot long cut-and-cover tail track structure which runs north of the Station site. These structures 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 A445:

- "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. This design unit also includes the Crossover Structure situated at 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. A445, North Hollywood Station," Sheets 1 to 19 of 26, dated July 1983, and "Report for the Development of Milestone 10: Fixed Facilities," dated September 1983 and revised plans A-67 through A-71. 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 North Hollywood Station and tail track structure, as shown on Drawings Nos. 2 and 3, are aligned southeast to northwest. The structures will be located under Lankershim Boulevard spanning Chandler Boulevard. The ground surface along Lankershim Boulevard slopes toward the southeast. Ground surface elevations vary from about Elevation 635 at the north end of the tail track structure to Elevation 628 at the south end of the Station structure.

The area around the Station has many different land uses. The Burbank Line of the Southern Pacific Railroad runs within the wide median divider of Chandler Boulevard. The tracks of this line crosses over the proposed Station site (refer to Drawing Nos. 2 and 3). Auto dealerships are located along Lankershim to the north. Low-rise commercial/retail space predominates along Lankershim to the south. The area along Chandler is used for industrial and warehousing purposes. An office/warehouse facility extending from Tujunga westward along Chandler was recently completed. Residential land use exists to the north and east of the station site.

3.2 PROPOSED STRUCTURES

To accommodate the two widely spaced entrances, the Station has a mezzanine at each end of the platform. A double Crossover structure will be located at the south end of the Station. The Crossover structure is not part of Design Unit A445 but is included as part of Design Unit A430. A 450-foot long tail track structure will proceed north from the North Hollywood Station beneath Lankershim Boulevard. This structure lies roughly between Stations 1053± and 1057.5±. The Station, Crossover, and tail track structures will be constructed using the cut-and-cover method. A traction power substation will be located over the Crossover track.

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 628 feet at the south end of the Station to Elevation 632 feet at the north end. The top of rail varies between about Elevation 580 and 581 feet. The depths of excavation for the Station structure will range from about 55 feet below the existing ground surface at the south end to a depth of 58 feet at the north end. After the Station is constructed, between 8 and 12 feet of fill will be placed above the Station box structure.

The tail track will consist of twin reinforced concrete box structures which are about 21 feet wide and 21 feet high. The top of rail varies between about Elevation 582 feet at the south end to Elevation 583 at the north end. The depth of the excavation for the tail track structure will be about 55 to 56 feet below the existing ground surface. After the tail track structure is built between 34 and 35 feet of fill will be placed above it.

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 samples.

4.2 BORINGS

A total of 10 exploratory boreholes have been drilled at or in relatively close proximity to, the proposed Station and tail track structures of Design Unit A445. Of the 10 borings, 8 are rotary wash type borings and 2 are large-diameter or "man-size" auger holes. One rotary-wash boring was drilled as part of the 1981 geotechnical investigation and 7 borings were drilled for this investigation during November of 1983. The large-diameter boreholes were drilled in September 1983.

Locations of all the borings used in the interpretation of the subsurface conditions present at the proposed North Hollywood Station, Crossover, and tail track structure sites are shown in Drawings 2 and 3. 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 3 of the borings drilled at or near the Station site. Groundwater was not observed in the large-diameter boreholes. Groundwater levels have been measured in only one of the piezometers installed at or near the site (i.e., the piezometer in Boring CEG-38). Groundwater conditions at the Station site and along the alignment of the tail track structure are discussed in Section 5.3).

4.3 GEOPHYSICAL MEASUREMENTS

A downhole compression and shear wave velocity survey was made in Borehole CEG-38 during the 1981 geotechnical investigation. This boring is about 600 feet east of the proposed North Hollywood Station site.

The downhole survey was conducted down to a depth of about 200 feet. The results of the survey are summarized in Appendix B. A discussion of the procedures employed in the field to perform the survey is also provided.

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 or along the tail track structure.

4.5 WATER QUALITY ANALYSES

Chemical analyses have been performed on one water sample obtained from near the site. The water sample was obtained at a depth of about 138 feet from Borehole CEG-38. This boring is located about 600 feet east of the North Hollywood Station site. Tests were performed as part of the 1981 geotechnical investigation. Results of these tests are reported in Section 5.3 and Appendix C.

4.6 GEOTECHNICAL LABORATORY TESTING

A laboratory testing program was performed on representative soil samples. The tests included classification tests, triaxial compression tests, unconfined compression tests, and direct shear tests.

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

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

Design Unit A445 includes the portion of the North Hollywood Station starting from about Station 1048± and ending near Station 1053±. It also includes a 450-foot long cut-and-cover tail track structure which begins at the northern end of the North Hollywood Station and ends at about Station 1057.5±

The Crossover structure situated on the southern end of the North Hollywood Station is part of Design Unit A430 and has therefore not been included as part of this report. However, the subsurface conditions which are described later in this Chapter are also applicable to the Crossover structure.

Drawing No. 2 shows a generalized subsurface cross-section at the North Hollywood Station site and along the tail track structure north of the Station. Drawing No. 4 shows a more detailed subsurface profile through the Crossover structure, Station, and tail track structure.

The geologic sequence in the site area consists of Young Alluvium (A_1) overlying Old Alluvium (A_3). The younger alluvial soils are believed to extend to a depth of about 90 feet (refer to Drawing No. 2 of the 1981 geotechnical investigation, CWDD/ESA/GRC, 1981). Below this depth, the Old Alluvium extends to a depth of at least 200 feet. This is the maximum depth that has been penetrated by the exploratory boreholes drilled at or near the Station site (see Appendix A, Boring CEG-38). The actual depth of the alluvial deposits at the site was not determined during the course of this investigation. However, the San Fernando Valley Groundwater Basin, in which the site is situated, has sediments which reach depths of up to 1000 feet in some places.

As shown in Drawing No. 4, the approximate depth of the planned excavation varies from about 55 feet at the south end of the Station to about 58 feet at the north end. The top of the rail in the cut-and-cover tail track structure is situated about 50 feet below the present ground surface. Therefore, the North Hollywood Station, as well as the tail track structure included in Design Unit A445, will be entirely founded on (or within) the Young Alluvium. Descriptions of the various soils which comprise the alluvium within Design Unit A445 and its general vicinity are described in detail in the following section.

During the field programs conducted for this and the 1981 investigations, the contact between the Old and Young Alluvium was difficult to identify since soils in these two deposits are generally very similar. For the purposes of this report, Young and Old Alluvium have not been differentiated and are simply referred to as Alluvium.

5.2 SUBSOILS

As discussed in Section 4.2, the subsurface conditions at the Station site were investigated by drilling a total of 7 rotary-wash borings (i.e., Borings 38-1 through 38-7) during the course of this investigation. In addition to these borings, 1 rotary-wash boring (Boring CEG-38) was drilled about 600 feet east of the North Hollywood Station site during the 1981 geotechnical investigation, and two large-diameter or man-sized borings (Borings 38A and 38B) were drilled in relatively close proximity to the Station site and tail track structure in September 1983 (refer to Appendix A and Drawing No. 2 for the locations of all borings).

Our interpretation of the subsurface conditions at the North Hollywood Station site is shown in Drawing No. 4. In general, the upper 45 to 50 feet of the alluvium consists primarily of sands and gravelly sands. Underlying the generally sandy soils, the alluvium consists of primarily gravelly sands and sandy gravels, some of which contain cobbles and boulders. These materials were encountered down to a depth of about 80 feet which is the maximum depth explored by the boreholes drilled at the Crossover and Station site and along the tail track structure alignment. Specific description of the various soils are as follows:

- Upper Sands: Within this generally sandy unit, the materials are predominantly silty sands, some clean fine to coarse sands, and gravelly sands. The thickness of these soils ranges from 45 to 50 feet across the Station site and along the cut-and-cover tail track structure. Some of these soils contain scattered cobbles or small boulders. Relatively thin, discontinuous lenses or layers of clays, silts, and/or clayey sands were also found to be present within the upper sands. Results of Standard Penetration Tests (SPT) in the various soils which comprise the upper sands range from 4 to well over 100 blows per foot. Blow count measurements believed to be affected by the presence of gravel have been eliminated from this and all other ranges that will be discussed. The lowest SPT blow count measurements were recorded in the upper 10 to 15 feet of the subsurface profile, with values ranging from a low of 4 blows per foot to a high of 22 blows per foot. These measurements together with laboratory test results indicate that some of these near-surface soils are generally loose to medium dense. At depths greater than 10 to 15 feet, the SPT blow counts increase rather significantly with values typically being between 30 and 70 blows per foot, although higher blow counts were recorded. These measurements and laboratory test results indicate that these soils are generally dense to very dense. A limited number of SPT measurements taken in the relatively thin, discontinuous lenses or layers of clays, silts, and clayey sands suggest that these soils are very stiff to hard and medium dense to very dense.

- Lower Gravels: The alluvium below a depth of about 45 to 50 feet consists primarily of sandy gravels. Interbeds of gravelly sand are also present. Some thin lenses/layers of sand, silt and clay were also occasionally encountered within this gravelly unit. Due to the gravel content, sample recovery was generally poor and was limited to soil particles smaller than the inside diameter of the samplers (i.e., 1.4 to about 3 inches). Observations made in the large-diameter or man-sized auger borings (Boring 38A and 38B) and on the drilling action noted in the logs of the rotary-wash borings suggest that the soils of this unit grade through coarse sand and gravels with occasional cobbly zones. Boulders up to about 1 foot in diameter are reported in the logs of the large-diameter and rotary-wash borings; however, boulders of larger diameter (on the order of 2 to 4 feet) may also be encountered during excavation.

In general, SPT measurements were not taken in the soils of this unit due to the high gravel content. When they were taken, they were exceptionally high and are considered unrepresentative. Some minor belling or sloughing occurred in these soils during the drilling of the large-diameter boreholes, but this could be due to the relatively high percentages of gravels and cobbles and/or the vibrations caused by drilling action of the auger bucket. Based on this observed behavior, the materials which make up this gravelly unit are judged to be medium dense to dense.

During the drilling of the rotary-wash borings at the Station site, some difficulty was experienced in sampling the first 10 to 15 feet of the upper sands. As was noted in the description of this material type, the SPT blow counts measured in some of these soils were relatively low. Sample recovery of these soils was also sometimes poor since the soil samples tended to wash out of the sampler during cutting, or pulled or fell out when bringing the sample to the surface. This type of sampling difficulty was noted in Borings 38-3 through 38-6 but not in Boring 38-1, 38-2, or 38-7.

The large-diameter borehole, Boring 38A, which was drilled just south of the Crossover structure, experienced some very minor ravelling between the depths of 10 and 14 feet. The log of the other large-diameter hole, Boring 38B, drilled at the extreme northern end of the cut-and-cover tail track structure, indicated that the hole stood up well with no caving from the ground surface to a depth of 50 feet. Therefore, this behavior suggests that the loose soil conditions which have been noted or inferred from the logs of the rotary-wash boreholes may be present at Boring 38A but not at Boring 38B. Based on the above information, it is likely that loose soils will be randomly encountered within the upper 10 to 15 feet of the excavation required at the Station site, and along the alignment of the cut-and-cover tail track structure.

The behavior of the soils encountered in the large-diameter boreholes (i.e., 38A and 38B) was in general quite good considering that the majority of the soils were cohesionless and contained cobbles and boulders. In addition to the minor ravelling that occurred in Boring 38A as noted above, some caving of the boring also occurred between the depths of 50 to 60 feet. However, this was confined to the deeper gravelly sands and sandy gravels that contained cobbles. In Boring 38B, minor caving also occurred between the 50- and 60-foot depths. The materials encountered in this hole at these depths were similar to those observed in Boring 38A. As previously stated, this behavior could also be the result of the drilling action of the auger bucket.

5.3 GROUNDWATER

The proposed North Hollywood Station site and tail track structure included in Design Unit A445 lie within the San Fernando Valley Groundwater basin. 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 from a general southeast to northwest direction, and the groundwater table in the vicinity of the North Hollywood Station site is at about Elevation 490 (or about 140 feet below the ground surface).

Groundwater and/or seepage was not encountered in the two large-diameter boreholes drilled in the vicinity of the site, even though they were each 60 feet deep. The piezometers that were installed in Borings 38-4 and 38-6 were placed at depths of about 80 feet. Neither piezometer has contained water since they were installed in November 1983.

Water levels measured in the 200-foot deep Boring CEG-38 during the 1981 geotechnical investigation were at about 140 feet below the ground surface. This closely corresponds to about Elevation 490 and is in excellent agreement with the reported regional groundwater conditions.

During the 1981 geotechnical investigation, one water sample was taken from Boring CEG-38 at a depth of about 140 feet and was subjected to chemical analyses. Results of the analyses performed indicate that the groundwater is of poor quality (see Appendix C). Total Dissolved Solids (TDS) of the sample tested was in excess of 900 PPM. For comparison, the U.S. Environmental Protection Agency TDS standard for potable domestic drinking water is 500 PPM. The sulfate content of the sample was 463 PPM. A sulfate content above 150 PPM is generally regarded to be deleterious to concrete lining.

Since the depth to groundwater appears to be at least 80 feet deeper than the proposed excavations of Design Unit A445, it should have no influence on the construction operations nor on the design of the planned structures. Recommendations regarding corrosion protection of the Station structure will be provided by others.

5.4 OIL OR GAS

No strong or unusual odors were detected during the drilling and logging of the borings located in the vicinity of the North Hollywood Station site. The Station and tail track structure are not located in an oil-producing area or near known oil fields.

5.5 FAULTS

An unnamed postulated fault crosses the cut-and-cover tail track structure near Station 1055± (refer to Drawing No. 2). The fault is not known to be active or potentially active, nor does it appear to act as a groundwater barrier. This fault is expected to have little or no effect on the Metro Rail Project.

Additional information regarding this fault is contained in the 1981 geotechnical investigation report (Volume 1, Section 4.4.2.12).

5.6 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.6.1 General

For purposes of our engineering evaluations, we have grouped the subsurface materials encountered at the North Hollywood Station site and along the alignment of the tail track structure into general subsurface units. The two main subsurface units that were described in detail in Section 5.2, and include the Upper Sand Unit and the Sandy Gravel Unit.

As was discussed in Section 5.2, evidence suggests that the sands encountered within 10 to 15 feet of the ground surface are generally loose to medium dense. Below this depth and to a depth of about 50 feet, subsurface and laboratory test data indicate that the generally sandy soils are dense to very dense. Below the depth of about 50 feet, the soils of the Sandy Gravel Unit were encountered.

Material properties were developed for the loose to medium dense soils that were encountered in the first 10 to 15 feet of the subsurface profile of Design Unit A445 and for the sands and gravelly sands that are present at depths up to about 50 feet. The engineering parameters developed for these two soil types are summarized in Table 5-1. These parameters are based mainly on laboratory and field test results and field observations of their behavior.

Because of the high gravel content and the presence of cobbles and boulders encountered in the soils at depths greater than about 50 feet, good quality, relatively undisturbed representative samples of these materials could not be obtained for laboratory testing. Thus, it was necessary to rely mainly on the results of laboratory tests performed on the shallower soils, published data for gravelly materials, observed behavior of these materials in the large-diameter boreholes, and engineering judgment in selecting appropriate material properties for these gravelly soils.

It is our judgment that the material properties selected for the sands and gravels provide a conservative estimates for the sandy and gravelly soils encountered below a depth of 50 feet. The parameters listed in Table 5-1 were used for engineering analyses, the results of which are presented in Chapter 6.0.

5.6.2 Upper Sands

The soils encountered within the first 10 to 15 feet of the surface consists of silty and poorly graded sands. These soils appear to be generally loose to medium dense. Below these soils and down to a depth of about 50 feet, the soil profile consists of similar soil types as well as gravelly sands and sandy gravels. Cobbles and boulders are also present in these soils. The soils of this unit are generally dense to very dense.

Table 5-1

RECOMMENDED STATIC MATERIAL PROPERTIES FOR USE IN DESIGN

Material Property	Alluvium	
	Upper Sands[a]	Sands and Gravels[b]
Moist Density (pcf)	115	130
Effective Stress Strength		
ϕ' (degrees)	35	38
c' (psf)	0	0
Initial Tangent Modulus (psf)	300 σ'_v[c]	500 σ'_v[c]
Poisson's Ratio	0.35	0.35

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[a] Apply to soils within the upper 15 feet.

[b] Apply to soils between the depths of 15 and about 50 feet. Below a depth of 50 feet and to a depth of at least 80 feet, the properties listed in this column are conservative estimates for the types of materials encountered in the boreholes.

[c] σ'_v is the effective overburden pressure (psf) equal to moist density times overburden depth.

The properties which are listed in the first column of Table 5-1 are appropriate for the soils encountered in the first 15 feet below the ground surface. Those listed in the second column are for the sands and gravels encountered between the depths of about 15 and 50 feet. Permeabilities are not listed for either material since the groundwater level within Design Unit A445 is well below the bottom of the planned excavations.

Strength tests performed on the materials include both direct shear and triaxial compression. Drained (effective) strength parameters are considered appropriate for static design. 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.

Relatively thin, discontinuous lenses or layers of clays, silts, and clayey sands are occasionally encountered within the main soil units. The consistency of these soils vary from stiff to hard and medium dense to very dense. Unconfined compression tests performed on three samples of the clayey soils ranged from 1850 psf to about 3000 psf; however, these results may be effected somewhat by sand or silt present in these soils.

5.6.3 Lower Gravelly Sands and Sandy Gravels

Below a depth of about 50 feet, the soils consist primarily of sandy gravels. Interbeds of gravelly sands are also encountered in this unit. Some thin lenses/layers of sand, silt, and clay are also present within this gravelly soil unit. Cobbles and boulders up to about 1 foot in diameter have been reported in the logs of the boreholes drilled in the vicinity of the Station and tail track structure; however, larger boulders will probably be encountered during excavation.

Since undisturbed sampling of the gravelly soils was not possible, a reasonable number of laboratory tests upon which to estimate material properties could not be performed. However, it is our judgment that the engineering parameters given in Table 5-1 for the sands and gravels are conservative estimates for these very gravelly soils.

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 North Hollywood Station, including the track structure at the north end of the 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 predominantly of a mixture of sands and gravels. As discussed in the previous section, the upper soils consist primarily of sands and gravelly sands, whereas the deeper soil deposits (at depths greater than about 50 feet) are generally sandy gravel with cobbles and boulders. As shown in Table 6-1, the depth of the excavations will range from 55 feet (Elevation 573) at the south end of the Station, to 58 feet (Elevation 574) at the north end of the Station, to 56 feet at the north end of the cut-and-cover tunnel segment. No groundwater was encountered at the Station site. The permanent structure will in essence be a concrete box bearing on the gravelly soils and retaining sand and gravel alluvial deposits.

The primary geotechnical considerations at the Station site include:

- Selection, design, and construction of the temporary shoring system and the permanent wall system.
- Determining the need for and type of underpinning.
- Establishing magnitude and distribution of soil pressures acting on the permanent structures, and designing for these loads.

6.2 EXCAVATION DEWATERING

No groundwater was encountered or observed at the Station site during the 1981 and 1983 field investigations. Thus, the only possible source of groundwater during excavation would be mainly due to infiltration of water from the ground due to rainfall, in addition to minor seeps. Dewatering due to these sources can be accomplished by use of sump pumps within the excavation combined wiht supplementary ditch drains. No major dewatering problems are expected to be encountered at the locations of the proposed structures.

Table 6-1

SUMMARY OF EXCAVATION AND GROUNDWATER DEPTHS AND ELEVATIONS, DESIGN UNIT A445--NORTH HOLLYWOOD STATION

	Elevation (feet)[1]				Depth (feet)[1]	
	Ground Surface	Top of Rail	Bottom of Excavation	Measured Water Level	Depth to Groundwater	Depth of Excavation
South End of Cross-over Structure	625	579	572	(2)	--	53
South End of Station	628	580	573	(2)	--	55
North End of Station	632	581	574	(2)	--	58
North End of Cut and Cover Tunnel Segment	635	583	579	(2)	--	56

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[1] All elevations and depths rounded to nearest foot.

[2] All piezometers at site have been dry. Water level at site believed to be in excess of 140 feet below existing ground surface.

6.3 UNDERPINNING

6.3.1 Underpinning/Support Methods

The need to underpin and the appropriate type of underpinning for specific structures located adjacent to the proposed excavations depends on many factors related to both engineering and economics. 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. Thus, each structure needs to be evaluated separately. The requirements for underpinning will be the responsibility of the section designers. However, to aid the designers in evaluating underpinning requirements, general geotechnical underpinning guidelines are presented in this section of the report.

There are several commonly used methods for underpinning. These include jacked piles, slant drilled piles (concrete cast-in-place), and hand-dug pit or pier underpinning. Another technique which has been used is the "column pick-up" method which provides a means of jacking up selected columns in the event that settlements do occur. These various techniques are discussed below.

- Jacked Piles: These piles generally consist of open end pipe piles 6 to 18 inches in diameter. These sections are generally preferred due to their relatively low volume displacement which facilitates placement. Open end pipe sections have the additional advantage of permitting clean-out to reduce point and shaft resistance during installation. If point resistance is to be relied on, the pipe should be filled with concrete prior to reaching its desired elevation.

- Slant Drilled Piles (Concrete Cast-in-Place): This method consists of placing a steel pile in a shaft (generally 12- to 24-inch diameter) drilled from the side of the foundation. The shaft is drilled at a small angle of slant under the foundation and then back-reamed to provide a vertical slot below the foundation. A steel pipe is placed under the foundation, and the shaft is filled with concrete. In weak soils or in ground subject to sloughing, this method can result in settlement if there is loss of ground into the drilled hole.

- Hand-Dug Pits: This method consists of excavating an approach pit beneath the footing and advancing square or rectangular shafts, normally 3 to 5 feet wide, down to the bearing stratum. The shaft excavations are lagged for the entire depth with the lagging normally left in place permanently. Reinforcement is placed, and concrete is tremied into the shaft(s).

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

6.3.2 Underpinning Considerations

From an engineering standpoint, the need to underpin is evaluated on the basis of expected ground movements and potential for structural damage. Figure 6-1 presents guidelines for assessing when underpinning needs to be considered. Review of Drawing 2 indicates that several low-rise commercial structures are located in close proximity to the tail track structure. In addition, an existing railroad crosses near the center of the Station site, and provision will be required for one operational track during construction of the Station. Thus, underpinning of these structures may be required. However, other considerations beyond the scope of this investigation should be considered in any final decisions regarding underpinning.

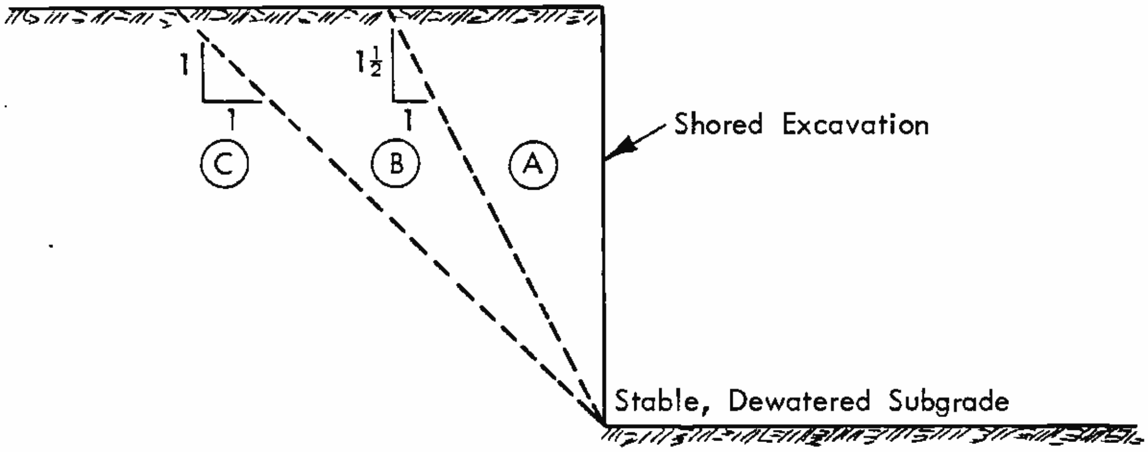
Based on the subsurface conditions existing at the Station site, underpinning piles can be adequately supported on the dense soils encountered at depths equal to or greater than 15 feet. The upper loose to medium dense sands may also provide adequate support for the lighter loads associated with some of the low-rise commercial buildings. Some sloughing and ravelling of these upper sands will occur during drilling of shafts and excavation of pits. Use of jacked piles below a depth of 15 feet will not prove feasible due to the denseness of these sandy and gravelly materials.

6.3.3 Design Criteria

Figures 6-2 through 6-5 present geotechnical criteria for jacked circular pipe piles and slant drilled piles. Figure 6-2 illustrates the procedures for determining the geometry of the support zones required to use Figures 6-3 through 6-5. No support should be allowed within any existing fill soils encountered or above the "no support" line shown on Figure 6-2.

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

Total capacity of hand-dug, lagged piers should be limited to end bearing only and must extend below the "no support" line shown on Figure 6-2. All piers are assumed to be 36-inch square or larger in section. For design, an allowable bearing capacity of 6 ksf may be used for piers which bear on the undisturbed alluvium and penetrate at least 10 feet below the ground surface. This value applies only if the bearing surface is properly cleaned and approved by a qualified engineer.



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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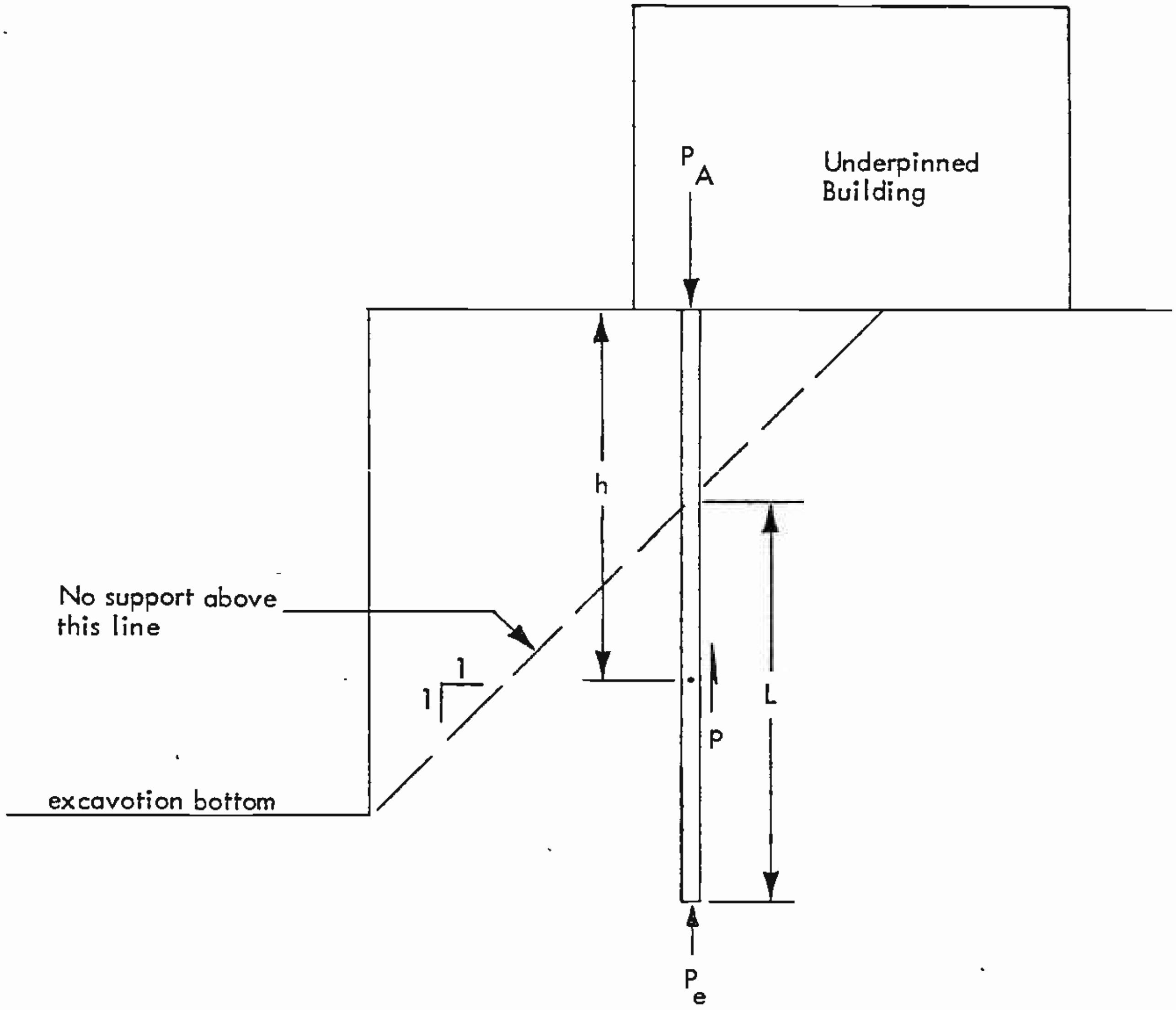
Figure No.
6-1



$$P_A = p \cdot L + P_e$$

WHERE:

*p = average frictional resistance at h

**P_e = end bearing support

* See Figures 6-3 and 6-4 for values of p

** See Figure 6-5 for values of P_e

UNDERPINNING - DESIGN CAPACITY CRITERIA

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Figure No.
6-2

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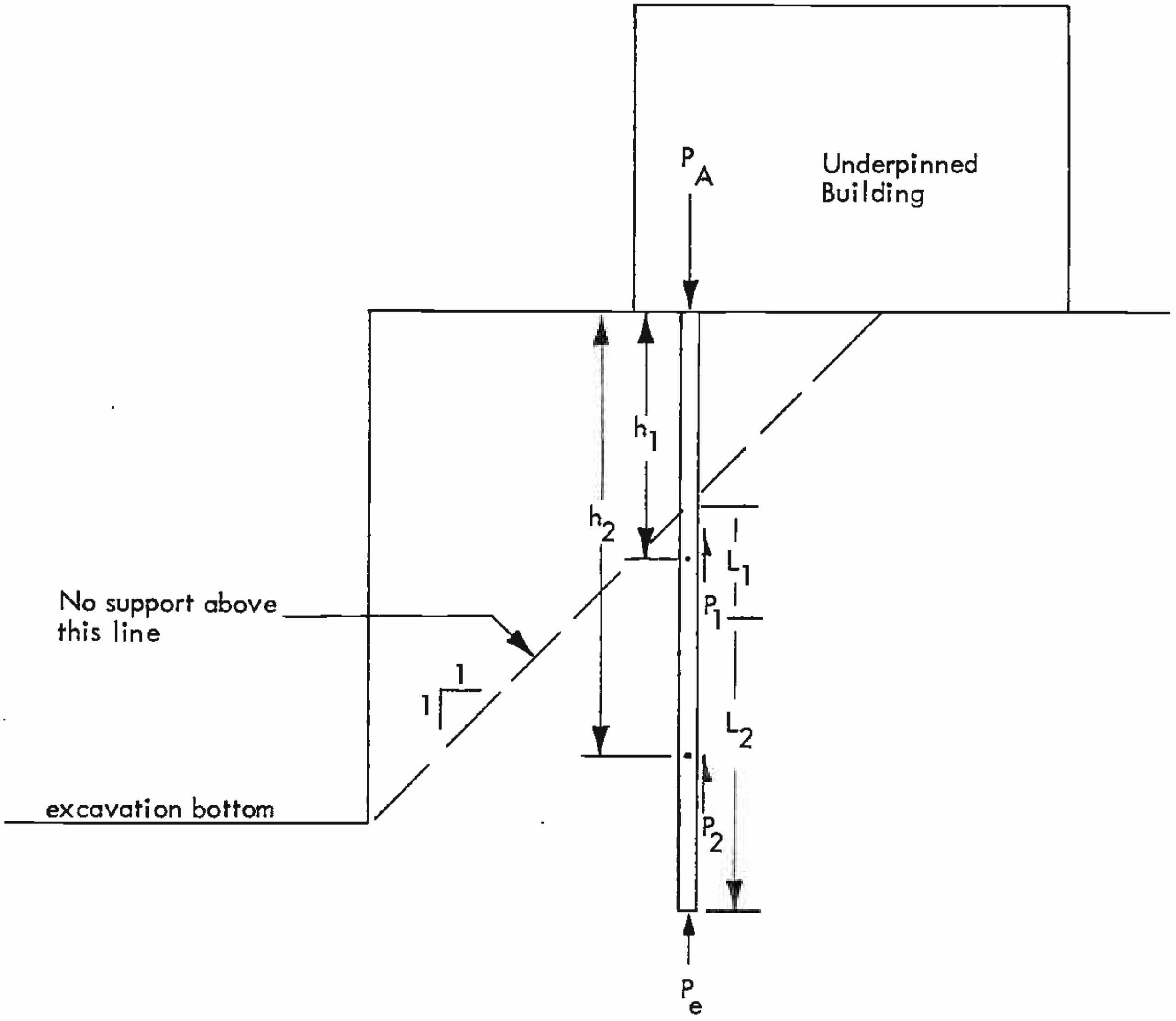

$$P_A = p_1 \cdot L_1 + p_2 \cdot L_2 + P_e$$

WHERE:
$*p_1$ = average frictional resistance at h_1
$*p_2$ = average frictional resistance at h_2
$**P_e$ = end bearing support

* See Figures 6-3 and 6-4 for values of P_1 and P_2

** See Figure 6-5 for values of P_e

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UNDERPINNING - DESIGN CAPACITY CRITERIA

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Project No.
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Figure No.
6-2

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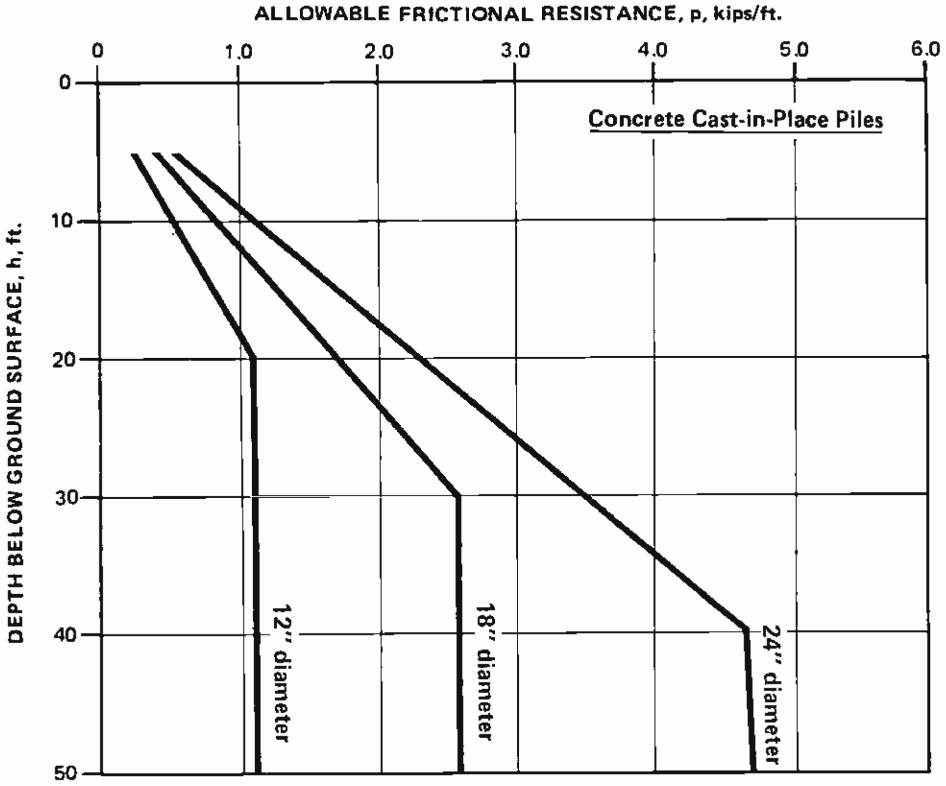

NOTES:
1) See Figure 6-2 for evaluation of total vertical pile capacity. Figure 6-5 should be used to evaluate capacity due to end-bearing support.

2) Maximum pile frictional resistance taken as value corresponding to a depth equal to 20 times diameter of pile.

3) Applicable to drilled cast-in-place piles.

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UNDERPINNING – VERTICAL PILE FRICTIONAL RESISTANCE IN ALLUVIUM (CONCRETE CAST-IN-PLACE PILES)

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83-1140

Figure No.
6-3

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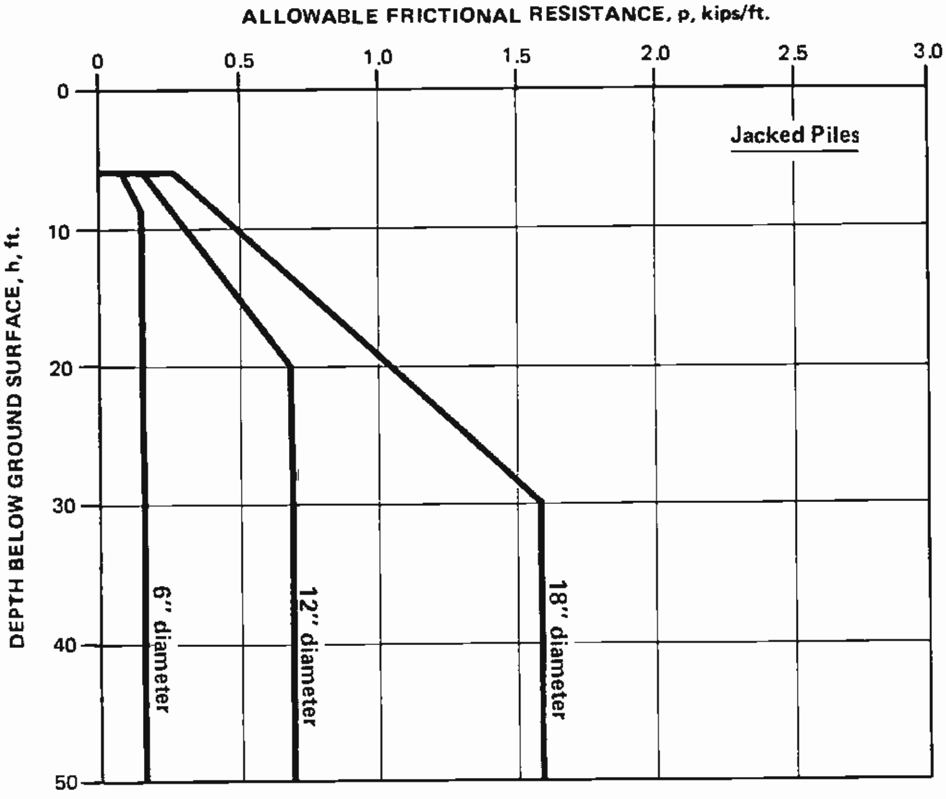

NOTES: 1) See Figure 6-2 for evaluation of total vertical pile capacity. Figure 6-5 should be used to evaluate capacity due to end-bearing support.

2) Maximum pile frictional resistance taken as value corresponding to a depth equal to 20 times diameter of pile.

3) Applicable to circular section pipe piles.

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UNDERPINNING – VERTICAL PILE FRICTIONAL RESISTANCE IN ALLUVIUM (JACKED PILES)

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83-1140

Figure No.
6-4

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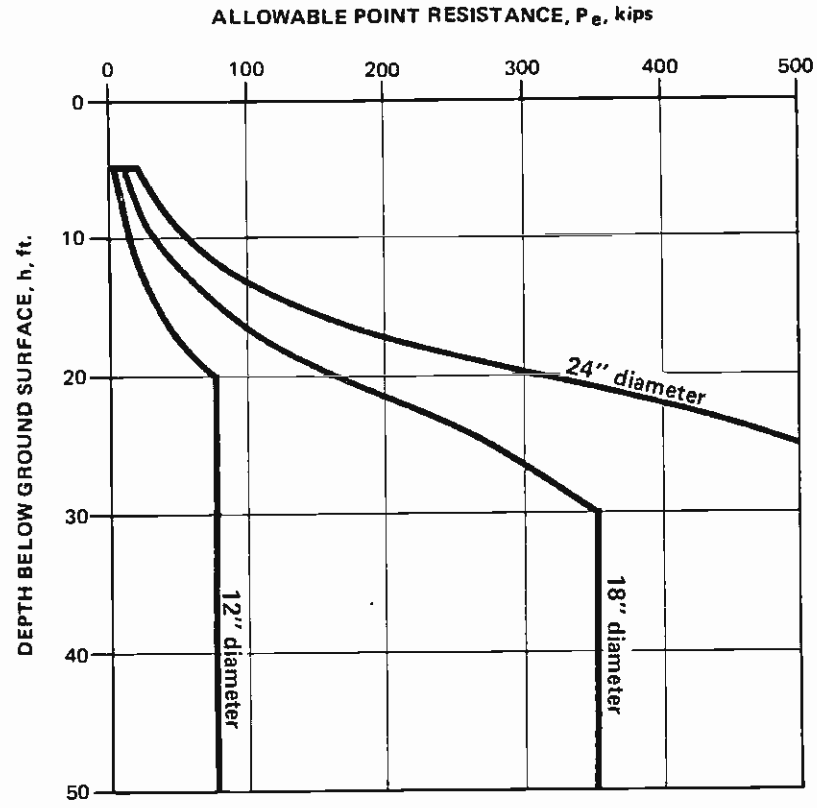

NOTES: 1) See Figure 6-2 for evaluation of total vertical pile capacity. Figures 6-3 and 6-4 should be used to evaluate capacity due to frictional resistance along length of pile.

2) Maximum pile capacity taken as value corresponding to a depth equal to 20 times diameter of pile.

3) Applicable only to properly cleaned out drilled cast-in-place piles and jacked piles consisting of circular pipe piles filled with concrete. Not applicable to H-piles.

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UNDERPINNING – VERTICAL PILE END-BEARING RESISTANCE IN ALLUVIUM
FOR CAST-IN-PLACE OR JACKED PILES

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Figure No.
6-5

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The expected lateral ground movements due to the Station and tail track structure excavations are discussed in Section 6.4.6. The capability of the existing structure and underpinning system to sustain these lateral movements should be evaluated. If it becomes necessary to reduce the magnitude of the expected movements, additional lateral restraint should be provided by tieback anchors or other methods.

6.3.4 Underpinning Performance

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

6.3.5 Underpinning Instrumentation

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

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

6.4 TEMPORARY SLOPED EXCAVATIONS AND SHORING SUPPORT SYSTEMS

6.4.1 General

The required excavation depths below the existing ground surface are tabulated in Table 6-1. There are several ways to construct the excavation for both the Station and tail track structure. A conventional shoring system with underpinning of adjacent structures as required, or a conservatively designed shoring system which would eliminate or reduce the need to underpin could be used. Driven sheet piles are not considered feasible due to the presence of the dense alluvial sands and gravels, which would make driving extremely difficult, if not impossible, in these 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 contractor may propose one of the following shoring systems with either tiebacks or internal bracing for lateral support:

- Conventional Shoring System: Buildings or structures located within the underpinning zone (see Figure 6-1) may require underpinning.

- Conservative Shoring System: This could consist of a conservatively designed wall which may limit ground movements sufficiently to eliminate or reduce the need for underpinning (refer to Section 6.4.6).

The discussions and design criteria presented in this section pertain to these general shoring methods. Other shoring support systems may also be appropriate and may be considered by the contractor.

6.4.2 Sloped Excavations

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 the 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 primary 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 upper sand deposits. These recommendations assume suitable site dewatering as necessary, no heavy loads at the top of the slope, slope protection, and some slope maintenance. In addition, these recommendations should not be constructed 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 Temporary 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. Either a conventional or a conservative shoring system may be used at the Station site, and for the tail track structure. The conservative wall should be designed for higher soil loads since this will reduce ground movements behind the wall. Appendix E.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 E.1 have not been verified by measurements during construction. However, the performance of shoring systems designed based on local practice has been satisfactory. 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 which are finer than those encountered at the proposed Station site. Within the coarse-grained materials, caving can be a problem. The contractor should recognize that caving conditions may be encountered in installation of soldier piles or other drilled shaft elements such as tiebacks, especially due to vibratory motions induced by construction equipment.

The granular 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 both conventional and conservative shoring systems. 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.

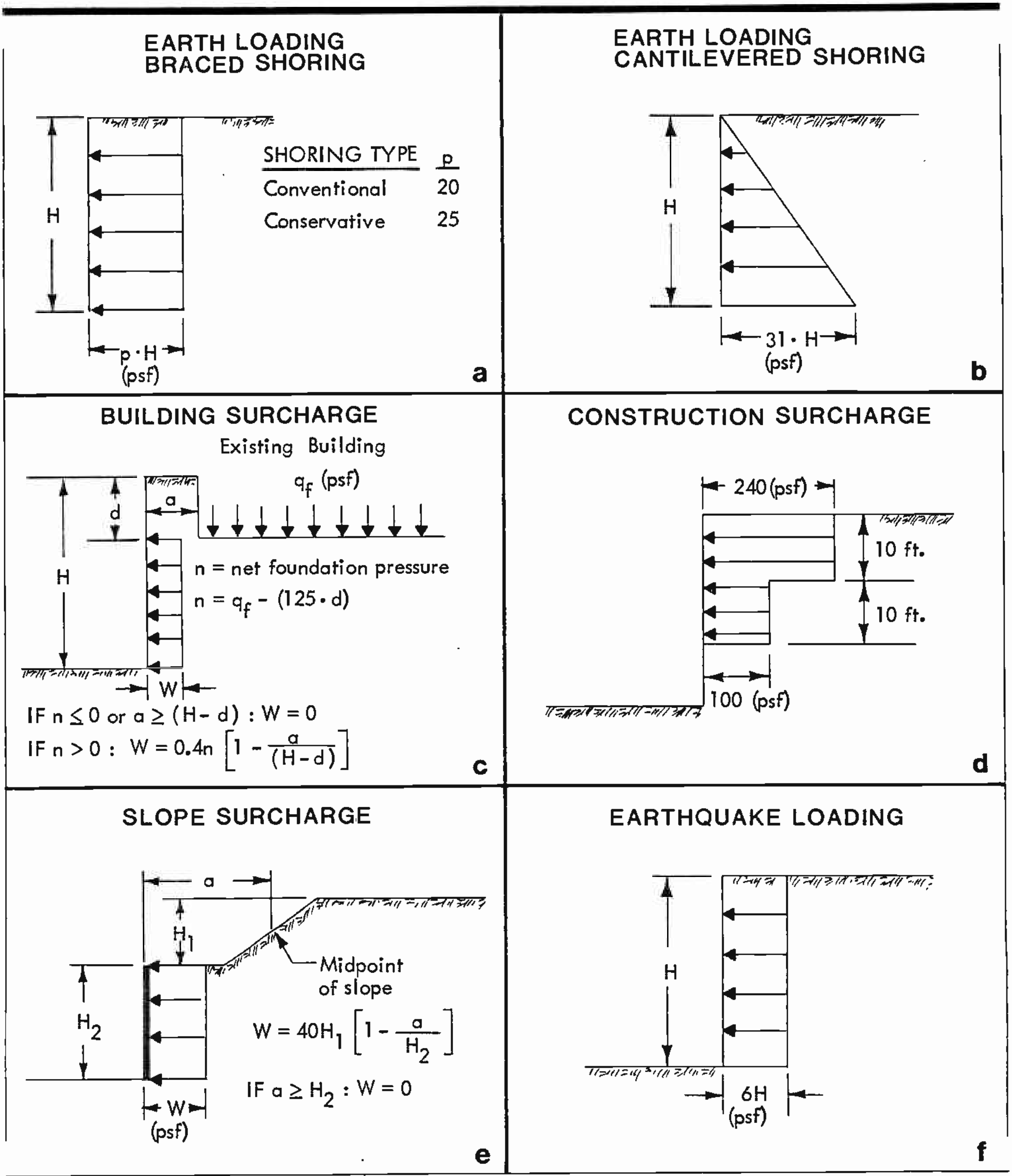
Specific shoring design criteria include:

- Design Wall Pressure: Figures 6-6a and 6-6b present the recommended lateral earth pressure on the temporary shoring walls. Design lateral pressures for both conventional and conservative shoring systems are presented in Figure 6-6a. Figure 6-6e 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 pile capacities under static and dynamic loading conditions.

  The required depth of embedment to satisfy vertical loads should be computed based on allowable vertical loads shown on Figure 6-7. This figure may also be used for design of piles to support the railroad crossing over the excavation.

  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-6 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 6-8 indicates the recommended method to compute net passive resistance.



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No.
6-6

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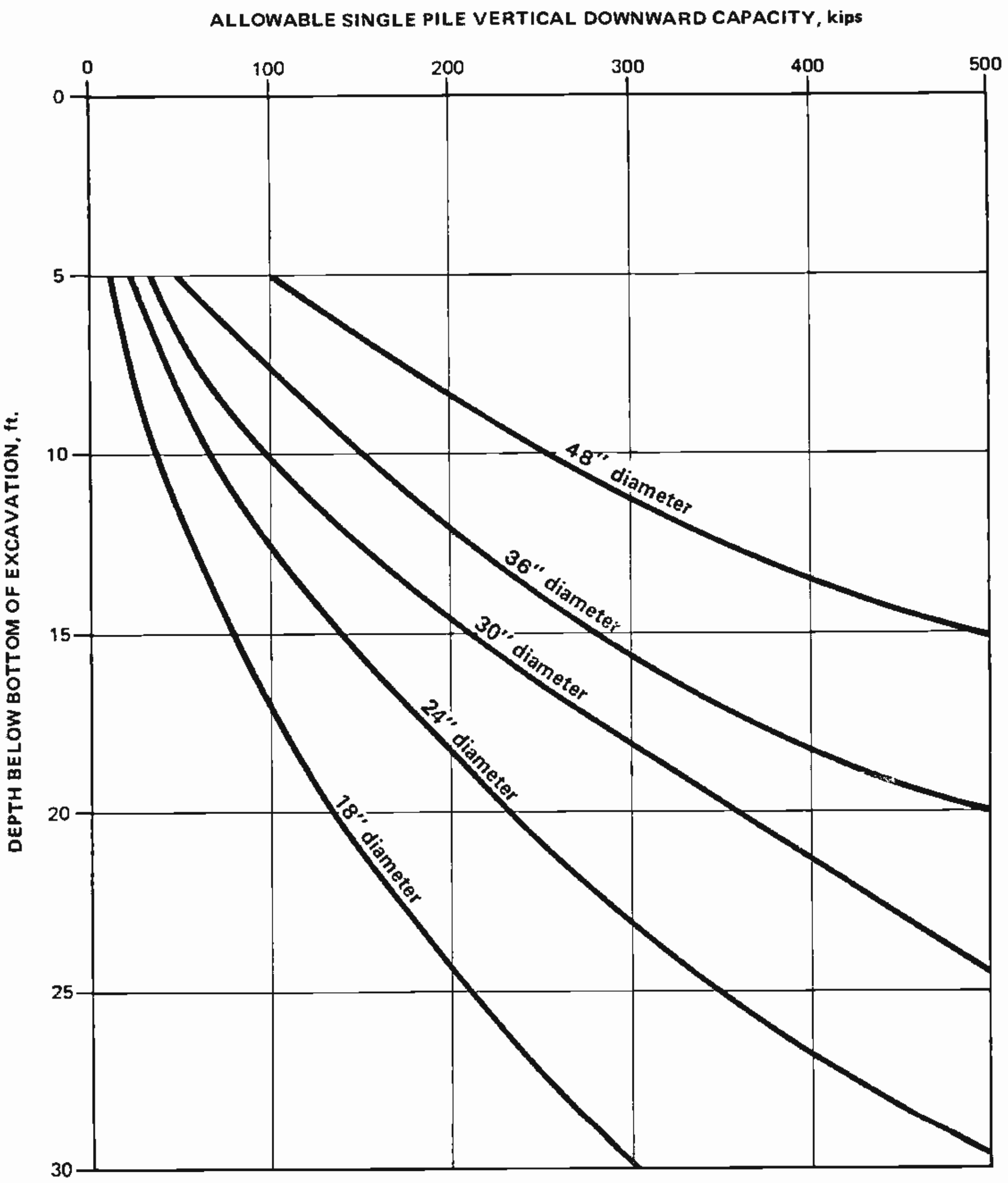

NOTES:
1) Total capacity includes contribution from both shaft frictional resistance and end-bearing.
2) For seismic design, capacities may be increased by one-third.
3) Applicable only for drilled shaft piles.

ALLOWABLE VERTICAL PILE CAPACITY IN ALLUVIUM

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Figure No.
6-7

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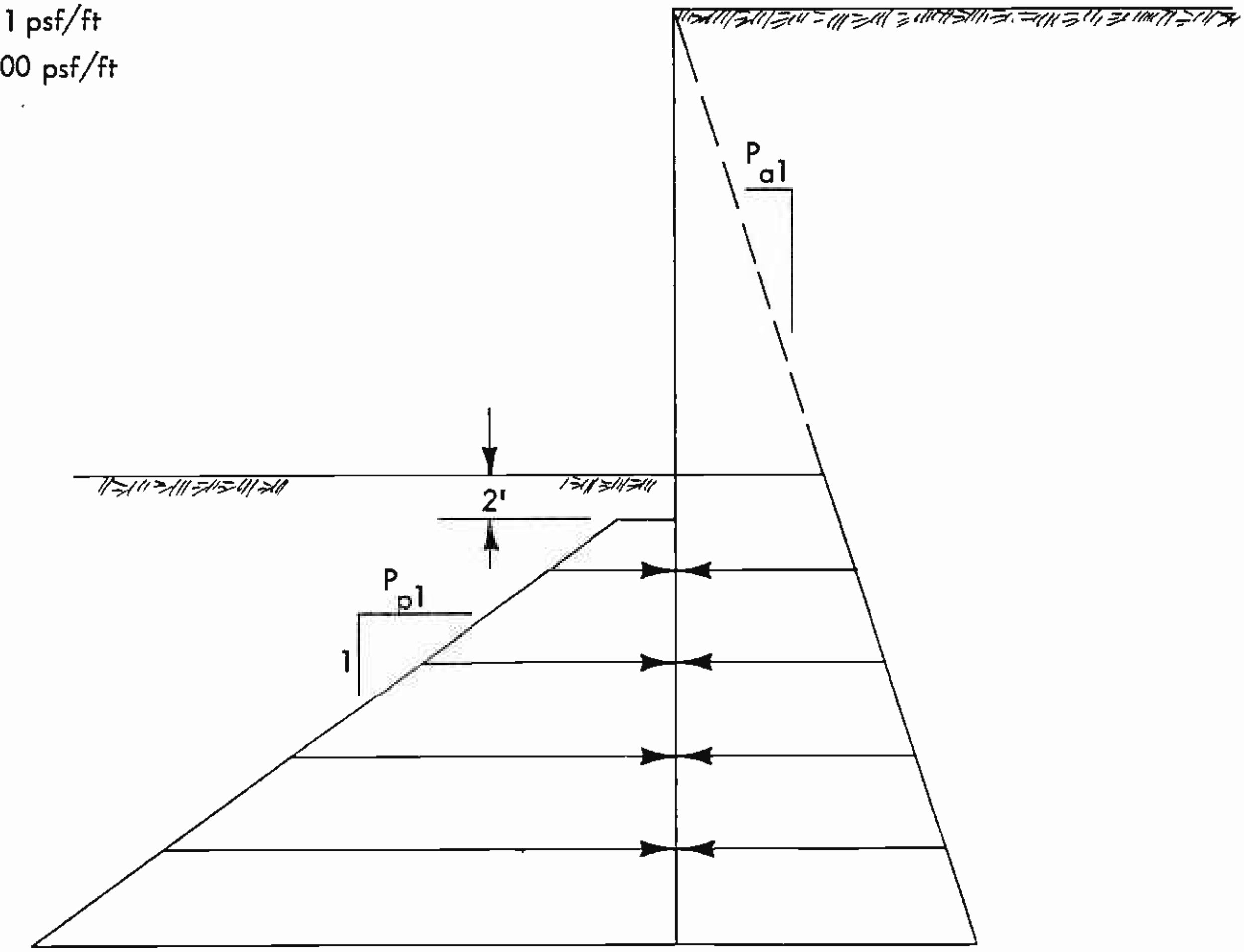
Recommended Unit Pressures

P_{a1} = 31 psf/ft

P_{p1} = 400 psf/ft



Where: P_p =Allowable unit passive pressure

P_a = Unit active pressure

NOTES: 1.) Available passive pressure = Total Passive - Active

2.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.

3.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-6

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SOLDIER PILE PASSIVE RESISTANCE

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Figure No.
6-8

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil loads, member sizes, and costs. At the Station site the upper soils are generally sandy which may make them subject to ravelling and sloughing. Thus, it is recommended that the pile spacing be limited to about 6 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 feet to control ravelling problems.

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. Obtaining permission to install tiebacks under adjacent properties and encountering obstructions from adjacent below grade structures (such as basements) can affect the feasibility of tiebacks.

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 consideration of the following maximum allowable vertical distances between struts:

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

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

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable. 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 in association with stable soil conditions. The contractor should be familiar with City and County of Los Angeles Requirements for removal of tieback anchors.

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 in the alluvium be computed based on the following equation:

$$P = \pi DLq \quad \text{(anchor capacity)}$$

where

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

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

$$q = 20d < 1000 \text{ psf} \quad ,$$

where:

d = average depth of the anchor in feet beyond the no-load line; 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, especially in the zone containing boulders and cobbles, could be a problem particularly as a result of vibratory motions produced by construction equipment. 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 combined with our engineering judgment, we estimate that the ground movements associated with properly designed and carefully constructed shoring systems will be as follows:

- Conventional Wall With Tieback Anchors: The maximum horizontal wall deflection will equal about 0.1% to 0.2% of the excavation depth. The maximum horizontal movement should occur near the top of the wall and decrease with depth. The maximum 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.

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

- Conservative Wall With Tiebacks: We believe that the wall systems designed by utilizing the higher earth pressures presented for conservative walls will reduce ground movements and limit the maximum horizontal and vertical movements to about 0.1% of the excavation depth.
- Conservative Wall With Internal Bracing: Similar to those described above for the conservative tieback supported wall.

6.5 SUPPORT OF TEMPORARY DECKING

Where temporary street decking and operational railroad trestle across the excavation require center support piles, the piles would have to extend below the maximum proposed excavation level for support. At these depths, the piles would be founded within the deeper gravelly deposits which contain cobbles and boulders. These materials are suitable for supporting pile loads.

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that he may use similar piles to support the center decking and railroad tracks. Accordingly, the allowable loads on these types of piles have been evaluated for several typical diameters. The recommended allowable design loads are shown on Figure 6-7. These values include both end bearing and shaft friction.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed excavation at the Station site and within the tail track structure 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 each 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.

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

- Inclinometers: It is recommended that a limited number of inclinometers be installed prior to excavation and monitored around the Stations' 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 each 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.

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

- Frequency of Readings: An appropriate frequency of instrumentation readings depends on many factors including the construction progress, the results of the instrumentation readings (i.e., if any unusual readings are obtained), costs, and other factors which cannot be generalized. The devices should be installed and initial readings should be taken as early as possible. Readings should then be taken 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 excavations will substantially change the ground stresses below and adjacent to the excavations. The proposed 55- to 58-foot excavation at the North Hollywood Station and adjacent tail track structure 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. The Station structures and subsequent backfilling will reload the soils. We estimate that the Station and backfill loads will be in the range of 4500 to 5000 psf. For the tail track structure, the backfill loads will be about 4500 psf.

The maximum heave at the center of the excavations was estimated to be on the order of 1-1/2 to 3 inches. The majority of this should occur during the construction phase. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (elastic heave) within the soils underlying the proposed excavations.

Settlement on the order of 1 to 3 inches were computed due to the imposed loads from the structures 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 and Tail Track Structures

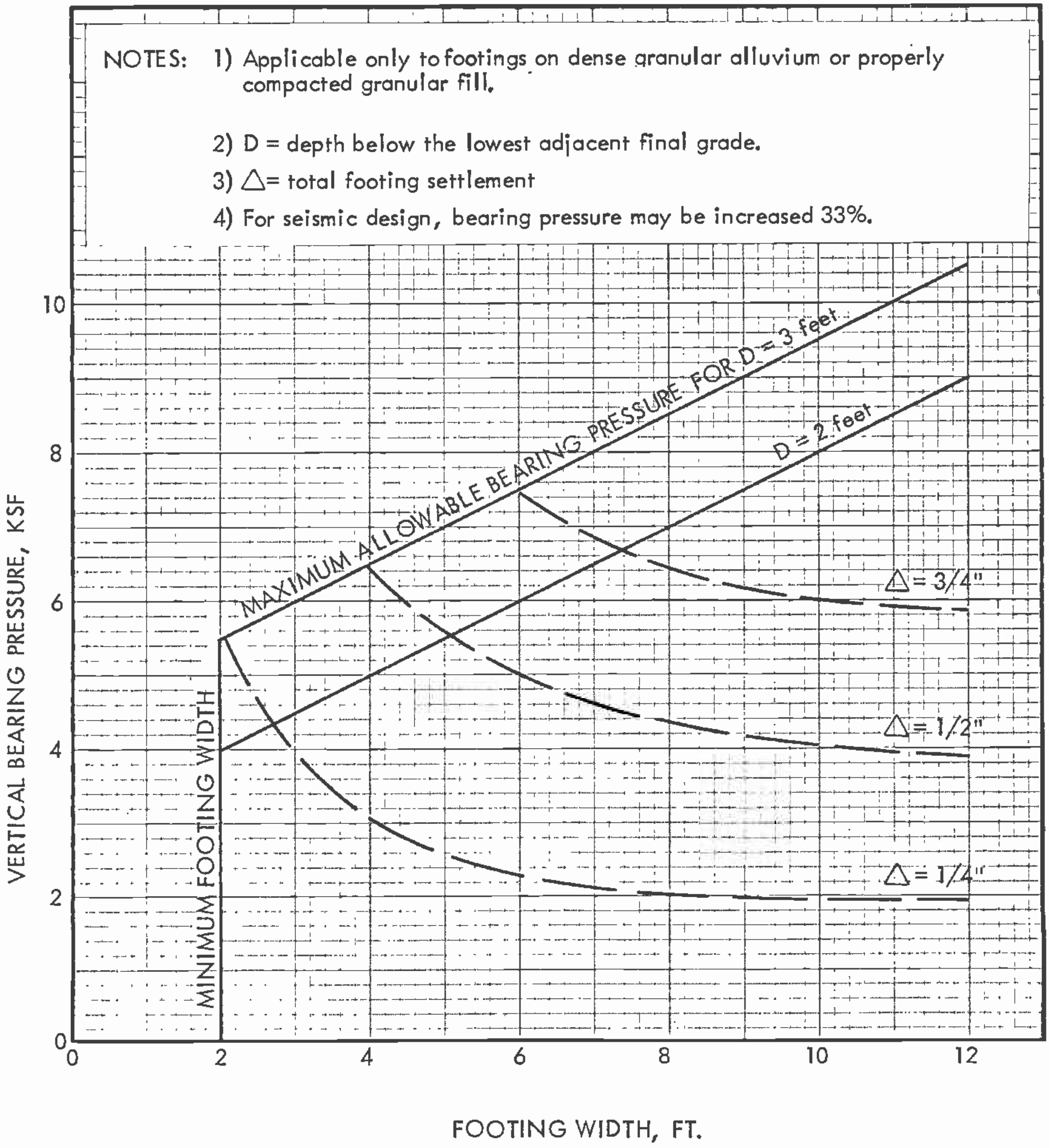
The base of the proposed Station and tail track structures will function as a massive mat foundation. At the proposed foundation level, the mat will be bearing on the gravelly alluvial deposits. We estimate the net mat foundation bearing pressures for the Station will range from about 4500 to 5000 psf. In our opinion, the Station and tail track structures can be adequately supported on a mat foundation bearing on the underlying granular alluvium 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 dense alluvium. Allowable bearing pressures and estimated total settlements of spread footings can be estimated based on Figure 6-9. These relationships are based on analytical procedures and local experience but are generally conservative due to lack of detailed information on structural loadings and site conditions at the surface structure locations.

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 Figure 6-9 are for full dead load and frequently applied live load. For transient loads, including seismic and wind loads, the bearing values can be increased by one-third. 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 Figure 6-9, whichever is larger.

For design, resistance to lateral loads on surface structures can be assumed to be provided by passive earth pressure and friction acting on the foundations. An allowable passive pressure of 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


NOTES: 1) Applicable only to footings on dense granular alluvium or properly compacted granular fill.

2) D = depth below the lowest adjacent final grade.

3) △= total footing settlement

4) For seismic design, bearing pressure may be increased 33%.

ALLOWABLE BEARING & SETTLEMENT FOR SPREAD FOOTING ON GRANULAR SOILS

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Figure No.
6-9

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should be determined using a frictional coefficient of 0.4 with dead load forces.

6.9 STATIC LOADS ON PERMANENT SLAB AND WALLS

6.9.1 Hydrostatic Pressures

Groundwater was not encountered within the borings drilled at the Station site in 1983 (see Table 6-1). It is recommended that for design the maximum groundwater levels be assumed to be below the base of the foundation slab.

6.9.2 Permanent Static Earth Pressures

The permanent static lateral and vertical earth pressures recommended for design are tabulated in Figure 6-10.

Vertical pressures on the roof of the Station and tail track structure should be taken equal to the full weight of the overburden soil plus surcharge.

6.9.3 Surcharge Loads

Lateral surcharge loads from existing buildings not underpinned 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 6-6. Vertical surcharge loads due to surface traffic, railroad, etc., should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.

6.10 PARAMETERS FOR SEISMIC DESIGN

6.10.1 General

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the 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.

6.10.2 Dynamic Material Properties

Values of apparent wave propagation velocities for use in travelling wave analyses have been presented in the May 1983 seismic design criteria report. Other dynamic soil parameters will also be required for input into the various types of analyses recommended in the seismic design criteria report. These include values of dynamic Young's modulus, dynamic should be determined using a frictional coefficient of 0.4 with dead load forces.

6.9 STATIC LOADS ON PERMANENT SLAB AND WALLS

6.9.1 Hydrostatic Pressures

As tabulated in Table 6-1, groundwater was not encountered within the borings drilled at the Station site in 1983. It is recommended that for design the maximum groundwater levels be assumed to be below the base of the foundation slab.

6.9.2 Permanent Static Earth Pressures

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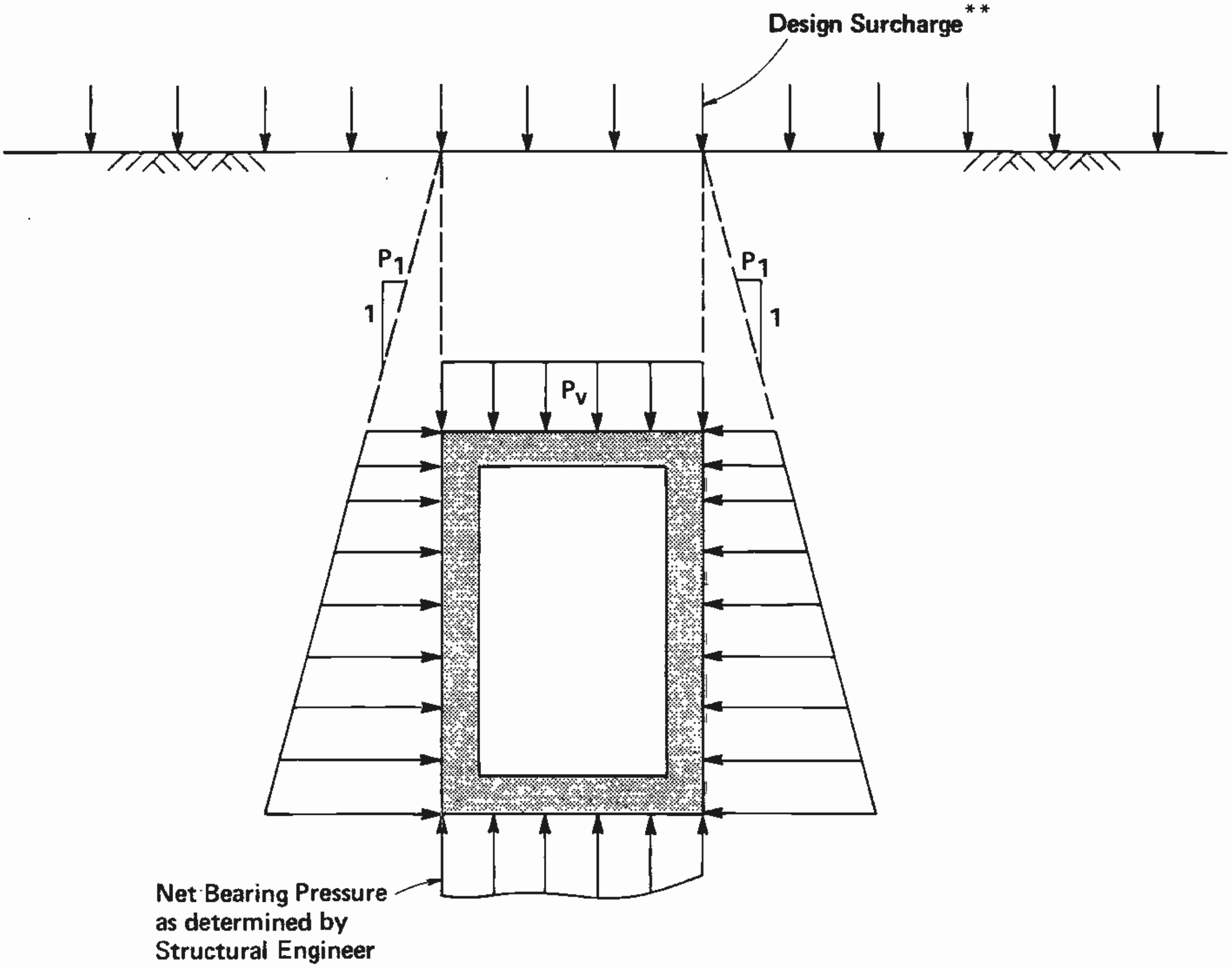

LOADING CONDITION	DESIGN LOAD PARAMETERS, psf	
	P_1	P_v*
End of Construction	31	—
Long Term	50	—
Side Sway	31/50	—

* P_v = Full overburden pressure (depth x total density of soil) + design surcharge

** Design surcharge should include loads imposed by railroad and standing traffic

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LOADS ON PERMANENT WALLS

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Figure No.
6-10

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 geophysical surveys performed in Boring CEG-38, 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 granular alluvium 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 6-11 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 6-12. 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.

6.11 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 are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill are presented in Appendix F. Recommended specifications for compaction of fill are also presented in Appendix F. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. 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.

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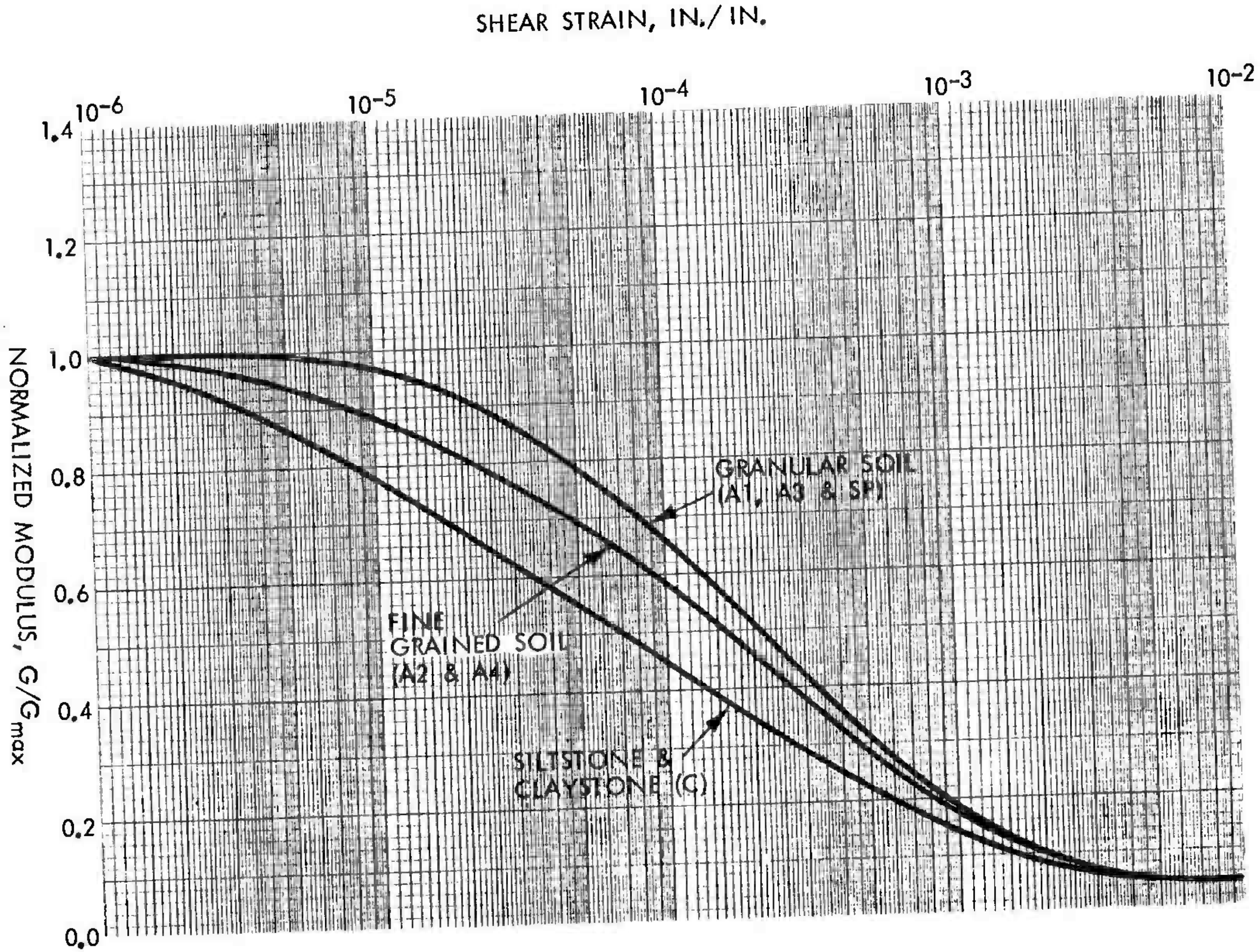

RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
6-11

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RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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83-1140

Figure No.
6-12

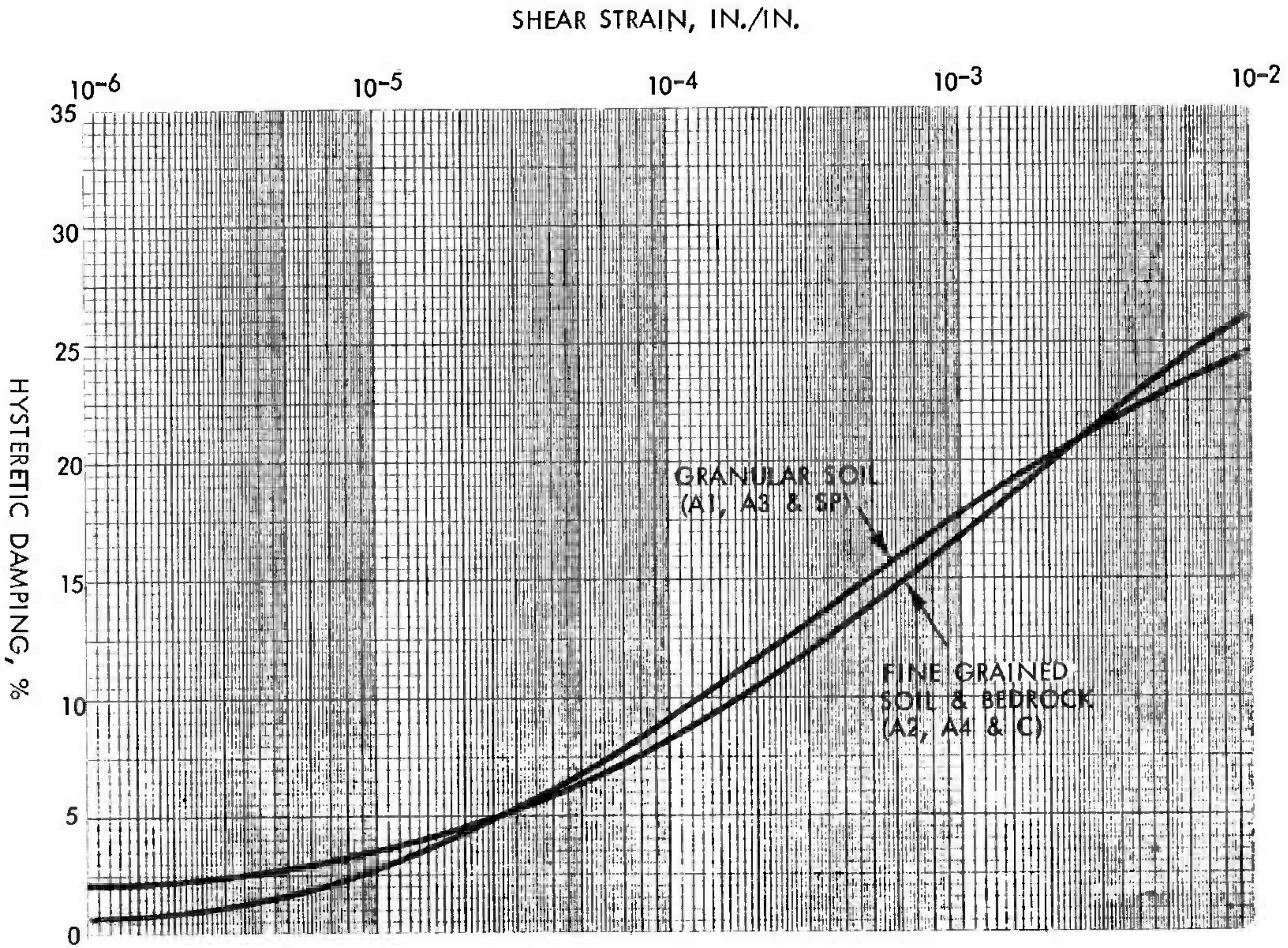


Table 6-2

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR COARSE-GRAINED ALLUVIUM FOR USE IN DESIGN

Property	
Average Compression Wave Velocity, V_p, ft/sec	2,400
Average Shear Wave Velocity, V_s, ft/sec	1,100
Poisson's Ratio	0.35
Young's Modulus, E, psi	100,000
Constrained Modulus, E_c, psi	160,000
Shear Modulus, G_{max}, psi	34,000

Note: Values apply below a depth of 15 feet.

6.12 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- Observe Well Monitoring: The existing groundwater observation wells should be measured several times a year until project construction and more frequently during construction if possible. These data will aid in confirming the observed groundwater levels.

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

- Shoring Design Review: 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.

- Construction Observations: 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.

References

REFERENCES

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

NEWMAN, S.P., 1975, Analysis of Pumping Test Data from Anisotropic Unconfined Aquifers Considering Delayed Gravity Responses: Water Resources Research, Vol. 11, No. 2, p. 329-342.

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

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

PRATER, E.G., 1977, An examination of some theories of earth pressure on shaft linings: Can. Geotechnical Journal, Vol. 14, No. 1, Feb. 1977, p. 91-106.

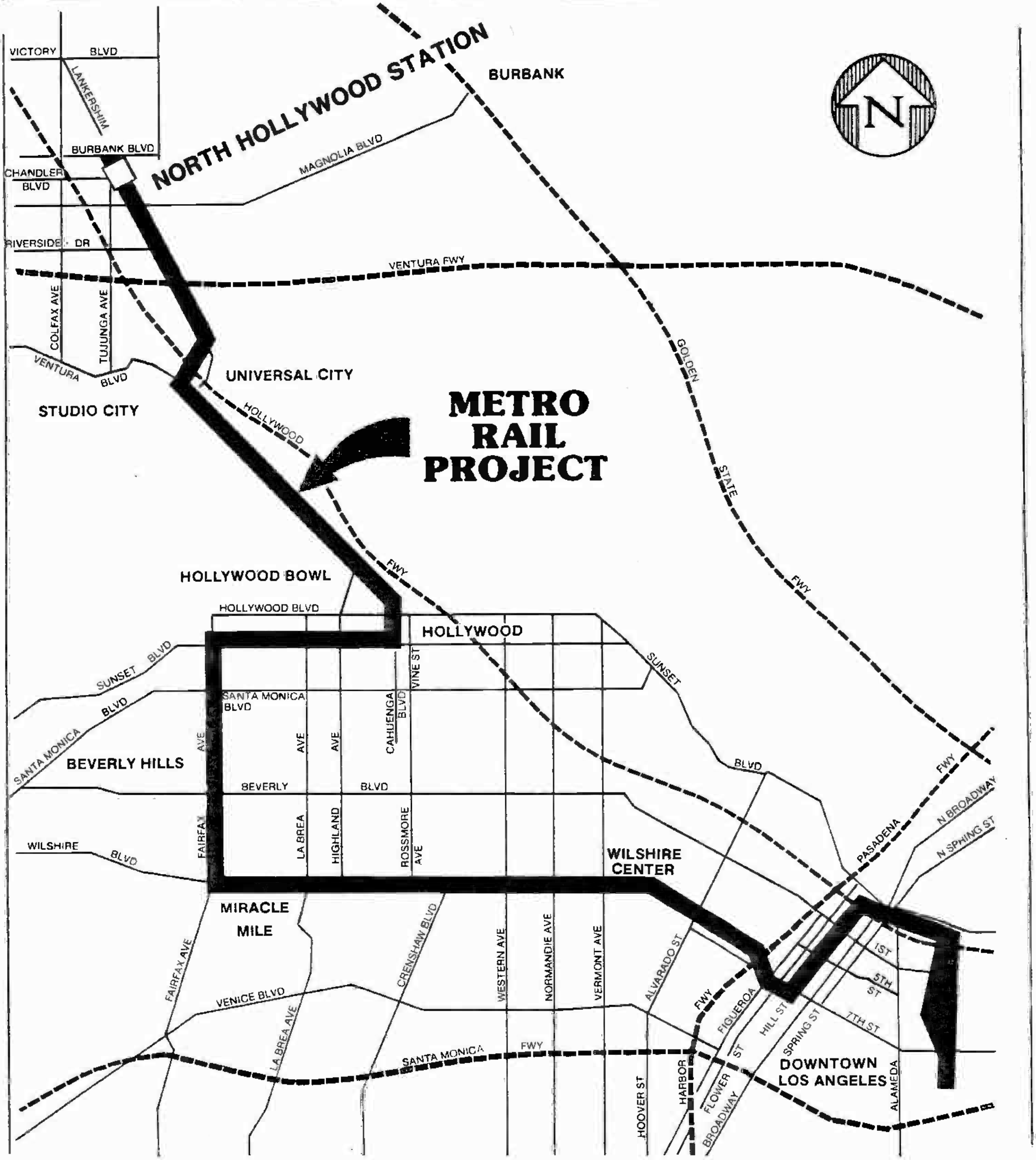
SCHULTZ, M.S., 1981, An empirical investigation into the behavior of diaphragm walls, Masters Thesis, Massachusetts Institute of Technology.

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

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

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

Drawings



VICINITY MAP

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Project No.
83-1140

Drawing No.
1

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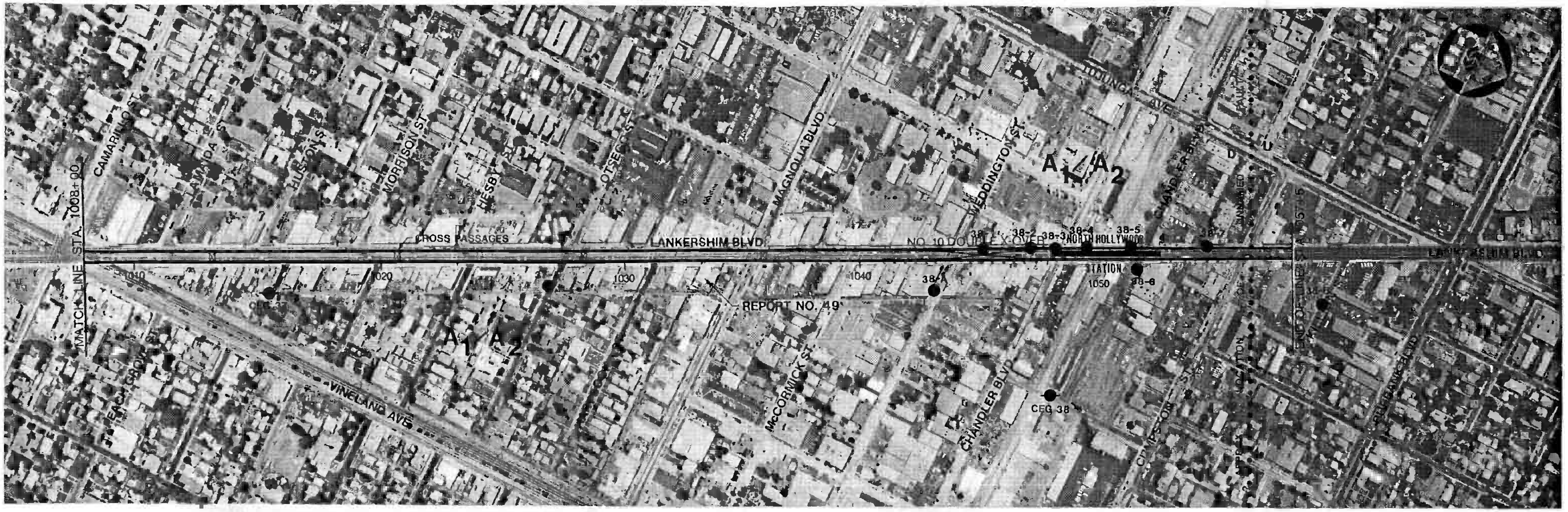
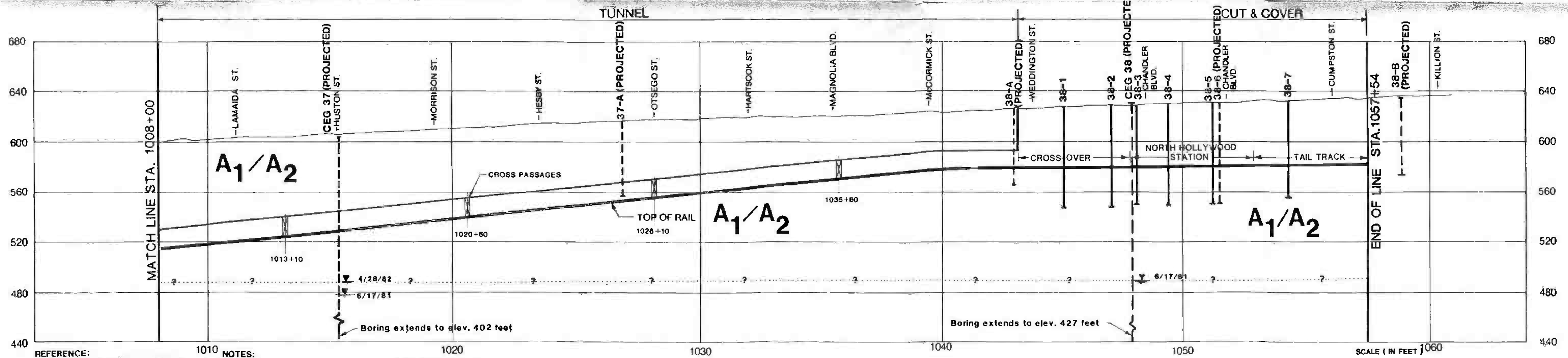




REFERENCE:
MILESTONE 10, SHEET 21 OF 21
ALIGNMENT PLAN AND PROFILE
STATION 1008+00 TO STATION
1057+54 DATED MARCH 1983.

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.

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

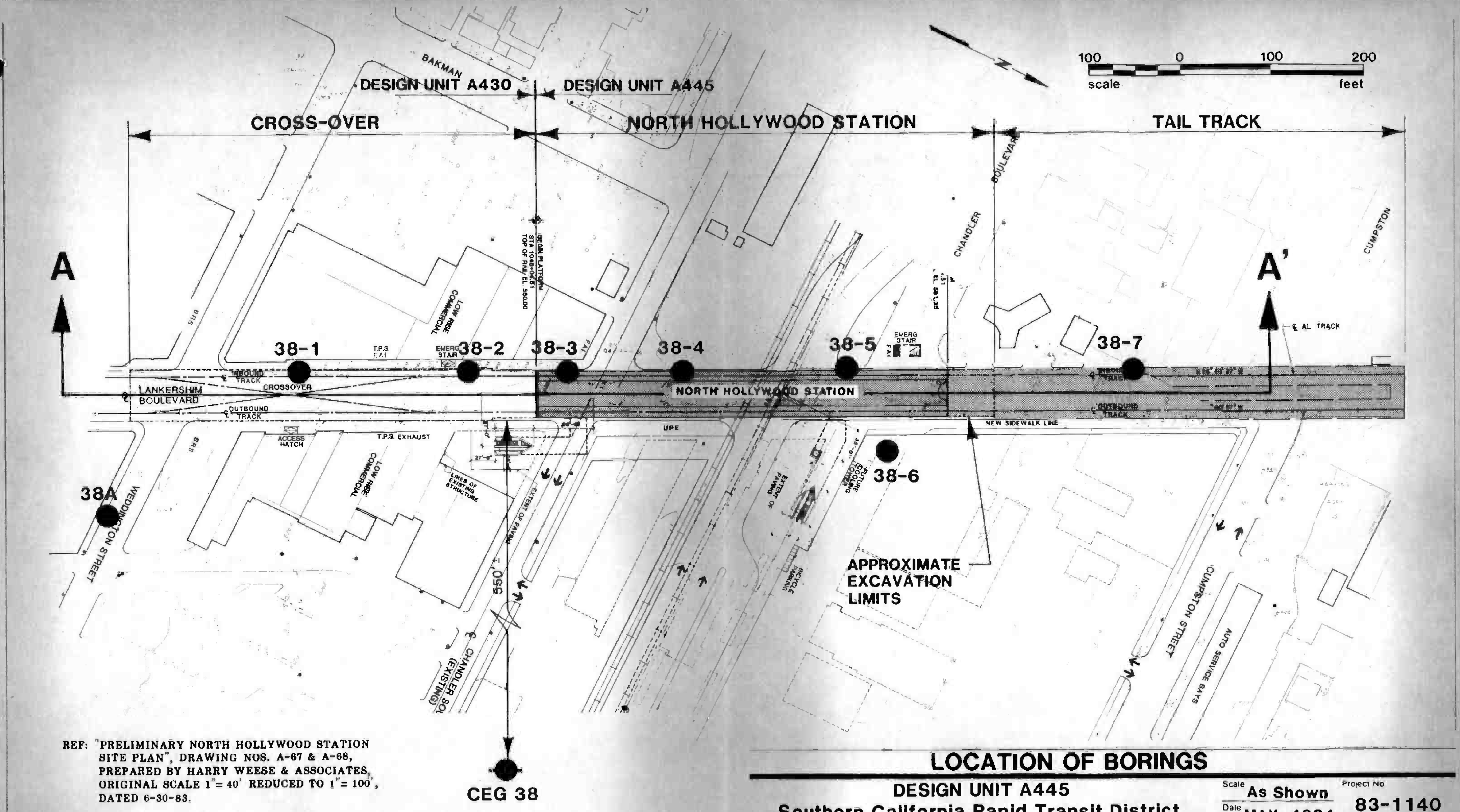
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted
Date 5/31/84

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

DESIGN UNIT A445
LOCATION OF BORINGS
AND GEOLOGIC SECTION

CONTRACT NO. 83-1140
DRAWING NO. 2
REV.
SCALE As Shown
SHEET NO.



REF: "PRELIMINARY NORTH HOLLYWOOD STATION SITE PLAN", DRAWING NOS. A-67 & A-68, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40' REDUCED TO 1" = 100', DATED 6-30-83.

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

LOCATION OF BORINGS

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METRO RAIL PROJECT

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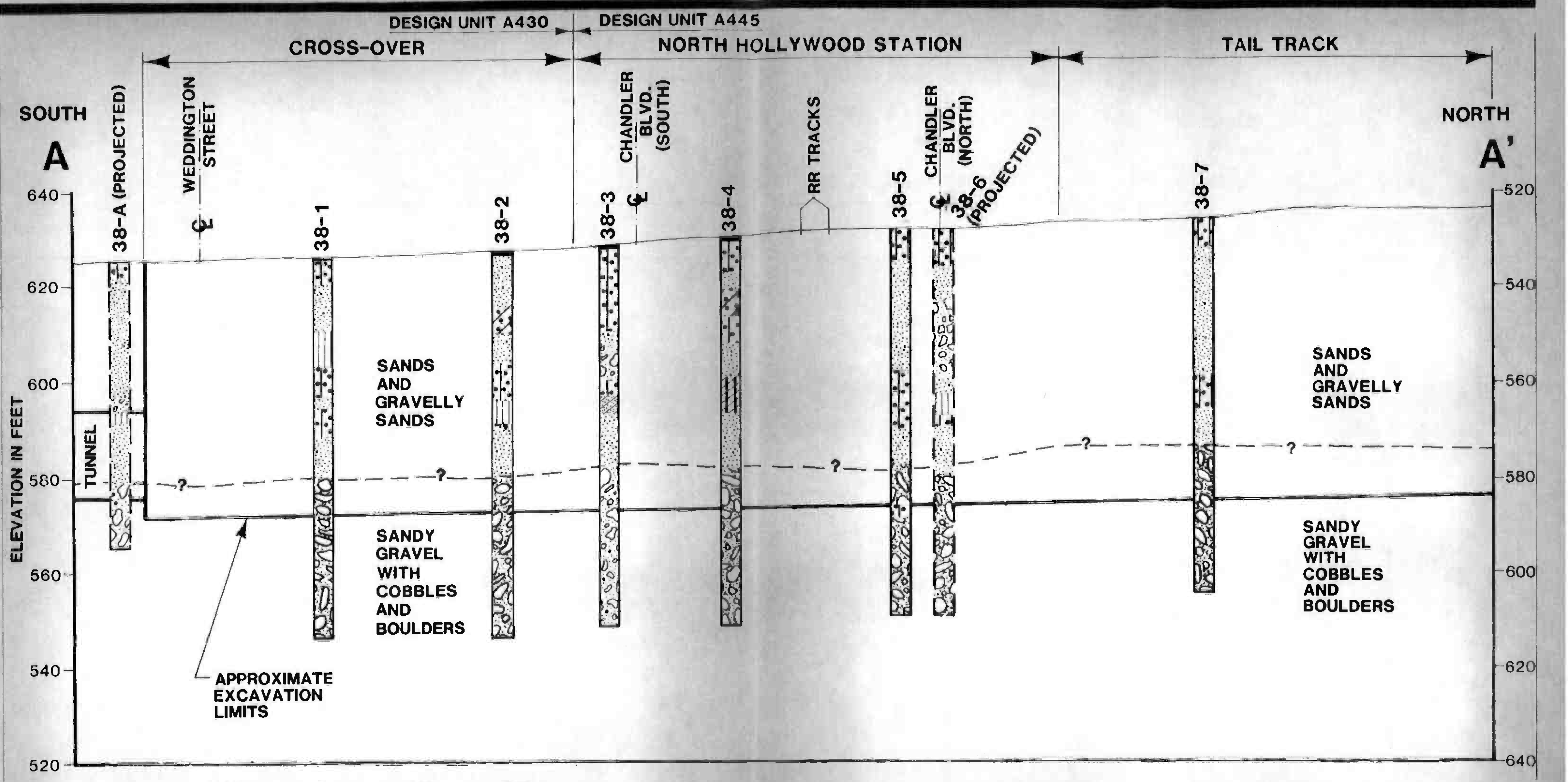
and Applied Sciences

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'

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METRO RAIL PROJECT

Converse Consultants
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Scale As Shown
Date May, 1984
Prepared by MMT
Checked by RG
Approved by JAD
Project No 83-1140
Drawing No 4

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

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

ROCK TUNNELLING
(Terzaghi Rock Condition Numbers apply)*

3 ← Terzaghi Rock Condition Number
← Approximate boundary between Terzaghi numbers

TERTIARY	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).

TERZAGHI 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 A445
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 A445 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 North Hollywood Station site and along the tail track structure. The locations of all boreholes listed in Table A-1 are shown on Drawing No. 2. Boring 38A, 38-1, and 38-2 were drilled within the bounds of the Crossover Structure of Design Unit A430, which is situated to the south of the North Hollywood Station. The logs of these holes are included at the end of this appendix because they contain information that has been used in the interpretation of the subsurface conditions at the Station site and along the tail track structure.

Of the 10 borings that have been drilled at or near the North Hollywood Station site and the tail track structure, 8 are rotary wash type borings and 2 are large-diameter or "man-size" auger holes. One rotary wash boring was drilled as part of the 1981 geotechnical investigation (i.e., CEG-38) and 7 borings were drilled for this investigation during November of 1983. The large-diameter boreholes were drilled in September 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 3 of the borings drilled at or near the Station site (see Table A-1). Groundwater samples were obtained from the rotary-wash boring (i.e., CEG-38) drilled as part of the 1981 geotechnical investigation.

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 within 15 feet of the ground surface and when gravelly materials were encountered. The large-diameter or "man-sized" auger holes were logged by downhole observer(s); however, soil samples were not obtained from these holes.

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.

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

TABLE A-1
BORING LOG SUMMARY
DESIGN UNIT A445

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-38(3)	12/80	RW	628±	201.3	Yes	0.0-195.0	Yes	No	Downhole survey
38A	9/83	LD	624	60.0	No	–	No	No	Minor raveling & caving
38B	9/83	LD	635	60.0	No	–	No	No	Minor caving
38-1	11/83	RW	626	79.8	No	–	No	No	
38-2	11/83	RW	627	80.3	No	–	No	No	
38-3	11/83	RW	628	79.2	No	–	No	No	
38-4	11/83	RW	630	81.0	Yes	0.0-81.0	No	No	
38-5	11/83	RW	631	80.3	No	–	No	No	
38-6	11/83	RW	632	80.2	Yes	0.0-80.2	No	No	
38-7	11/83	RW	633	77.7	No	–	No	No	

NOTES: (1) Types – RW: Rotary wash boring (small diameter).
LD: 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.

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-38 was drilled during the 1981 geotechnical investigation. The soils encountered in this boring were sampled about every 10 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 very dense or gravelly soils were encountered, the Pitcher Barrel sampler was used instead of the Converse ring sampler to obtain relatively undisturbed soil samples for laboratory testing.

Seven rotary wash borings were drilled at the Station site during the month of November of 1983. Borings 38-1 through 38-7 were drilled to depths ranging between about 78 and 81 feet. In general, the soils encountered in the upper 50 feet of 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. Below the depth of about 45 to 50 feet, gravelly materials, which are very difficult to sample, were usually encountered. Attempts were made to sample these materials using the Converse ring and SPT samplers at about 5-foot intervals. The Pitcher Barrel sampler was also sometimes used. In most cases, however, sample recovery was very poor and only a few samples suitable for laboratory testing were obtained.

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 within the first 15 feet of the ground surface at the site tended to fall, pull, or wash 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 Borings 38A and 38B, which were large-diameter or "man-size" boreholes. 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 32-inch bucket.

A.3.3 Drilling Operations

These operations consisted of drilling both auger borings to a prescribed depth of 60 feet. Corrugated metal pipes (sections 20 feet long) with windows cut on 5-foot vertical intervals were used to case the holes. 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 holes.

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.

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+

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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.

Converse Consultants, Inc.
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BORING LOG CEG 38

Proj: DESIGN UNIT A430
Date Drilled 12-15-80
Ground Elev. 628'
Drill Rig FAILING 1500
Logged By Gallinatti
Total Depth 201.3
Hole Diameter 4 7/8"
Hammer Weight & Fall SS 140 lbs @ 30", DR 325 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	SM	0-2.0 SILTY SAND: moderate olive brown; low plasticity fines; fine to coarse grained sand; moist; loose			AD	Started drilling 1:00
2	SP	2.0-34.0 SILTY SAND: pale greyish olive; fine to medium grained sand; loose; dry				Auger to 10', then set 10' of 5" surface casing. Mix mud, sample and begin rotary drilling. Drill with 4 7/8" RTC bit
4						
6						
8						
10			J-1	7 13 12	SS	1.0/1.5 recovery
		11.0 silty sand, occasional 1" gravel; one 0.1' layer of very fine sand; moist				
12		12.0-13.0 gravelly lens			RD	
14		slight increase in content of medium to coarse sand with depth				
16						
18						
20						

Sheet 1 of 9

Project DESIGN UNIT A430 Date Drilled 12-15-80 Hole No. CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	2.0-34.0 SILTY SAND: (continued)	C-1	66	DR	0.8/1.0 recovery
			J-2	19	SS	
22				24		
				38		
					RD	
24						
26						
28						
		29.5-31.0 gravelly lens				
30				29	SS	ss - no recovery due to gravel blocking split-spoon
			J-3	22	SS	
32				22		
				50		
					RD	1.2/1.5 recovery
34	SM	34.0-36.0 SILTY SAND: moderate brown; low plasticity fines; fine grained sand				
36	SP	36.0-49.0 GRAVELLY SAND: light brown; fine to coarse sand; gravel, concentrated in layers, sub-rounded; up to 1" medium dense; moist				
38						
40			C-2	69	DR	drive sample partially disturbed while removing from sampler
			J-4	30	SS	
42		42.0 silty sand with trace gravel		29		
				50		0.6/1.0 recovery
					RD	1.2/1.5 recovery
44						

Project DESIGN UNIT A430 **Date Drilled** 12-15-80 **Hole No.** CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	36.0-49.0 GRAVELLY SAND: (continued)			RD	
46						
48						
	GW/SW	49.0-85.0 SANDY GRAVEL/GRAVELLY SAND: light brown; fine to coarse sand; rounded to sub-rounded ¼" to 3" gravel; medium dense to dense; clasts are mostly granitic				
50				50	SS	0.0/.3 recovery
					RD	no recovery - probably due to gravel/cobble
52		interlayered SAND 0.5' to 2' thick				
						rig chatter
54						
56						
58						
						small recovery due to large gravel in sample
60		60.0 sand	C-3	100÷6"	DR	0.4/0.6 recovery
				50	SS	0.0/0.5 recovery
					RD	no recovery due to gravel jammed in split-spoon
62						
64						
						12-15-80
						12-16-80
66						
68						

Sheet 3 of 9

Project DESIGN UNIT A430 **Date Drilled** 12-15-80 **Hole No.** CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GW/SW	49.0-85.0 SANDY GRAVEL/GRAVELLY SAND: (cont)			RD	
70						
	SM	70.5-72.0 SILTY SAND: dark yellowish brown; low plasticity fines; fine sand; medium dense; moist to wet	J-5	23 31 50	SS	0.8/1.2 recovery pocket pen : 0.5tsf (broke apart) 2-9-81
72	GW-SW (SP)				RD	
74		large cobbles present				
76						
78						
80			C-4	115-4"	DR	poor recovery
					PB	no recovery due to large gravels and cobbles - tube bent
82			J-6	50	SS	0.1/0.3 recovery
					RD	
84						
	CL	85.0-88.0 SANDY CLAY: moderate brown; moderate plasticity fines; fine grained sand; medium dense				
86						
88	SW	88.0-97.5 GRAVELLY SAND: pale yellowish brown fine to coarse angular; sub-angular gravel; medium dense; material seems to be of a granitic source				
90			J-7	50	SS	0.1/0.3 recovery
					RD	
92						

Project DESIGN UNIT A430 Date Drilled 12-16-80 Hole No. CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SW	88.0-97.5 GRAVELLY SAND: (continued)			RD	
94						
96						
98	ML	97.5-99.0 SANDY SILT: moderate brown; low plasticity fines; fine grained sand; medium dense/stiff				
	SP (GP)	99.0-103.0 SAND with interbedded SANDY GRAVEL: moderate yellowish brown; fine to coarse sand; interbedded with sand and gravel				poor recovery due to gravels
100			C-5	100-6"	DR	0.3/0.5 recovery
			J-8	50	SS	0.2/0.3 recovery
					RD	
102						
	SM (ML)	103.0-112.0 INTERBEDDED SILTY SAND and CLAYEY SILT: moderate brown; silty sand; very fine to fine sand; dense to medium dense; clayey silt; low to moderate plasticity fines; stiff				
104						
106			S-1		PB	1.3/2.0 recovery
108			J-9	29 50	SS	0.8/0.8 recovery
					RD	
110						
112	SW/ (SM)	112.0-140.5 GRAVELLY SAND: light brown; trace low plasticity fines; fine to medium grained angular sand; subrounded gravel; dense; some silt lenses				
114						
116						

Project DESIGN UNIT A430 **Date Drilled** 12-16-80 **Hole No.** CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	SW	112.0-140.5 GRAVELLY SAND: (continued)			RD	
118			S-2		PB	1.2/2.0 recovery bottom 0.9' fell out of tube while still in the hole
120		decrease content of gravels	J-10	50	SS	0.1/0.3 recovery
					RD	
122						
124	ML	124.0-125.0 SANDY SILT:				
	SW					
126		becomes fine to medium with depth				
128						
130				50	SS	0.0/0.3 recovery
					RD	no recovery
						stop to reseal hole at surface
132						
134						
136						
138						
140						

Sheet 6 **of** 9

Project DESIGN UNIT A430 **Date Drilled** 12-16-80 **Hole No.** CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	SW	112.0-140.5 GRAVELLY SAND: (continued)			RD	
		140.5-168.0 SILTY SAND: moderate brown; low plasticity fines; very fine to fine grained sand; dense; wet;				
142			S-3		PB	2.1/2.2 recovery
			J-11	50	SS	0.2/0.3 recovery
144					RD	
146						
148						
150						
			J-12	50	SS	0.4/0.4 recovery
152					RD	
154						
156						
158						
160			S-4		PB	1.9/2.5 recovery
162						0.3/0.4 recovery 12-16-80
			J-13	50	SS	12-17-80
164					RD	

Sheet 7 **of** 9

Project DESIGN UNIT A350 Date Drilled 12-16-80 Hole No. CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	SM	140.5-168.0 SILTY SAND: (continued)			RD	
166						
168	CL	168.0-173.0 SILTY CLAY: light olive brown; moderate plasticity fines; stiff; damp; MnO staining				
170			J-14	21 48 50	SS	1.1/1.3 recovery
172					RD	
	SM	173.0-187.0 SILTY SAND: yellowish moderate brown; low plasticity fines; very fine to fine grained sand; dense; moist; abundant fine grained pyrite				
174						
176						
178						
180		180.0 clayey fine sand	S-5		PB	1.3/1.5 recovery
			J-15	50	SS	0.1/0.3 recovery
182		increase in grain size with depth			RD	
184						
186		187.0-201.3 SAND: moderate yellowish brown; fine to medium grained sand; dense; moist; some pyrite and mica				
188	SP					

Sheet 8 of 9

Project DESIGN UNIT A350 **Date Drilled** 12-17-80 **Hole No.** CEG 38

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188	SP	187.0-201.3 <u>SAND:</u> (continued)			RD	
190			J-16	50	SS	0.2/0.3 recovery
					RD	pocket pen : 0.5tsf (broke apart) 2-9-81
192						
194	(SM)	194-200' some interbedded silty sand lenses				
196						
198						
200			S-6		PB	1.0/1.0 recovery
			J-17	50	SS	0.2/0.3 recovery
202		B.H. 201.3 Terminate Hole				hole completed 12-17-80 e-log 12-17-80 down-hole survey on morning of 12-18-80, flush-out hole and install perforated casing water sampled 2-25-81 peizometer: from 200' to surface, perforated from 180' to 195', from 120' to 140', and 60' to 100'
204						
206						
208						
210						
212						

Sheet 9 of 9

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

Converse Consultants, Inc.
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Geo/Resource Consultants

BORING LOG 38A

Proj: DESIGN UNIT A430
Date Drilled 9-28-83
Ground Elev. 624
Drill Rig BUCKET
Logged By J. Stellar
Total Depth 60'
Hole Diameter 32"
Hammer Weight & Fall

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 CONCRETE (6")				
	SM	0.5-4.0 FILL SILTY SAND: medium brown; medium dense; moist				
2						hole stands well in general
4		ALLUVIUM				
	SP	4.0-29.0 SAND: very light yellow; medium grained; clean; medium dense; moist				sand cuttings falling from bucket between 4' - 30'
6						
		7.0-trace gravel to 1"				
8		8.0-9.0 gravelly sand with gravel to 3.0 inches				
						very minor ravelling 10'-14'
10						
12						
14		sand grades coarse grained poorly graded sand layers 2"to 6" thick				
16						
18						
	(SM)	few cobbles to 8." lenses of silty sand				
20						

Sheet 1 of 3

Project DESIGN UNIT 430 Date Drilled 9-28-93 Hole No. 38-A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	4.0-29.0 SAND: (continued) slightly moist to moist				
22		contains trace coarse gravel; small cobbles to 5"				
24						
26						
28						
	SP	29.0-32.0 GRAVELLY SAND: light yellow brown; clean coarse grained sand; granitic coarse gravel 3"; medium dense; moist				
30						
32	ML (SM)	32.0-34.0 SILT: dark brown; minor fine sand lenses;stiff; moist to wet				
34	SP (SM) (ML)	34.0-47.0 SAND: light brown lenses of silt and silty sand; medium dense; moist				
36						
38						
40						
42		sand is clean and coarse grained				
44						

Sheet 2 of 3

Project DESIGN UNIT 430 Date Drilled 9-28-83 Hole No. 38-A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44 46	SP (SM) (ML)	34.0-47.0 SAND: (continued) trace gravel to 2"				
48 50 52 54 56 58	SP (ML)	47.0-60.0 GRAVELLY SAND: coarse clean sand; granitic gravel and cobbles to 4" 48.0-silt lens; very moist 49.0-cobbles to 10" 50.0-60.0 cobbles, small boulders to 12"; Fe Oxide staining on cobbles				caving below 50" slowing drill progress
60 62 64 66 68		B.H. 60' Terminate boring				Hole completed 9-28-83. Very minor ravelling (4 inches) from 4 to 31 feet. Caving (1 to 1.5 feet) from 50 to 60 feet. Downhole Observers HAS JRS B.I. MADUKE

Sheet 3 of 3

BORING LOG 38-B

Proj: DESIGN UNIT A445
Date Drilled 9-27-83
Ground Elev. 635
Drill Rig BUCKET
Logged By J.R. Stellar
Total Depth 60.0
Hole Diameter 32"
Hammer Weight & Fall

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT				
	SM (SP)	0.2-4.0 ALLUVIUM SILTY SAND: alternating and merging with fine sand; medium dense; moist				Fast easy drilling from 0 to 44 feet 0'-50' stands well (no caving)
2						
4	SP	4.0-27.0 SAND: very light yellow; medium dense; moist sand becomes medium to coarse grained				
6						
		7.0-trace pea-sized gravel				
8						
10		10.0-trace gravel to 1"				
		11.0-few cobbles to 8" (granitic)				
12						
14						
16						
18						
20		20.0-trace gravel to 1"				

Project DESIGN UNIT A445 **Date Drilled** 9-27-83 **Hole No.** 38-B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	4.0-27.0 SAND (continued) 21.0-22.0 trace gravel to 2"				
22						
24						
26						
	ML	27.0-30.0 SILT: grayish brown, firm, micacious, moist				
28						
30	SM (ML)	30.0-34.0-SILTY SAND: very fine with trace silt; alternating lenses of sandy silt; slightly moist to moist				
32						
34	ML (SM)	34.0-39.0 SANDY SILT: alternating and merging with lenses of silty sand; slightly moist to moist				
36						
38						
	SP	39.0-42.0 SAND: clean sand; trace gravel to 2"; dense; slightly moist to moist				
40						
42	SP	42.0-46.0-GRAVELLY SAND: coarse grained sand; trace gravel to 3"; slightly moist to moist				6" belling ouside of casing
44						

Project DESIGN UNIT A445 **Date Drilled** 9-27-83 **Hole No.** 38-B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	42.0-46.0-GRAVELLY SAND: (continued)				drilling becomes harder at 44'-60'
46	ML	46.0-49.5-SANDY SILT: with gravel to 2"; very moist				
48		48.0-49.0-becomes wet				48'-49' free water in cuttings; no seepage in hole
50	GM	49.5-60.0-SILTY GRAVEL: with silt; gravel to 3", slightly moist to moist; some cobbly boulders to 10" Fe Oxide stain on cobbles/ boulders				minor belling
52		52.0-56.0-contains less silt; more coarse sand				
54						
56		56.0-60.0-decrease in gravel size to 3"				
58						
60		B.H. 60.0' Terminate boring				Hole completed 9-27-83 Minor caving 50.0-60.0' Set casing to 59.0' No groundwater encountered No seepage Downhole observers HAS JRS
62						
64						
66						
68						

Sheet 3 **of** 3

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

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

BORING LOG 38-1

Proj: DESIGN UNIT A430 **Date Drilled** 11-10-83 **Ground Elev.** 626'

Drill Rig FAILING 1500 **Logged By** L. Schoeberlein **Total Depth** 79.8'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.9 CONCRETE			GB	start drilling 7:30
	GP	0.9-1.2 BASEROCK				
2	SM	1.2-5.8 ALLUVIUM SILTY SAND: dark yellowish brown; fine sand; with non-plastic fines; loose; dry to moist	SH 1		SH	recovery 2.5/2.5
4			J-1	1 3 5	SS	recovery 1.0/1.5
6	SP	5.8-15.5 SAND: dark yellowish brown; to salt and pepper; fine sand; trace silt; dense; dry			RD	set tub and cased to 5', mixed mud
8			C-1	13 22	DR	recovery 0.9/1.0
10					RD	
12			PB 1		PB	recovery 1.6/2.5
14		14.5 silty sand	J-2	12 24 25	SS	recovery 1.5/1.5
16	ML	15.5-23.0 SILT: dark yellowish brown; non-plastic fines; very fine sand; hard dry			RD	
18			C-2	25 49	DR	recovery 0.9/1.0
20		19.5 sandy silt/silty sand; very dense to hard			RD	

Sheet 1 **of** 4

Project DESIGN UNIT A 430 **Date Drilled** 11-10-83 **Hole No.** 38-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	ML	15.5-23.0 SILT: (continued)			RD	
22			PB 2		PB	recovery 1.0/2.5 rig chatter
	SM/ML	23.0-29.4 SILTY SAND/SANDY SILT INTERBEDDED: dark yellowish brown; variable percentages of sand and silt; occasional gravelly zones; very dense to hard; moist				
24			J-3	13 25 43	SS	recovery 1.5/1.5
26					RD	
	SM	grades to primarily silty sand				
28			C-3	31 72	DR	recovery 0.8/1.0
	SP	29.4-31.5 SAND: salt and pepper; fine sand; trace silt; very dense; dry to moist			RD	
30						
	SM/ML	31.5-37.5 SILTY SAND/SANDY SILT INTERBEDDED: dark yellowish brown; variable percentages of fine sand and silt; dense ; very stiff; moist				
32			PB 3		PB	recovery 2.0/2.2
34		34.2 sandy silt	J-4	8 14 18	SS	recovery 1.5/1.5
36					RD	
	SP	37.5-45.5 SAND: salt and pepper; fine to medium sand; trace silt; very dense; moist; granitic origin; occasional coarse sand and gravel lenses				
38			C-4	97 82-3"	DR	recovery 0.5/1.0 5 rings
40					RD	slight chatter
42		43.5 gravelly sand	PB 4		PB	recovery 1.5/1.5 recovery 0.4/0.5
44			J-5	62	SS	

Project DESIGN UNIT A 430 **Date Drilled** 11-10-83 **Hole No.** 38-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	37.5-45.5 SAND: (continued)			RD	
46	SP/ GW	45.5-79.8 SANDY GRAVEL/GRAVELLY SAND INTERBEDDED: salt and pepper; variable percentages of sands and gravels; very dense; moist; granitic origin; rounded; occasional sand lenses				intense chatter smoother mixed mud
48			C-5	150-5"	DR	recovery 0.4/0.4 disturbed
					RD	chatter
50						
52						
			C-6	200- 4.5'	DR RD	recovery 0.4/0.4 all from shoe
54		becomes sandier				smooth drilling
56						chatter
58			C-7	200 4"	DR	recovery 0.3/0.3
60		cobbles				intense chatter
62		62.9 gravelly sand	C-8	200 5"	DR RD	recovery 0.4/0.4 2 good rings rest from shoe
64						mild chatter
66		coarse gravel				moderate chatter
68						

Sheet 3 **of** 4

Project DESIGN UNIT A430 **Date Drilled** 11-10-83 **Hole No.** 38 -1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP/ GW	45.5-79.8 SANDY GRAVEL/GRAVELLY SAND INTER BEDDED: (continued) contains some weathered cobbles			RD	recovery 0.3/0.3
			C-9	200	DR	all from shoe
				3.5"	RD	
70						
72		cobbles				intense chatter
74	(SM) (ML)	thin silty sand, sandy silt lenses				
			C-10	200	DR	recovery 0.5/0.5
76				5.5"	RD	rock in top, 2 rings good; rest from shoe
						moderate chatter
78						
			C-11	200 4"	DR	1 good ring, rest from shoe recovery 0.3/0.3
80		B.H. 79.8" Terminate Hole				Complete hole 11-10-83
82						Tremied grout to surface
84						
86						
88						
90						
92						

Sheet 4 of 4

Converse Consultants, Inc.
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BORING LOG 38-2

Proj: DESIGN UNIT A430 **Date Drilled** 11-3-83 **Ground Elev.** 627
Drill Rig FAILING 1300 **Logged By** L. Schoeberlein **Total Depth**
Hole Diameter 4 7/8 **Hammer Weight & Fall** SS: 140 lbs. @30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.6 CONCRETE:			GB	start drilling 7:15
	GP	0.6-1.0 BASEROCK:				
	SP	1.0-11.5 ALLUVIUM SAND: dark yellowish brown fine sand; trace non-plastic silt; loose; dry to moist				
2			SH-1		SH	recovery 2.5/2.5
4		density increases to medium dense, dry	J-1	5 9 10	SS	recovery 1.4/1.5 set up tub and cased to 5' mixed mud
6					RD	
8			C-1	9 22	DR	recovery 0.8/1.0
					RD	
10						
12	SC	11.5-17.5 CLAYEY SAND: dark yellowish brown, fine sand, dense; moist	PB-1		PB	recovery 2.3/2.5
14	(SM) (ML)	14.0-gravelly sand lens 14.5-sandy silt/silty sand with trace clay content of fines decrease contains wood fragments	J-2	11 19 22	SS	recovery 1.5/1.5
16					RD	
18	SP (GM) (GP)	17.5-23.0 SAND: salt and pepper coloration, fine sand; trace silt; very dense; moist, occasional coarse sand or gravel lenses	C-2	27 50-3"	DR	recovery 0.1/0.7 rig chatter
					RD	
20						

Sheet 1 **of** 4

Project DESIGN UNIT A430 **Date Drilled** 11-3-83 **Hole No.** 38-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	17.5-23.0 SAND: (continued)			RD	
22			PB-2		PB	recovery 1.8/2.5
	SM SP	23.0-31.0 SAND/SILTY SAND: dark yellowish brown very fine to fine sand; low non-plastic fines; very dense; moist; contains occasional gravel				
24			J-3	14 17 34	SS	recovery 1.1/1.5
26					RD	
28				44	DR	recovery 0.8/1.0
	(ML)	29.0-sandy silt lens	C-3	56		
					RD	
30						
	ML	31.0-35.8 SILT: dark yellowish brown low plastic fines; trace fine sand; hard; moist				
32			PB-3		PB	recovery 2..5/2.5
		34.0-clayey sand lens				
34		35.0 sandy silt	J-4	11 25 44	SS	recovery 0.8/1.5
		35.8-37.0 SILTY SAND: salt and pepper; fine to medium sand; trace non-plastic fines; very dense; moist				
36	SM				RD	
	SP	37.0-46.5 SAND: salt and pepper; fine to medium sand; trace silt; very dense; moist; contains occasional gravelly sand lenses; granitic origin				
38			C-4	79 100-	DR 5"	recovery 0.8/1.0
					RD	
40						
42			PB-4		PB	recovery 1.5/2.3
44		44.5-sand/silty sand				

Sheet 2 of 4

Project DESIGN UNIT A430 **Date Drilled** 11-3-83 **Hole No.** 38-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	37.0-46.5 SAND: (continued)			PB	
		6" gravelly lens	J-5	50- 4.5"	SS RD	recovery 0.3/0.4 rig chatter
46						
	GW	46.5-80.3- SANDY GRAVEL: variably colored, granitic/metamorphic origin; fine to coarse gravel; fine to coarse sand; very dense; moist				
48				145- 4.5"	DR RD	fell out recovery 0.0/0.4
50						
52				110 100-2"	DR RD	recovery 0.5/0.7 disturbed but representative
			C-5			
54						
56						falling in on bit from above
58						mixed mud
		sand lens	C-6	200	DR RD	recovery 0.2/0.5
60						
62						
64				69	DR	recovery 0.4/0.5
		65.0-sand lens	C-7	195	RD	1/2 disturbed 1/2 in rings ok
66						
68						

Sheet 3 **of** 4

Project DESIGN UNIT A430 **Date Drilled** 11-3-83 **Hole No.** 38-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GW	46.5-80.3 SANDY GRAVEL: (continued)			RD	recovery 0.0/0.5
				50	SS	
					RD	
70						
72						mixed mud
		gravelly sand lens	C-8	125-2"	DR	recovery 0.2/0.2 disturbed but representative
74					RD	
		gravelly sand lens				less chatter
76						
			C-9	200-3.5"	DR	
					RD	recovery 0.3/0.3 disturbed but representative
78						
						recovery 0.3/0.3 disturbed but representative
80			C-10	200-4"	DR	
		B.H. 80.3 Terminate Boring, tremied grout to surface				Complete drilling 11/3/83
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

Converse Consultants, Inc.
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BORING LOG 38-3

Proj: DESIGN UNIT A430
Date Drilled 11/4/83
Ground Elev. 628
Drill Rig Pitcher
Logged By L. Schoeberlein
Total Depth 79.2'
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	conc	0.0-0.8 CONCRETE			GB	start drilling 7:15
	GP	0.8-1.0 BASEROCK				
	SM	ALLUVIUM 1.0-17.5 SILTY SAND: dark yellowish brown, fine sand, non-plastic fines; loose; dry to moist	SH-1		SH	recovery 2.2/2.5
2						
4		decrease silt content	J-1	6 6 3	SS	recovery 1.0/1.5
6					RD	set tub and cased to 5', mixed mud
						down 25 min., joint on kelly hose breaks
8			C-1	4 6	DR	recovery 0.4/1.0
					RD	
10						
12					PB	recovery 0/2.5
14			PB-1		PB	recovery 1.3/2.5
16						
			J-2	14 25 32		recovery 1.2/1.5
18	SP	17.5-21.0 SAND: salt and pepper and yellowish brown; fine sand; trace silt; occasional gravel; dense; moist				
					RD	minor chatter
20		19.5-slight increase in gravel				

Sheet 1 **of** 4

Project DESIGN UNIT A430 **Date Drilled** 11/4/83 **Hole No.** 38-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	17.5-21.0 SAND: (continued)				
	SP SW	21.0-28.0 GRAVELLY SAND: salt and pepper; granitic origin; fine to medium sand; fine to coarse gravel; very dense; moist; gravel occurs in lenses to 1' thick with sand interbeds; sand-subangular; gravel-subanggular to subround; some metamorphic gravels				intense chatter
22			PB-2		PB	recovery 1.5/2.5
24			J-3	16 39 50-5"	SS	recovery 0.6/1.0
26					RD	intense chatter
28				46	DR	recovery 0.5/1.0
	SM (SP)	28.0-31.0 SILTY SAND: dark yellowish brown; fine to very fine sand; non-plastic fines; very dense; moist; occasional sand and gravelly sand lenses	C-2	53		
					RD	
30						rig chatter
	CL	31.0-34.8 SANDY CLAY: dark yellowish brown; moderately plastic fines; fine sand; hard; moist	PB-3		PB	
32						recovery 2.3/2.5
		33.5-sandy clay/clayey sand	J-4	9	SS	
34		33.8-silty sand		23 43		recovery 1.5/1.5
	SP (GW)	34-8-46.0 SAND: salt and pepper; fine sand; silt; very dense; moist; occasional gravel lenses			RD	
36						
						slight chatter
38				66	DR	recovery 0.8/1.0
			C-3	100-5.5		
					RD	
40						slight chatter
	SW	becoming well graded, fine to coarse sand, occasional gravel				
42			PB-4		PB	recovery 1.9/2.5
44						

Sheet 2 **of** 4

Project DESIGN UNIT A430 Date Drilled 11/4/83 Hole No. 38-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW	34.8-46.0 SAND: (continued)			PB	
			J-5	50-4"	SS	recovery 0.3/0.3
					RD	
46	SP/GW	46.0-79.2 GRAVELLY SAND/SANDY GRAVEL: salt and pepper colored; granitic; metamorphic origin; percentages variable of sand and gravel; very dense; moist; gravel content increases with depth				mixed mud, attempted sample, hole caving from 25', redrilled to 49.5', sampled again
48						
50		sand lens	C-4	74 70-5"	DR RD	recovery 0.3/0.5 1 good ring, remainder disturbed
52		gravel & cobbles 6" cobble				intense chatter
54		sandy gravel	C-5	135-5"	DR	recovery 0.2/0.4
					RD	
56		1' cobble/boulder				intense chatter
58				9 16 34	SS	recovery 0/1.0 sluff rock stuck in shoe, blows not valid
					RD	
60						
62						intense chatter
						mixed mud
			C-6	175-4"	DR	drove on rock
64					RD	recovery 0.3/0.3
66						
68						

Sheet 3 of 4

Project DESIGN UNIT A430 Date Drilled 11-4-83 Hole No. 38-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP GW	46.0-79.2 GRAVELLY SAND/SANDY GRAVEL: (continued)			RD	intense chatter
		iron stained	C-7	145 55-1"	DR	recovery 0.5/0.6 2 rings good
70					RD	remainder disturbed
72						
74						
			C-8	180-4.5"	DR	recovery 0.3/0.4
76					RD	disturbed but representative
78						
				175 3"	DR	recovery 0.0/0.2
80		B.H. 79.2' Terminated hole; tremied grout to surface				completed drilling 11/4/83
82						
84						
86						
88						
90						
92						

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 38-4

Proj: DESIGN UNIT A445 **Date Drilled** 11-7-83 **Ground Elev.** 630

Drill Rig FAILING 1500 **Logged By** L. Schoeberlein **Total Depth** 81.0

Hole Diameter 4 7/8" **Hammer Weight & Fall** 130 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.7	CONCRETE:			PB	Start drilling 8:00
	GP	0.7-1.0	BASEROCK:				
	SM	1.0-7.0	ALLUVIUM SILTY SAND: dark yellowish brown fine sand; non-plastic fines; loose; moist			SH	
2				SH-1			recovery 2.5/2.5
4			decrease silt content; thin caliche zone	J-1	3 3 4	SS	recovery 1.5/1.5
6						RD	set tub and cased to 5'
	SP	7.0-10.5	SAND: light yellowish brown; trace non-plastic silt; medium dense; moist				
8				C-1	5 9	DR	recovery 0.8/1.0
						RD	mixed mud
10		10.5-16.5	CLAYEY SAND: dark yellowish brown fine sand; moderate plastic fines; medium dense to dense; moist				slow drilling
	SC						
12						PB	recovery 0.0/2.5 slid out
14	(SM) (SP)		sand lenses				
				PB-1		PB	recovery 1.0/2.5
16							
	SM	16.5-22.0	SILTY SAND: dark yellowish brown fine sand; non-plastic fines; dense; moist				
18				J-2	13 20 35	SS	recovery 0.7/1.5
						RD	
20							

Sheet 1 of 4

Project DESIGN UNIT A445 **Date Drilled** 11-2-83 **Hole No.** 38-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	16.5-22.0 SILTY SAND: (continued)			RD	
						rig chatter
		gravelly lens 1" thick				
22	SP	22.0-29.0 SAND: salt and pepper; fine to medium sand; trace silt; very dense; dry to moist	PB-2		PB	recovery 1.5/2.5
24						
			J-3	9 23 47	SS	recovery 0.0/1.5
26					RD	
		gravelly lens 1' thick granitic origin				
28			C-2	42 48	DR	recovery 0.7/1.0
	ML	29.0-36.5 CLAYEY SILT: dark yellowish brown low to moderately plastic fines; stiff; moist			RD	
30						
32			PB-3		PB	recovery 1.8/2.5
34						
		34.5-clayey silt/clayey sand	J-4	4 12 17	SS	recovery 1.1/1.5
36					RD	
	SP	36.5-48.0 SAND: salt and pepper; fine to medium sand; trace non-plastic silts; very dense; moist				
38			C-3	42 50-3"	DR	recovery 0.5/0.7
					RD	
40						
42			PB-4		PB	recovery 2.0/2.5
	SW	grading coarser with some fine gravel				
44						

Sheet 2 **of** 4

Project DESIGN UNIT A445 Date Drilled 11-2-83 Hole No. 38-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW	36.5-48.0 SAND: (continued)			PB	
			J-5	62	SS	recovery 0.5/0.5
					RD	
46						
48	GW	48.0-81.0 SANDY GRAVEL: salt and pepper; fine to coarse gravel;fine to coarse sand; occasional boulders and cobbles; very dense; moist; some sand lenses		50-3"	DR	
					RD	recovery 0.0/0.2 intense chatter
50	(SW)	48.5-50.0-boulder; granitic origin				
52						drilled on, too gravelly mixed mud
54			B-1	110-2."	DR	recovery 0.0/0.2
					RD	cuttings sample
			J-6	23	SS	recovery 0.6/1.3
56	(SP)	sand lens 6" thick		50-4	.5"	
					RD	
58						
			PB-5		PB	recovery 0.6/1.0
60	(SP)	sand lens				
					RD	
62						
				107 7"	DR	slightly quieter recovery 0.0/0.5
64			B-2		RD	
66						excessive cuttings big hole in places cleaned tub mixed mud
68						

Sheet 3 of 4

Project DESIGN UNIT A445 **Date Drilled** 11-2-83 **Hole No.** 38-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		48.0-81.0 <u>SANDY GRAVEL</u>: (continued)	J-7	60	SS	recovery 0.2/0.5
					RD	rock in shoe
		70.6 sandy clay lens				
70						
	(CL)					
					PB	
72			PB-6			recovery 0.2/2.0 tube destroyed
			J-8	17 29 50-3"	SS	recovery 0.4/1.2
74						
					RD	
76						
78				28 59 50-3"	DR	recovery 0.0/1.2 fell out when driven out
					RD	
80						
		B.H. 81.0' Terminate boring				Installed piezometer to bottom complete drilling 11-2-83
82						
84						
86						
88						
90						
92						

Sheet 4 **of** 4

BORING LOG 38-5

Proj: DESIGN UNIT A445
Date Drilled 11-7-83
Ground Elev. 631
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 80.3
Hole Diameter 4 7/8
Hammer Weight & Fall SS 140 lb. @30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.8 CONCRETE:			GB	Start drilling 7:15
	GP	0.8-1.2 BASEROCK:				
	SM	1.2-6.5 ALLUVIUM SILTY SAND: moderate yellowish brown, fine sand; non-plastic fines; loose; dry to moist				
2			SH-1		SH	recovery 2.5/2.5
4						
			J-1	8 11 12	SS	recovery 1.0/1.5
6					RD	set tub & cased to 5'
	SP	6.5-20.5 SAND: salt and pepper granitic origin; fine sand; trace silt; occasional medium and coarse sand lenses; medium dense; moist				
8			C-1	7 28	DR	recovery 0.7/1.0
		9.0-sand/silty sand			RD	
10						
						mixed mud rig chatter
12			PB-1		PB	recovery 0.5/2.5 fell out
14		silt content increases	J-2	19 40 50-5"	SS	recovery 0.8/1.4
16					RD	
18						
			C-2	43 80	DR	
20					RD	

Sheet 1 of 4

Project DESIGN UNIT A445 **Date Drilled** 11-7-83 **Hole No.** 38-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	6.5-20.5 SAND: (continued)			RD	
	SW/GP	20.5-29.8 SAND/GRAVELLY SAND: salt and pepper; granitic origin; interbedded with varying percentages of sand and gravel; very dense; moist to wet; occasional cobbles				rig chatter mixed mud
22			C-3	130	DR	too gravelly for pitcher recovery 0.2/0.6 rock in sample
					RD	
24						
			J-3	25 50-5"	SS	recovery 0.7/0.9
26					RD	
28						
				51 77	DR	recovery 0.0/1.0 fell out smoother
30	SM/SP	29.8-42.0 SILTY SAND: moderate yellowish brown fine sand; non-plastic fines; very dense; moist; increased silt content with depth to 37.0 feet			RD	
			C-4	36 42	DR	recovery 1.0/1.0
					RD	
32						
34						
			PB-2		PB	recovery 0.9/1.4
36		silt content decreases with depth	J-4	28 42 50-5"	SS	recovery 1.4/1.4
					RD	
38						
40			C-5	33 96-5"	DR	recovery 0.8/0.9
					RD	
42	SP/SW	42.0-48.5 SAND/GRAVELLY SAND INTERBEDDED: salt and pepper; varying percentages of fine to coarse sand and gravel; (see next page)				
44						

Sheet 2 **of** 4

Project DESIGN UNIT A445 **Date Drilled** 11-7-83 **Hole No.** 38-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP SW	42.0-48. 5 <u>SAND/GRAVELLY SAND INTERBEDDED</u>:(continued) granitic origin; very dense; moist			RD	
					PB	chatter lost sample
46						
					RD	
48						
	GP	48.5-58.0 <u>SANDY GRAVEL</u>: salt and pepper; granitic origin; fine to coarse gravel; fine to coarse sand; very dense; moist; several 6" cobbles with sand matrix		90 110-	DR 4"	all slough in barrel lost sample
50					RD	intense chatter
52						
54						
			C-6	150-5	"DR	mixed mud
					RD	recovery 0.3/0.4
56						intense chatter
58	SM	58.0-60.0 <u>SILTY SAND</u>: moderate brown; Fe staining; very fine sand; non-plastic fines; very dense; moist	J-5	44 50 -	SS 4.5"	recovery 0.7/0.9
					RD	rig chatter
60	GP	60.0-80.3 <u>SANDY GRAVEL</u>: moderate brown; Fe stained; fine to coarse gravel; fine to coarse sand; very dense; moist				
62						
64			C-7	150-4	"DR	recovery 0.3/0.3
					RD	intense chatter
66						
68						

Sheet 3 **of** 4

Project DESIGN UNIT A445 **Date Drilled** 11-7-83 **Hole No.** 38-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GP	60.0-80.3 SANDY GRAVEL: (continued)			RD	
		weathered granitic cobble	J-6	103-35"	DR	rocks coming in quit driving
					RD	
70						recovery 0.1/0.3
						intense chatter
72						
74			J-7	150-3"	DR	recovery 0.2/0.2
					RD	fell out
						intense chatter
76						
78						
80			C-8	165-4"	DR	recovery 0.3/0.3
		B.H. 803 Terminate boring				hole tremied grout to surface
82						Complete drilling 11-7-83
84						
86						
88						
90						
92						

Sheet 4 **of** 4

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

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

BORING LOG 38-6

Proj: DESIGN UNIT A445
Date Drilled 11/8-9/83
Ground Elev. 632'
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 80.2
Hole Diameter 4 7/8"
Hammer Weight & Fall SS 140lbs @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.1 ASPHALT			GB	started drilling 4:30
	SM	0.1-7.5 ALLUVIUM SILTY SAND: moderate yellowish brown; very fine to fine sand; non-plastic fines; loose; dry to moist				
2			SH 1		SH	recovery 2.5/2.5
4			J-1	1 2 2	SS	recovery 1.2/1.5 set tub & cased to 5' 11-8-83
6					RD	11-9-83
			C-1	9 14	DR	recovery 0.9/1.0 rings broke apart
8	SP	7.5-14.5 SAND: salt and pepper; fine sand trace silt; medium dense; dry to moist; granitic origin			RD	
10						
12		becoming well graded; fine to coarse sand with some graded bedding; contains occasional fine gravel; very dense	J-2	17 21 31	SS	recovery 0.7/1.5
					RD	
14					PB	recovery 0.0/2.5 fell and washed out
	SW	14.5-30.9 GRAVELLY SAND: salt and pepper; to coarse sand; fine to coarse gravel trace silt; very dense; moist; granitic origin				
16					RD	
				29 60	DR	
18					RD	rock in shoe
20						

Sheet 1 of 4

Project DESIGN UNIT A445 **Date Drilled** 11/8-9/83 **Hole No.** 38-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	14.5-30.0 GRAVELLY SAND: (continued)			RD	
22			PB 1		PB	recovery 1.4/2.0
		increase silt content	J-3	4 25 50-4	SS 5"	mostly slough recovery 0.3/1.4
24					RD	
26						light chatter
				48	DR	3 rings rest loose
28			C-2	90-3		recovery 0.5/1.0
					RD	mixed mud
30	SP	30.0-33.4 SAND: salt and pepper; fine to medium sand; trace silt; medium dense to dense; moist	PB2		PB	smoothed out recovery 2.1/2.5
32		becoming silty sand	J-4	15 11 14	SS	recovery 1.0/1.5
34	ML	33.4-38.0 SILT: dark yellowish brown low plastic fines; fine sand; stiff; moist			RD	
36		becoming sandy silt	PB 3		PB	recovery 1.6/2.5
38	SM	38.0-40.8 SILTY SAND: dark yellowish brown; very fine sand; non-plastic fines; contains beds of sandy silt; dense; moist				
40			J-5	12 14 27	SS	
					RD	recovery 1.2/1.5
42	SP	40.8-50.5 SAND: salt and pepper; fine to medium sand; trace silt; dense; moist; granitic origin				
44						

Sheet 2 **of** 4

Project DESIGN UNIT A445 **Date Drilled** 11/8-9/83 **Hole No.** 38-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	40.8-50.5 SAND: (continued)		84	DR	5 good rings
			C-3	100-3"		recovery 0.5/0.7
					RD	rig chatter
46						
			C-4	157	DR	recovery 0.4/0.5
					RD	2 good rings rest disturbed
48						
50						
	GW	50.5-80.2 SANDY GRAVEL: salt and pepper; variable percentages with some sand lenses; very dense; moist; granitic origin	J-6	59	SS	intense chatter recovery 0.3/0.5
52	(SP)				RD	mixed mud
54		contains coarse gravel and cobble				
						intense chatter
			C-5	170-3"	DR	rock in shoe
56					RD	
58						
60		fe staining		86	DR	recovery 0.5/0.7
			C-6	64-3"		
					RD	representative sample disturbed rock in shoe
62		coarse gravel				intense chatter
64						
			C-7	150 3-5"	DR	
66						recovery 0.2/0.3 mixed mud light chatter losing circulation
68						

Project DESIGN UNIT A445 **Date Drilled** 11/8-9/83 **Hole No.** 38-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GW	50.5-80.2 SANDY GRAVEL: (continued)			RD	drilling smoother
	(GM)	silty lens	C-8	36-3"	DR	drove 3" pulled out [illegible]
				100-3"	RD	[illegible] drove 3" more recovery 0.4/0.4 intense chatter
70						
		contains cobbles				
72						
74						
			C-9	150-4"	DR	recovery 0.3/0.3
76					RD	
78						
80			C-10	200-2.5"	DR	recovery 0.2/0.2
		B.H. 80.2 Terminated boring				completed drilling 11-9- [illegible] hole flushed; installed piezometer to bottom, 60-80' slotted
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

Converse Consultants, Inc.
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Geo/Resource Consultants

BORING LOG 38-7

Proj: DESIGN UNIT A445
Date Drilled 11/7-8/83
Ground Elev. 633
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 77.7
Hole Diameter 4 7/8"
Hammer Weight & Fall SS 140 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.7 CONCRETE: 0.7-1.0 BASEROCK:			GB	start drilling 4:15
	GP					
	SM	ALLUVIUM				
2		1.0-6.5 SILTY SAND: dark yellowish brown fine sand; non-plastic fines; medium dense; dry to moist	SH-1		SH	recovery 2.5/2.5
4						
			J-1	5 5 6	SS	recovery 1.5/1.5 set tub & cased to 5' 11/7/83
6	SP	6.5-32.5 SAND: salt and pepper; fine sand; trace silt; medium dense; moist; granitic origin; subangular occasional fine to coarse gravel lenses			RD	11/8/83 mixed mud
8	(GP)					
			C-1	8 14	DR	recovery 1.0/1.0
10					RD	
12		becoming dense	PB-1		PB	recovery 1.5 /2.5
14		14.5-sand/silty sand	J-2	26 32 29	SS	recovery 0.5/1.5
16					RD	
18		gravelly sand lens				rig chatter
			C-2	20 49	DR	recovery 0.6/1.0
20					RD	

Sheet 1 of 4

Project DESIGN UNIT A445 **Date Drilled** 11/7-8/83 **Hole No.** 38-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	6.5-32.5 SAND: (continued) contains scattered gravel			RD	
22			PB-2		PB	recovery 1.0/2.0
24	SW	sand becoming fine to coarse in lenses	J-3	17 52	SS	recovery 0.7-1.0
					RD	
26						
28				48	DR	
		29.5-gravelly sand	C-3	67		recovery 0.7/1.0
30					RD	
32	SM SC	32.5-40.0 SILTY SAND/CLAYEY SAND: moderate yellowish brown fine sand, low plastic fines; very dense; moist	PB-3		PB	recovery 1.7/2.0 tube screwed up at top taking another sample
34			PB-4		PB	recovery 1.1/1.9 good sample
36			J-4	19 33 44	SS	recovery 1.5/1.5
38					RD	
40	SP	40.0-47.5 SAND: salt and pepper; medium sand; trace gravel; trace silt; very dense; moist to wet; granitic origin; gravels occur in interbeds				rig chatter
42			PB-5		PB	recovery 1.7/2.1
44				28	SS	

Sheet 2 **of** 4

Project DESIGN UNIT A445 Date Drilled 11/7-8/83 Hole No. 38-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	40.0-47.5 SAND: (continued)	J-5	50-3"	SS	
					RD	recovery 0.4/0.8
	SW	becoming well graded fine to coarse sand; graded bedding				
46						
48	GW	47.5-77.7 SANDY GRAVEL: salt and pepper; fine to coarse gravel; fine to coarse sand; very dense; moist; occasional cobbles		150	DR	intense chatter
					RD	mixed mud recovery 0.0/0.5
50						
						intense chatter
52						
54						
			B-1			mixed mud
56		sand content decreases		31	DR	mostly slough recovery 0.3/1.0
			C-4	68		
					RD	
58						
60				64	DR	slough barrel full
			C-5	100-	5"	recovery 0.5/1.0
					RD	mixed mud intense chatter
62						
64						
	(SW)			SS	DR	recovery 0.2/0.7
66		gravelly sand lens	C-6	100-	3"	
					RD	
68						

Sheet 3 of 4

Project DESIGN UNIT A445 **Date Drilled** 11/7-8/83 **Hole No.** 38-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GW	47.5-77.7 SANDY GRAVEL: (continued)			RD	
70						attempted SS, 3' of slough in hole, mixed mud washed out and tried DR
	(SP	sand lens	C-7	156	DR RD	recovery 0.4/0.6
72						1 good ring, rest disturbed
74						intense chatter
76						recovery 0.2/0.5
			C-8	200	DR	
78		B.H. 77.7 Terminate Boring				Due to drilling and sampling problems in gravel; end hole early tremied grout to surface Complete drilling 11-8-83
80						
82						
84						
86						
88						
90						
92						

Sheet 4 **of** 4

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-38 during the 1981 geotechnical investigation of the Metro Rail Project. It should be noted that this boring is about 600 feet east of the proposed location of the North Hollywood Station (Design Unit A445). The results of the survey conducted in this borehole is, however, included in this appendix since it is considered generally representative of the soil conditions present at the Station site. 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.

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*
38	10-65	2343	126	117	12	2340±240	1040	98	52	12	1040±150
	65-115	2619	292	131	11	2620±420	1940	180	97	11	1940±280
	115-145	2330	313	117	7	2330±430	1359	144	68	7	1360±210
	145-199	4076	1457	204	12	4080±1600	1441	340	72	12	1440±410

$\bar{V}p$ = 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

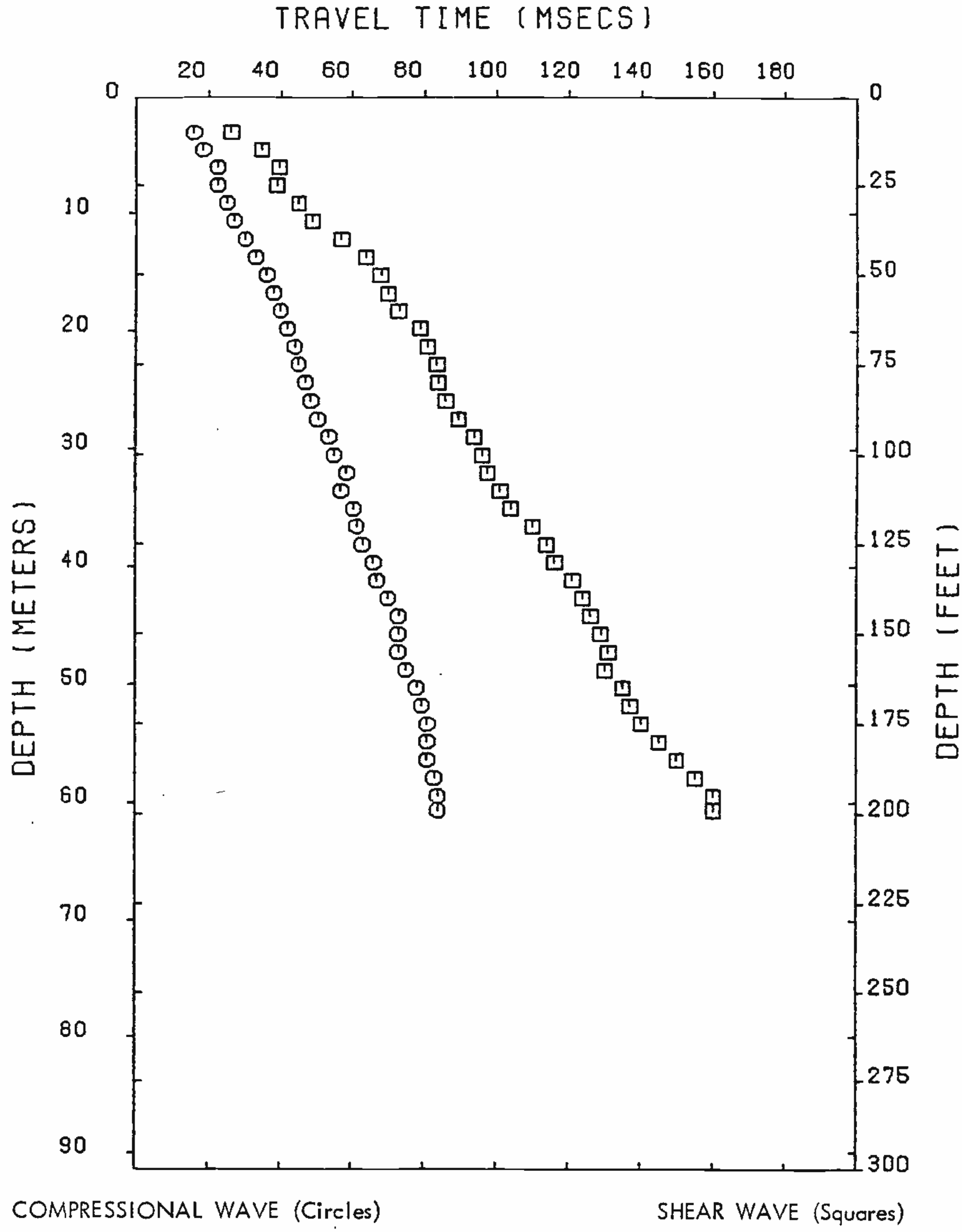
Es = estimated accuracy of shear survey

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 38

DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
B-1

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication ____ by ____

Appendix C

Water Quality Analysis

APPENDIX C WATER QUALITY ANALYSIS

C.1 INTRODUCTION

Chemical analyses were performed on one groundwater sample obtained from Boring CEG-38 at a depth of about 138 feet during the 1981 geotechnical investigation. This boring is located about 600 feet east of the proposed North Hollywood Station site. The water sample was subjected to chemical analyses by Jacobs Laboratories (formerly PJB Laboratories in Pasadena, California). 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:

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

Chemical constituents tested by Jacobs Laboratories include:

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

C.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 Jacobs Laboratories, the results of which are presented herein. The results of the water quality tests are summarized in Table C-1 and the data summary sheets.

TABLE C-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
38	2	138.0	02-25-81	7.8	906	463	0.44	Ca/SO_4

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 $CaCO_3$/l ÷ 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. P81-03-017-5

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

Sample labeled: HOLE 38-2"

Conductivity: 1,200 µ mhos/cm

Turbidity: NTU

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

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	133	6.14
Magnesium, Mg	28	2.30
Sodium, Na	105	4.88
Potassium, K	6.6	0.17
		Total 13.49
Anions determined:		
Bicarbonate, as HCO_3	165	2.70
Chloride, Cl	34	0.95
Sulfate, SO_4	463	9.64
Fluoride, F	0.4	0.02
Nitrate, as N	5.5	0.39
		Total 13.70
Carbon dioxide, CO_2, Calc.	4	
Hardness, as $CaCO_3$	447	
Silica, SiO_2	29	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.44	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	906	

Appendix D

Geotechnical Laboratory Testing

APPENDIX D GEOTECHNICAL LABORATORY TESTING

D.1 INTRODUCTION

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

The soil 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 D-1, while data from the 1981 geotechnical investigation are presented in Table D-2. The soils listed in these tables are described in Section 5.0 of the report.

D.1.1 Data Analysis

The summary of laboratory test results is presented in Tables D-1 and D-2. Figures D-1 through D-3 summarize strength and modulus data appropriate for the Alluvium found at depths less than about 15 feet. Figures D-4 through D-6 summarize strength and modulus data appropriate for the Alluvium found at depths greater than about 15 feet. 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.

D.2 INDEX AND IDENTIFICATION

D.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.

D.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 D-7 through D-12.

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.

D.2.3 Atterberg Limits

Because of the granular nature of the soil samples obtained from the field, Atterberg Limit Tests were not performed during the course of this and the 1981 geotechnical investigation.

D.2.4 Moisture Content

Moisture content determinations were performed on selected soil 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 D-1 and D-2.

D.2.5 Unit Weight

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

The test procedure entailed measuring specimen dimensions with a precision ruler or micrometer. Weights of the sample were 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 D.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables D-1 and D-2.

D.3 ENGINEERING PROPERTIES: STATIC

D.3.1 Unconfined Compression

Unconfined compression tests were performed on selected samples of cohesive soils 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 D-1 and D-2.

D.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:

D.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 reached an axial strain of 5 percent, 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 D-13 through D-26.

D.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 D-1 and D-2 and are shown on Figures D-1 and D-4.

D.3.4 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 D-1 and D-2.

D.3.5 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. Calculated porosities are summarized in Table D-2.

D.4 ENGINEERING PROPERTIES: DYNAMIC

D.4.1 Resonant Column

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

D.4.1.1 Sample Preparation and Handling

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

D.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures (σ_{3c}), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures D-27 through D-32.

D.4.1.3 Data Reduction

Data obtained from the resonant column tests were reduced in accordance with the ASTM "Suggested Methods* of Test for Shear Modulus and Damping of Soils by the Resonant Column."

D.4.2 Cyclic Triaxial Compression--Dynamic Shear Strength

This test evaluates soil shear strength, liquefaction, and deformation characteristics under cyclic loading conditions. A cylindrical specimen of soil is encased in a thin rubber membrane, subjected to a confining pressure in a closed cell, brought to the desired equilibrium stress and saturation conditions, and cyclically loaded in the axial direction.

D.4.2.1 Sample Preparation and Handling

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

D.4.2.2 Test Conditions and Parameters

Test conditions and parameters may vary in the cyclic triaxial test. The procedures followed for this project were:

- Stress controlled: Cyclic axial loads of relatively constant magnitude and loading frequency were applied, and the resulting axial strains and specimen pore pressures were measured.

- Saturation: The specimens were artificially saturated using flushing and back pressure techniques. Typical back pressures of 60 to 100 psi were required to saturate the specimens. The degree of saturation was measured using Skempton's B parameter, $\Delta u/\Delta\sigma_{3c}$. The saturation level criterion for this project was a minimum B value of 0.95, except for a few tests which reached a minimum of 0.94.

- Consolidation: Specimens were allowed to consolidate under the specified static ambient stress levels. Consolidation was monitored either by measuring specimen volume changes or by closing the drainage lines and verifying that buildup of pore pressures

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*ASTM Special Technical Publication 479.

did not occur. A consolidation ratio ($K_c = \sigma_{1c}/\sigma_{3c}$) of 1.0 was used for this program.

D.4.2.3 Apparatus

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

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

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

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

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

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

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

D.4.2.4 Data Reduction

The following methods and definitions were used in the reduction of test data from the continuous strip chart recording:

- Axial stress: Given in terms of axial load and the unconsolidated specimen cross section area.
- The cyclic testing apparatus is designed to maintain relatively constant axial loads, and no correction is made for changing cross sectional areas of the sample during the test. This is common practice for this type of test.
- Axial stress: Given in terms of the consolidated specimen length. No correction is made for changing specimen length during the test.
- Cyclic axial strain: The larger of the zero-to-peak axial strain or the double amplitude, peak-to-peak, strain for the given cycle of loading.
- Pore pressure ratio: Ratio of the maximum net pore pressure change recorded during the cycle, divided by the net confining pressure, σ_{3c}.
- Failure criteria: A 10% double amplitude axial strain in the cyclic triaxial tests was selected for plotting.

Graphs of the test results appear on Figure D-33.

TABLE D-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						
38-1	C-1	9.5	Sand	A	104	6					32	0.42						
	PB-1	14.5	Silty Sand	A	99	20									X			X
	C-2	19.5	Sandy Silt/Silty Sand	A	103	10					30	0.17			X			
	PB-2	24.5	Silty Sand	A	108	16												
	PB-3	34.2	Sandy Silt	A	107	19									X			X
	C-4	39.2	Sand	A	114	12					30	1.45						
	PB-4	43.5	Gravelly Sand	A	117	16									X			X
	C-8	62.9	Gravelly Sand	A	130	7					(1)	(1)			X			
38-2	C-1	9.0	Sand	A	97	3					27	0.56						
	PB-1	14.5	Sandy Silty/Silty Sand with Trace Clay	A	97	25									X			X
	PB-1	14.0	Gravelly Sand	A	99	15												
	PB-2	24.5	Sand	A	105	14									X			X
	C-3	29.0	Sandy Silt	A	95	14					27.5	1.10			X			
	PB-3	34.5	Clayey Sand	A	97	27				1.85								
	C-4	38.8	Gravelly Sand	A	124	8					34	1.00						

TABLE D-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						
38-2	PB-4	44.5	Sand/Silty Sand	A	109	20									X			X
	C-7	65.0	Sand	A											X			
38-3	C-1	9.0	Silty Sand	A	92	20					30	0.10			X			
	PB-1	17.0	Silty Sand	A	103	15									X			X
	PB-3	33.5	Sandy Clay/Clayey Sand	A	106	22				3.03								
	C-3	39.0	Sand	A	115	13												
38-4	PB-1	17.0	Silty Sand	A	105	17									X			X
	PB-1	17.0	Silty Sand	A	100	20				(2)								
	C-2	29.0	Sand with Gravel	A	112	15					30	1.10			X			
	PB-3	34.5	Clayey Silt/Clayey Sand	A	104	22				2.05								
	PB-4	44.5	Gravelly Sand	A	120	10									X			X
38-5	C-1	9.0	Sand/Silty Sand	A	105	17									X			
	C-2	19.5	Sand	A	117	9												
	C-4	31.0	Silty Sand	A	112	12					30	0.55						
	PB-2	35.9	Silty Sand	A	110	19									X			X

TABLE D-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						
38-6	PB-1	23.0	Sand	A	110	12												X
	C-2	28.2	Sand with Gravel	A	111	12								X				
	PB-3	39.5	Silty Sand	A	104	20												
	PB-3	39.5	Silty Sand/Sandy Silt	A	105	21								X				X
38-7	C-1	9.5	Sand	A	97	12					35	0.25						
	PB-1	14.5	Sand/Silty Sand	A	103	10								X				X
	C-3	29.5	Gravelly Sand	A	118	11					30	0.90		X				
	PB-4	43.6	Silty Sand/Clayey Sand	A	113	17				(2)								
	PB-4	43.6	Silty Sand/Clayey Sand	A	111	16								X				X

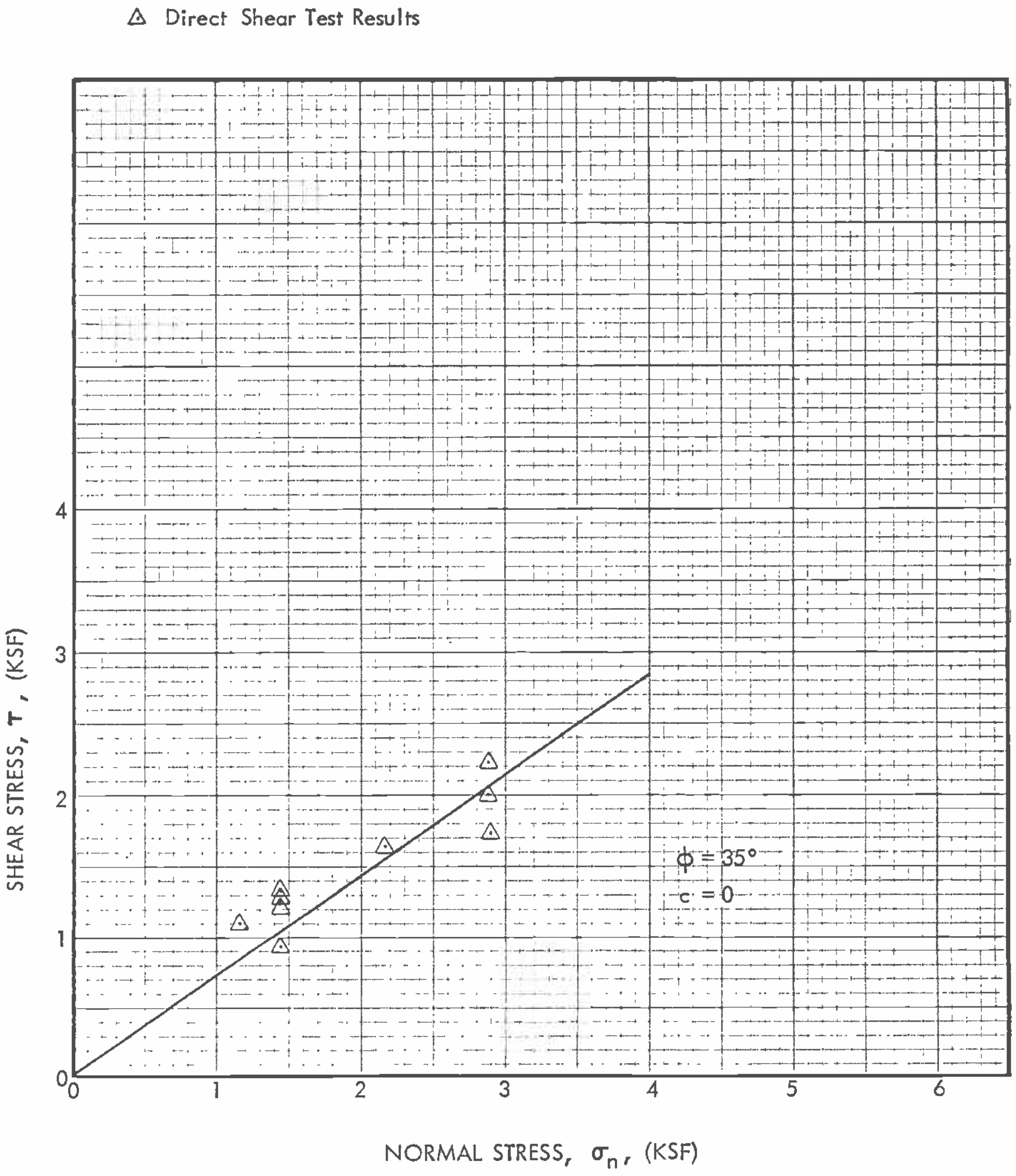
NOTES:

(1) One point test.

(2) Unconfined test performed, but sample too sandy to yield meaninful results.

TABLE D-2 COMPREHENSIVE LIST OF ENGINEERING PROPERTIES FROM LABORATORY TESTS

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits LL	Atterberg Limits PI	Particle Size Cumulative % Passing Sieve No. 4	Particle Size Cumulative % Passing Sieve No. 40	Particle Size Cumulative % Passing Sieve No. 200	Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Coefficient of Consolidation Cc (in/in Per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	Undrained Quick Direct Shear c, ksf	One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
38	J1	11	Silty sand	A_1					88	50	18												
	C1	20	Silty sand	A_1	120	11											37	0.32					
	C2	40	Gravelly sand	A_1	101	21							2.3E-4			40.1				X			
	J4	41	Silty sand						93	48	8												
	C3	60	Sand	A_1	127	10											38	0.53					
	S1	107	Silty sand	A_3	112	19																	
	S1	107	Silty sand	A_3	110	20						41.0										X	
	S1	108	Sand & silt	A_3	112	19																X	
	S2	119	Fine to medium micaceous sand	A_3	108	12							1.0E-4				34	0.39					
	S3	142	Sandy silt/Silty sand	A_4	109	19						9.3											
	S3	142	Sandy silt/Silty sand	A_4	115	18						46.2											
	S4	161	Silty fine sand, cemented	A_3	106	21						15.2										X	
	S4	161	Silty fine sand, cemented	A_3	115	17						36.3											
	S5	180	Clayey fine sand, cemented	A_3	97	27						28.7											
	S5	180	Clayey fine sand, cemented	A_3	112	17						12.2											
	S6	200	Gray fine sand	A_3	108	15											33	0.44					
	C16	201	Sand	A_3	111	14																X	



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SUMMARY OF DIRECT SHEAR TEST RESULTS-ALLUVIUM (DEPTH ≤ 15 FT.)

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METRO RAIL PROJECT

Project No.
83-1140

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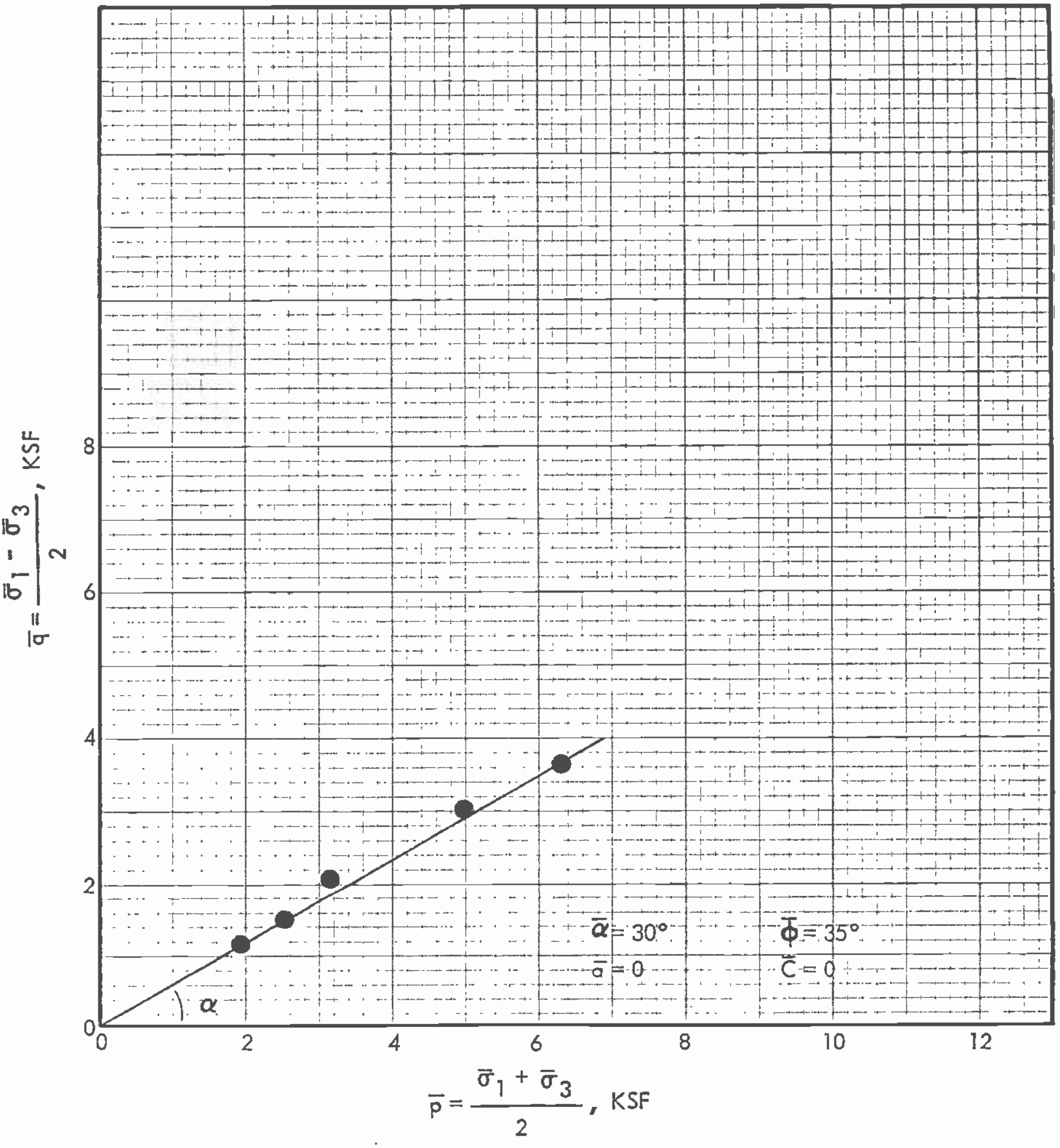
Figure No.
D-1

● Triaxial Test Results

NOTE : 1) tan α = sin ϕ



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SUMMARY OF EFFECTIVE STRENGTH DATA - ALLUVIUM (DEPTH≤15')

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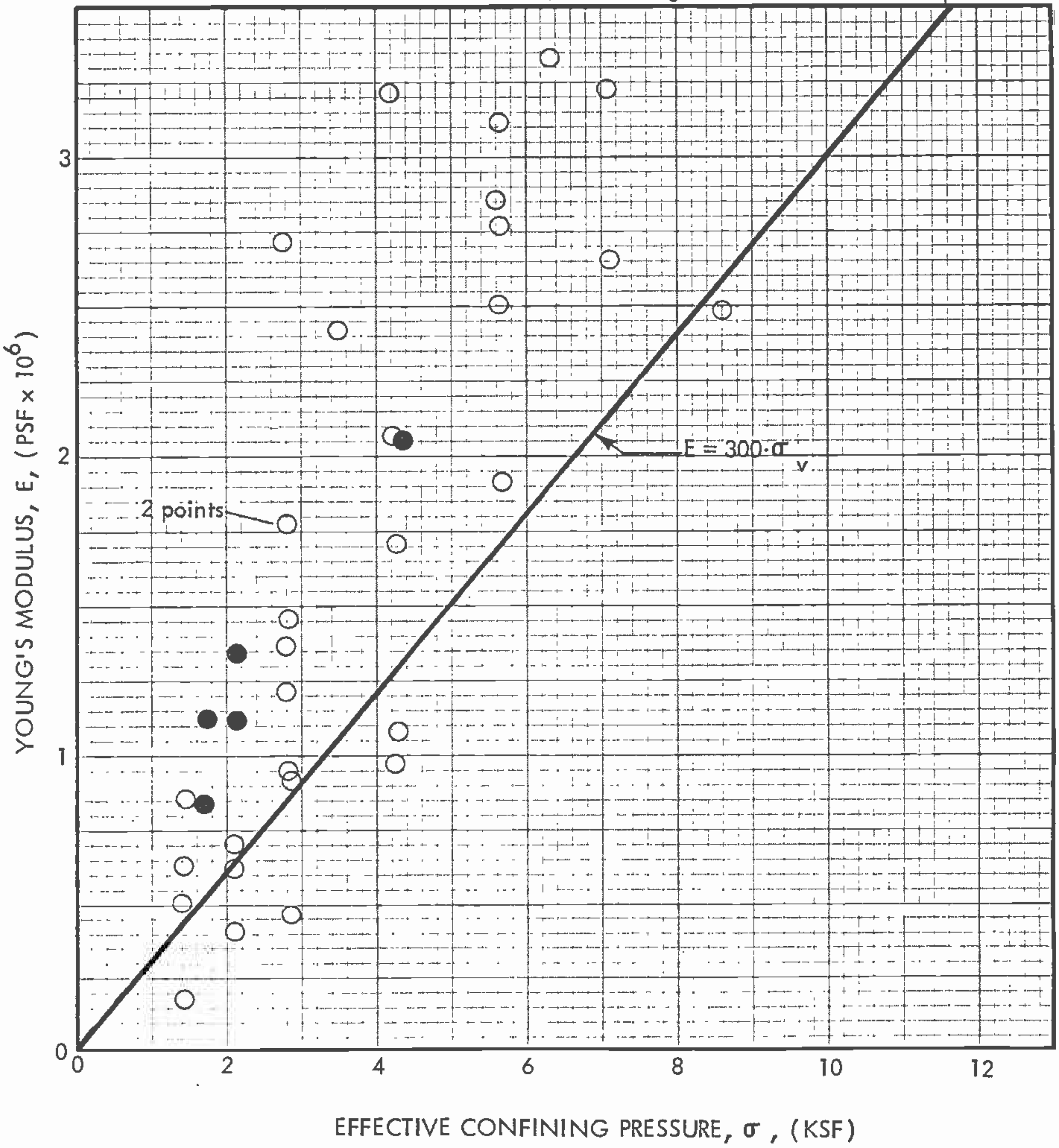
Figure No.
D-2

● 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 - ALLUVIUM (DEPTH≤15')

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Figure No.
D-3

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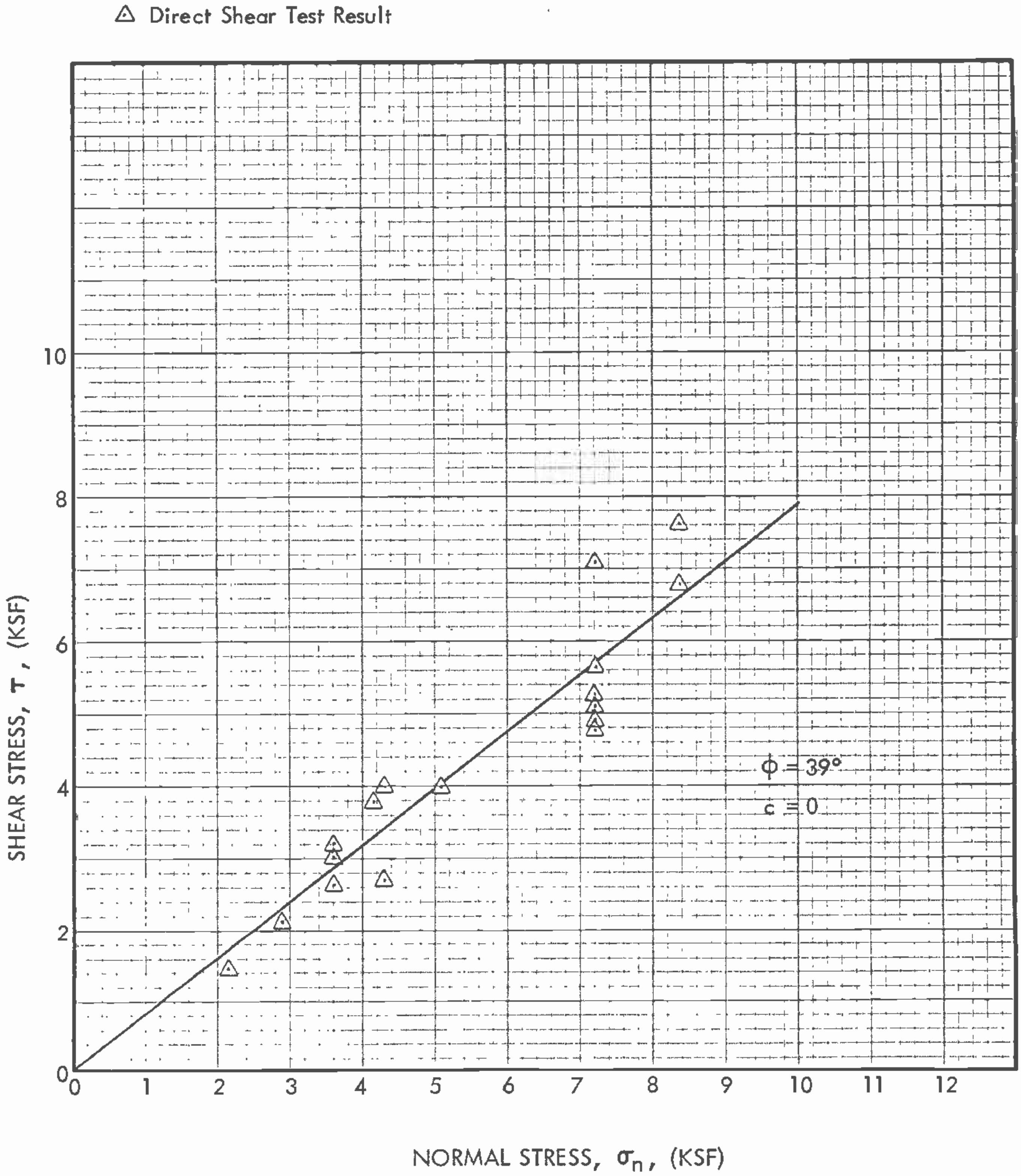

SUMMARY OF DIRECT SHEAR TEST RESULTS - ALLUVIUM (DEPTH >15 FT.)

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Figure No.
D-4

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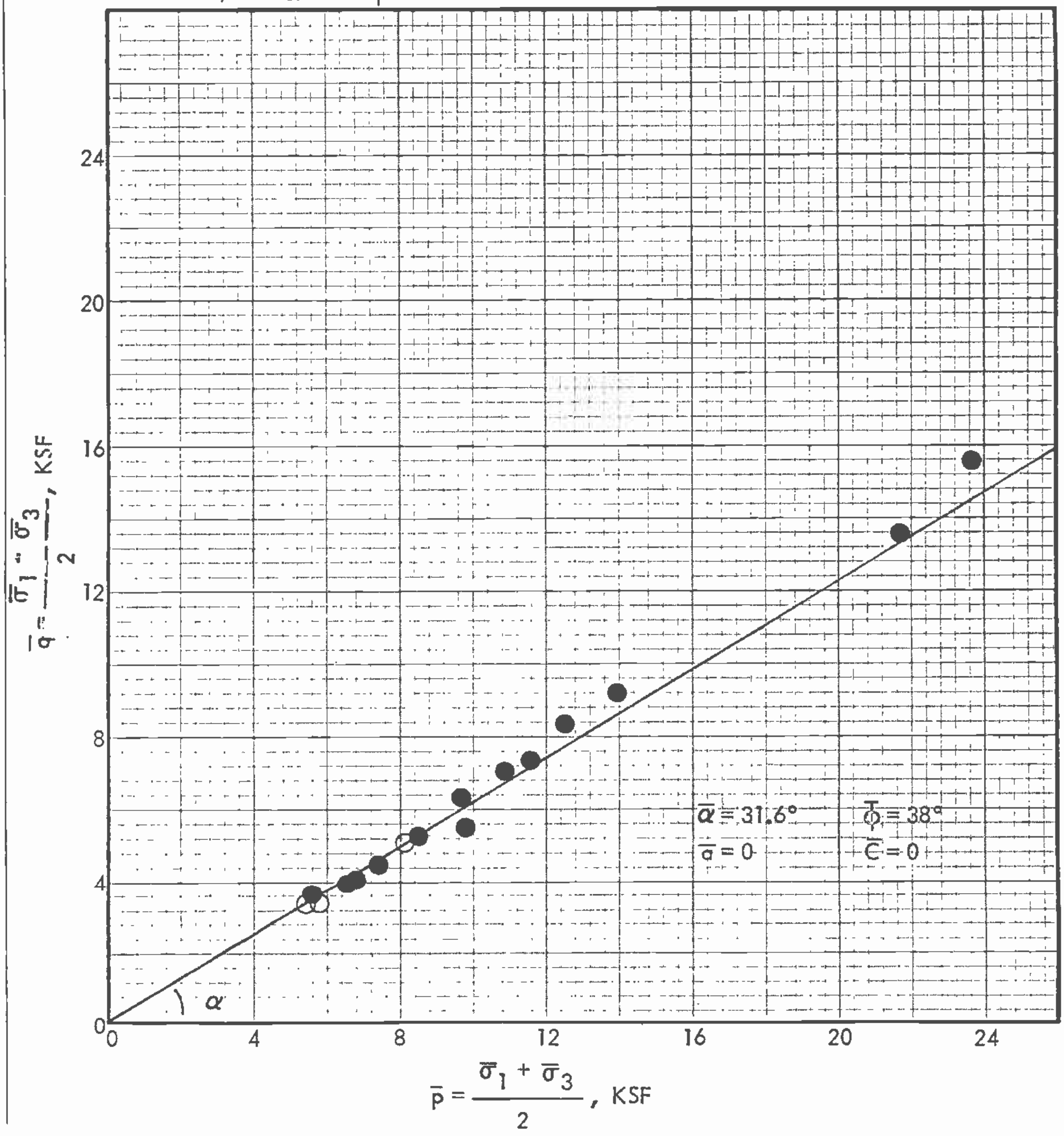
● Triaxial Test results

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



SUMMARY OF EFFECTIVE STRENGTH DATA - ALLUVIUM (DEPTH > 15')

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Figure No.
D-5

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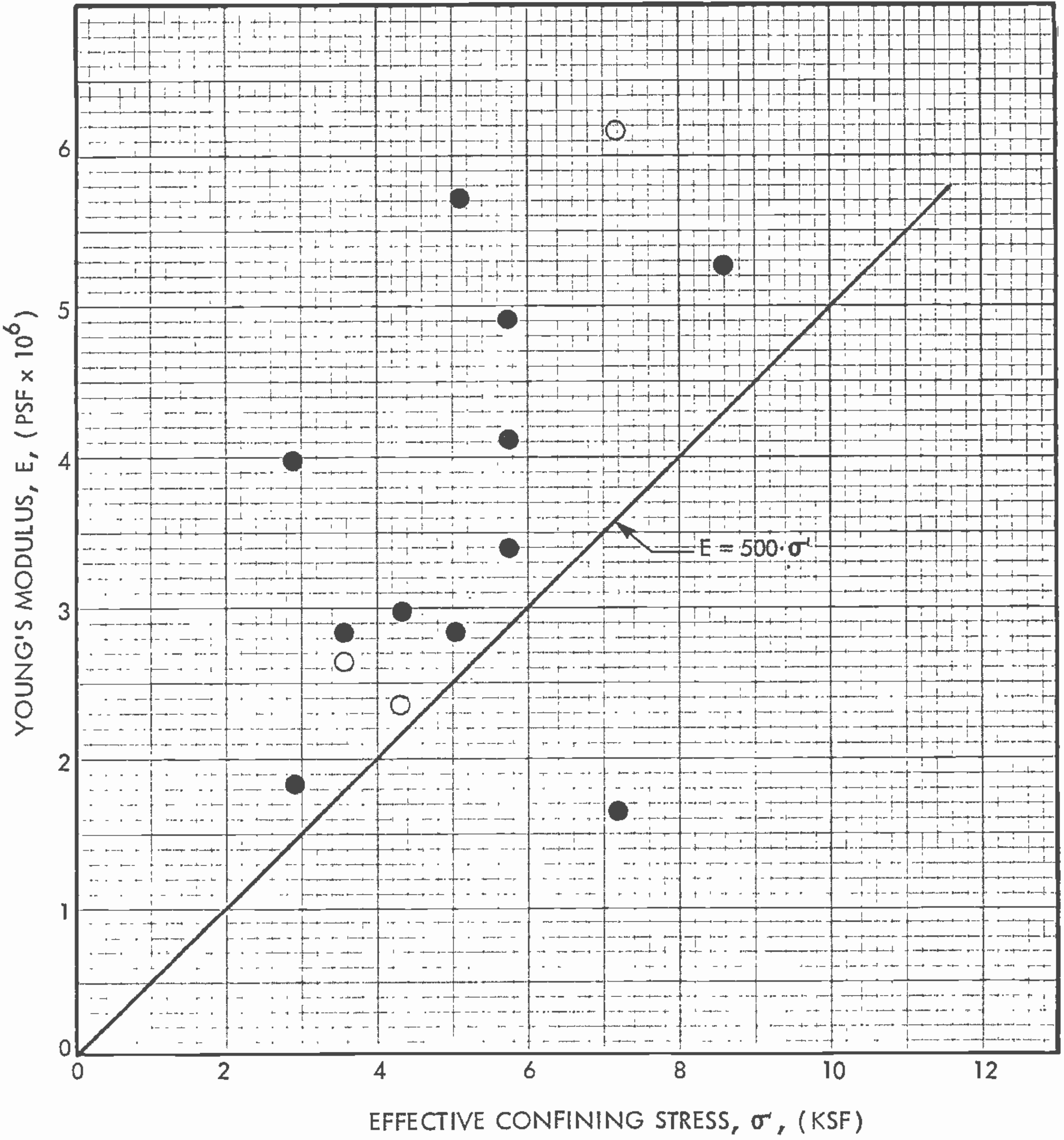
● Triaxial Test results

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 - ALLUVIUM (DEPTH>15')

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Figure No.
D-6

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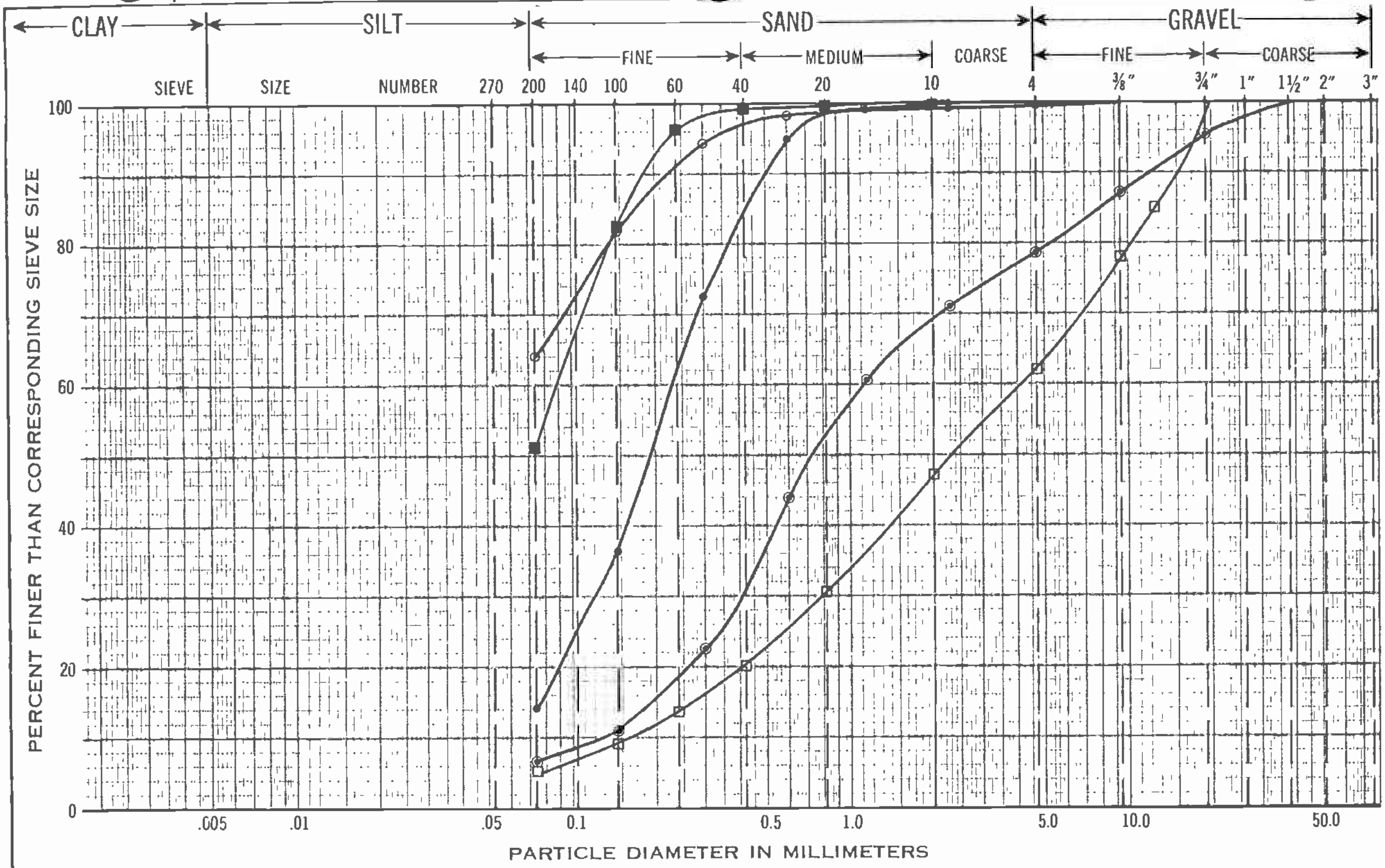

SYMBOL	BORING	SAMPLE	DEPTH, ft.
●	38-1	PB-1	12.0-14.5
■	38-1	C-2	19.0-19.5
○	38-1	PB-3	32.0-34.2
◉	38-1	PB-4	42.0-43.5
□	38-1	C-8	62.5-63.0

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
D-7

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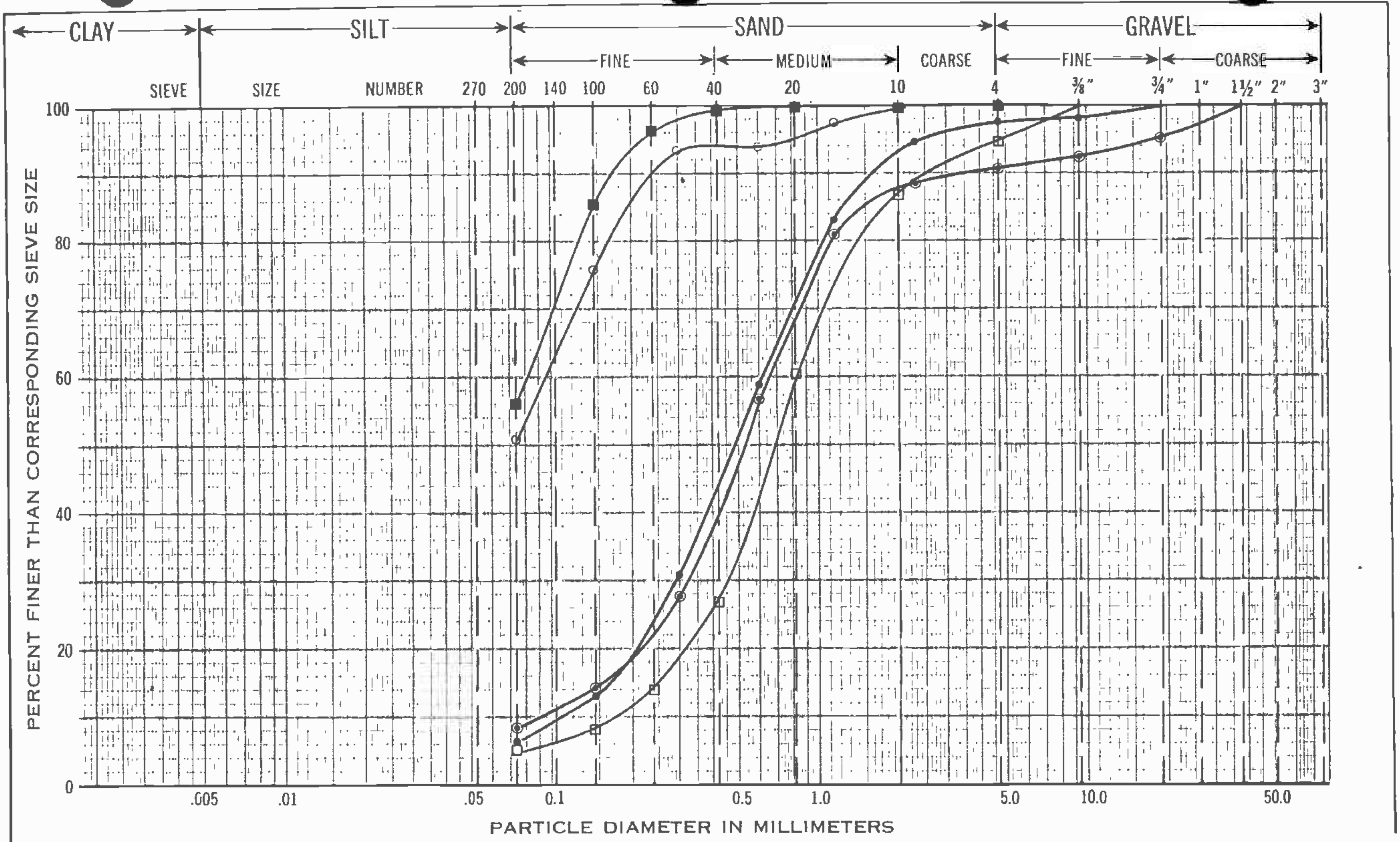

SYMBOL	BORING	SAMPLE	DEPTH, ft.
○	38-2	PB-1	12.0-14.5
●	38-2	PB-2	22.0-24.5
■	38-2	C-3	28.5-29.0
◉	38-2	PB-4	42.0-44.5
□	38-2	C-7	64.5-65.0

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
D-8

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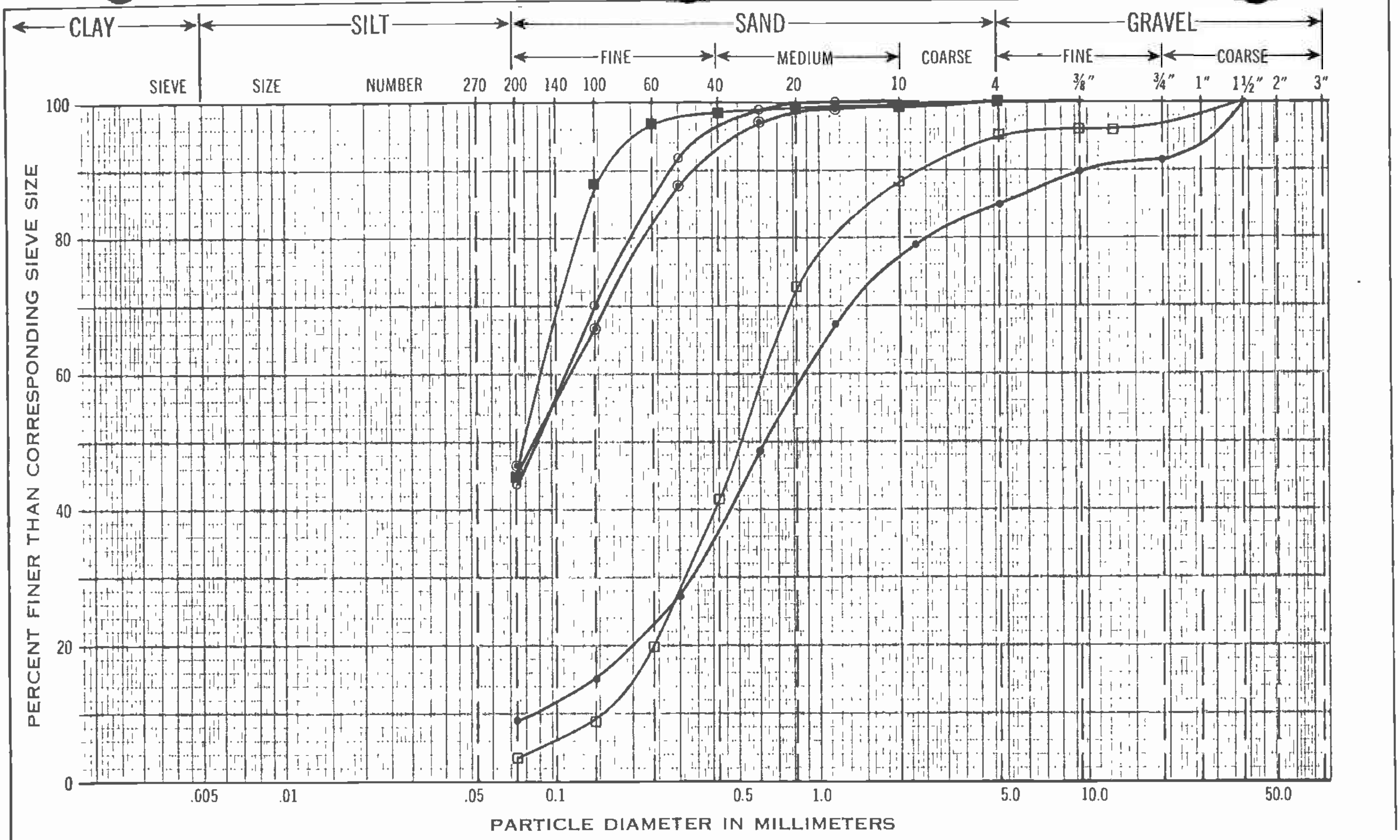

SYMBOL	BORING	SAMPLE	DEPTH, ft.
■	38-3	C-1	8.5-9.0
◉	38-3	PB-1	14.5-17.0
○	38-4	PB-1	14.5-17.0
□	38-4	C-2	28.5-29.0
●	38-4	PB-4	42.0-44.5

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Figure No.
D-9

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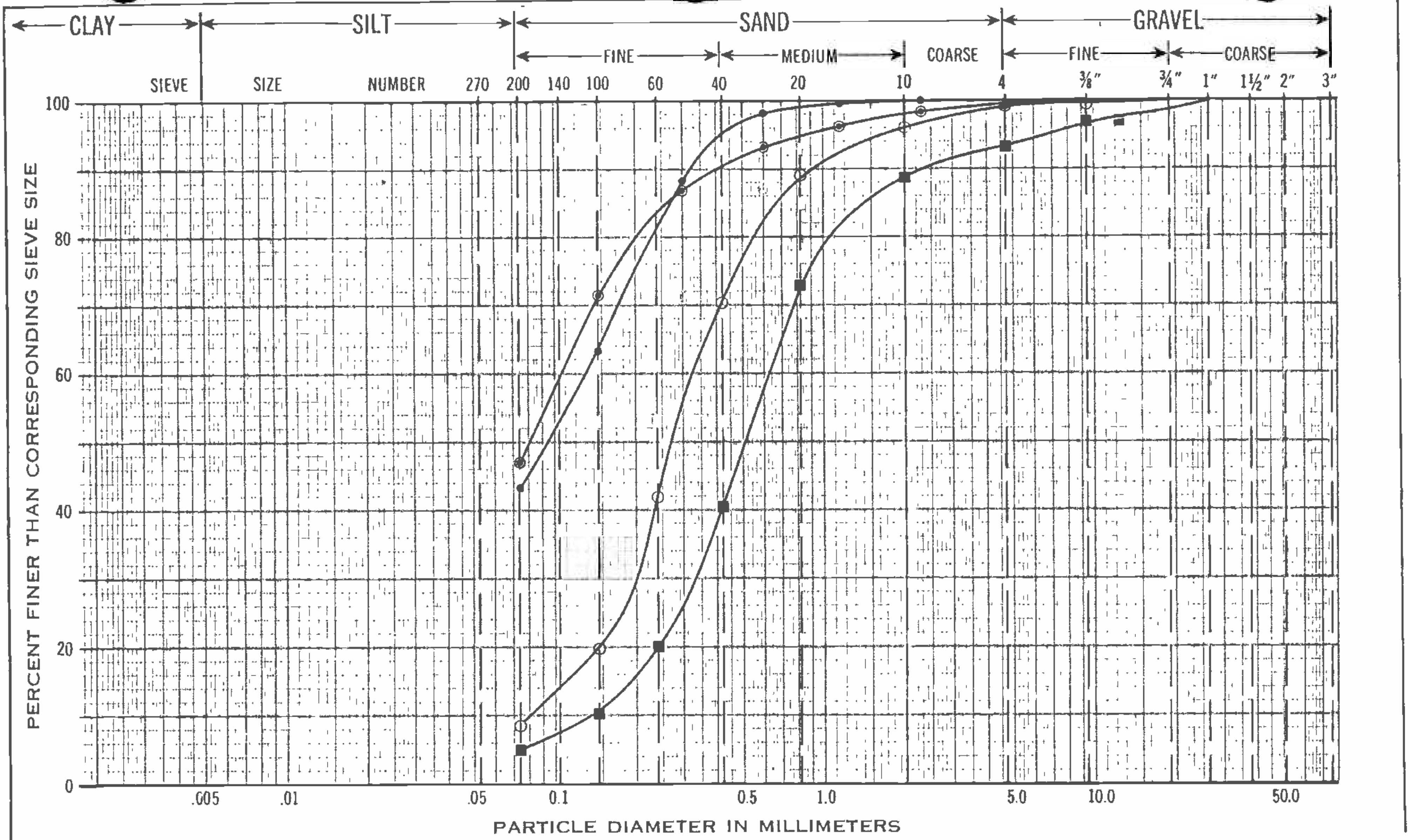

SYMBOL	BORING	SAMPLE	DEPTH, ft.
○	38-5	C-1	8.5-9.0
●	38-5	PB-2	34.5-35.9
■	38-6	C-2	27.5-28.0
◉	38-6	PB-3	37.0-39.5

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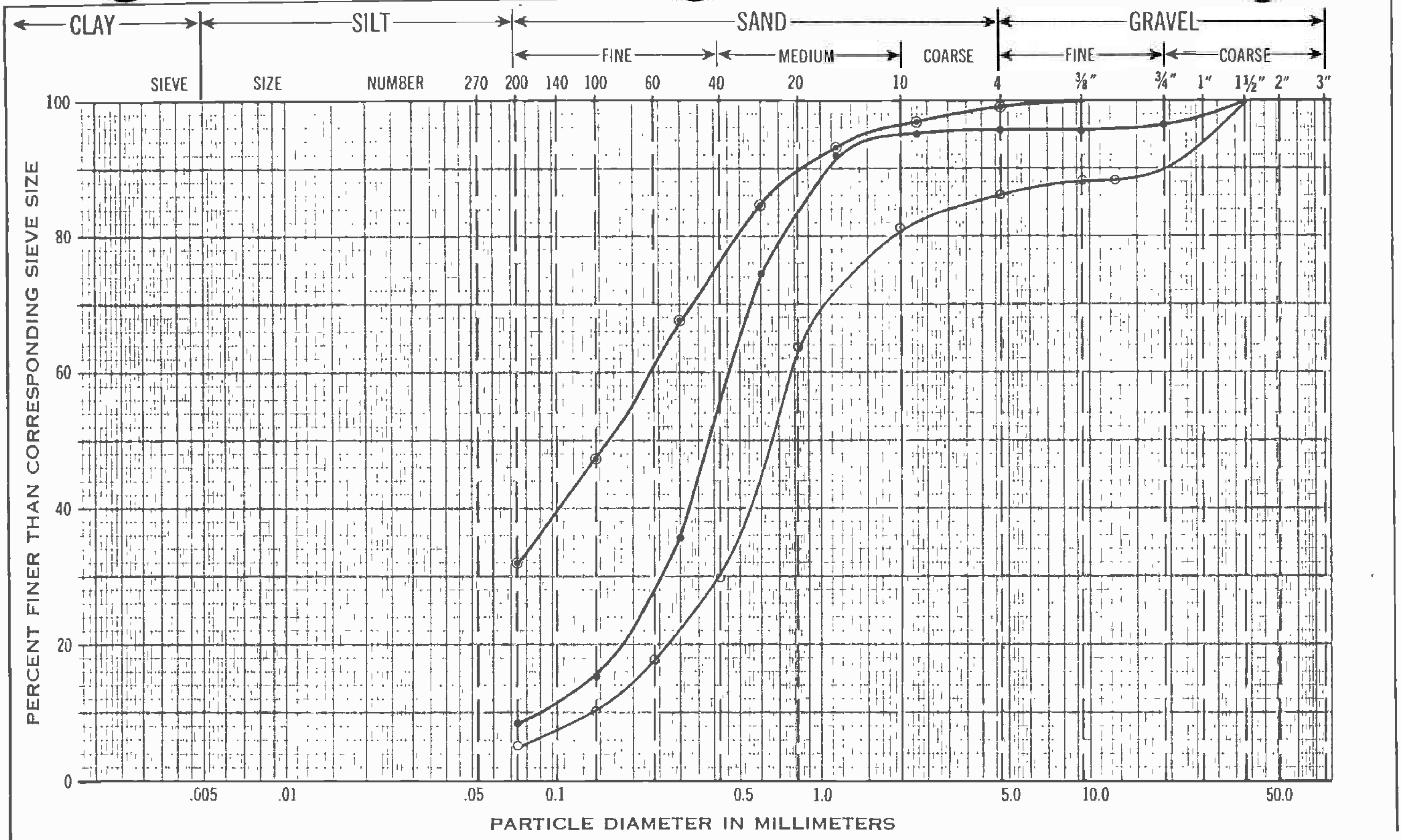

SYMBOL	BORING	SAMPLE	DEPTH, ft.
●	38-7	PB-1	12.0-14.5
○	38-7	C-3	29.0-29.5
◉	38-7	PB-4	41.4-43.6

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
D-11

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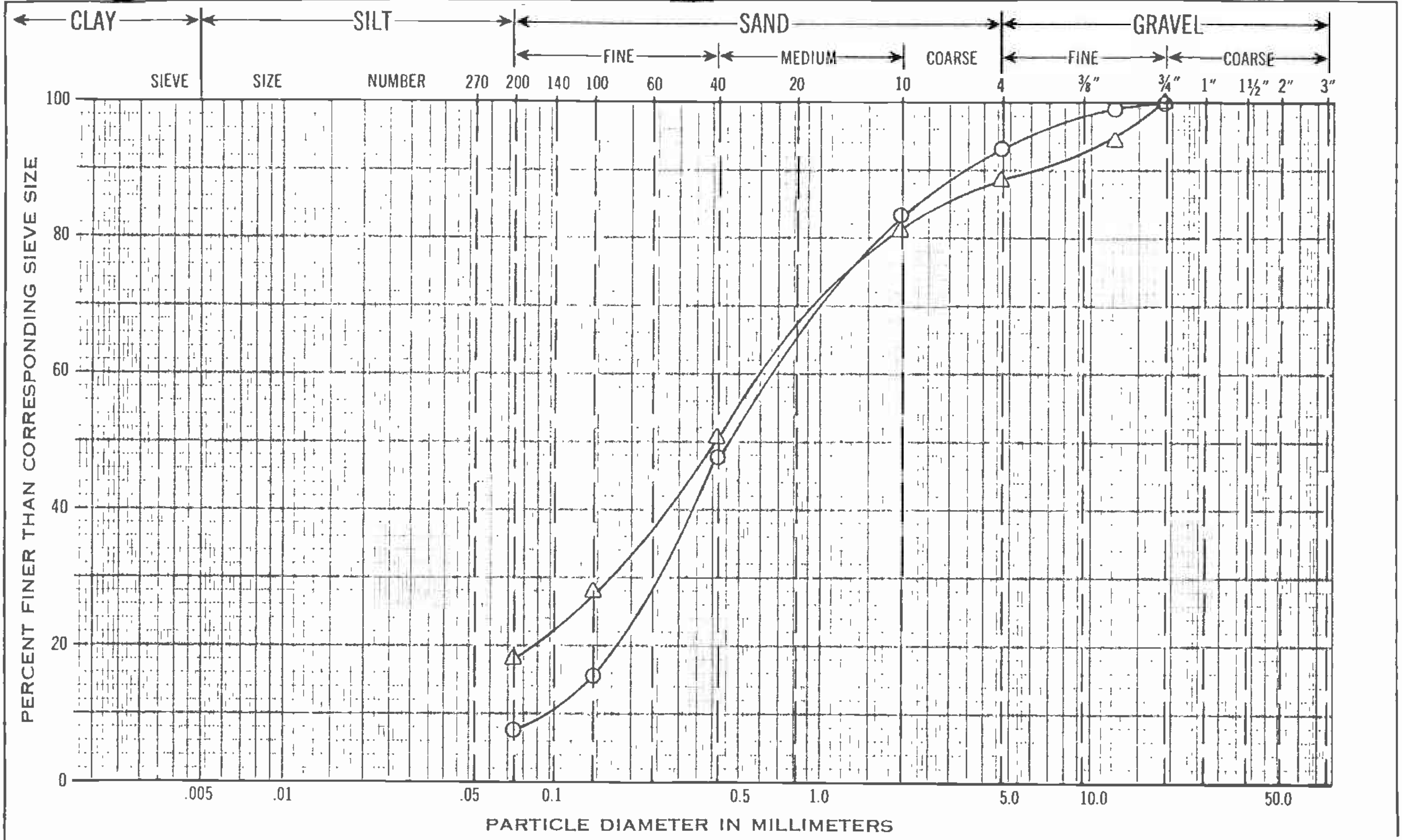

SYMBOL	BORING	SAMPLE	DEPTH
—△—	38	J-1	11'
—○—	38	J-4	42'

GRAIN-SIZE DISTRIBUTION CHART

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Drawing No.
D-12

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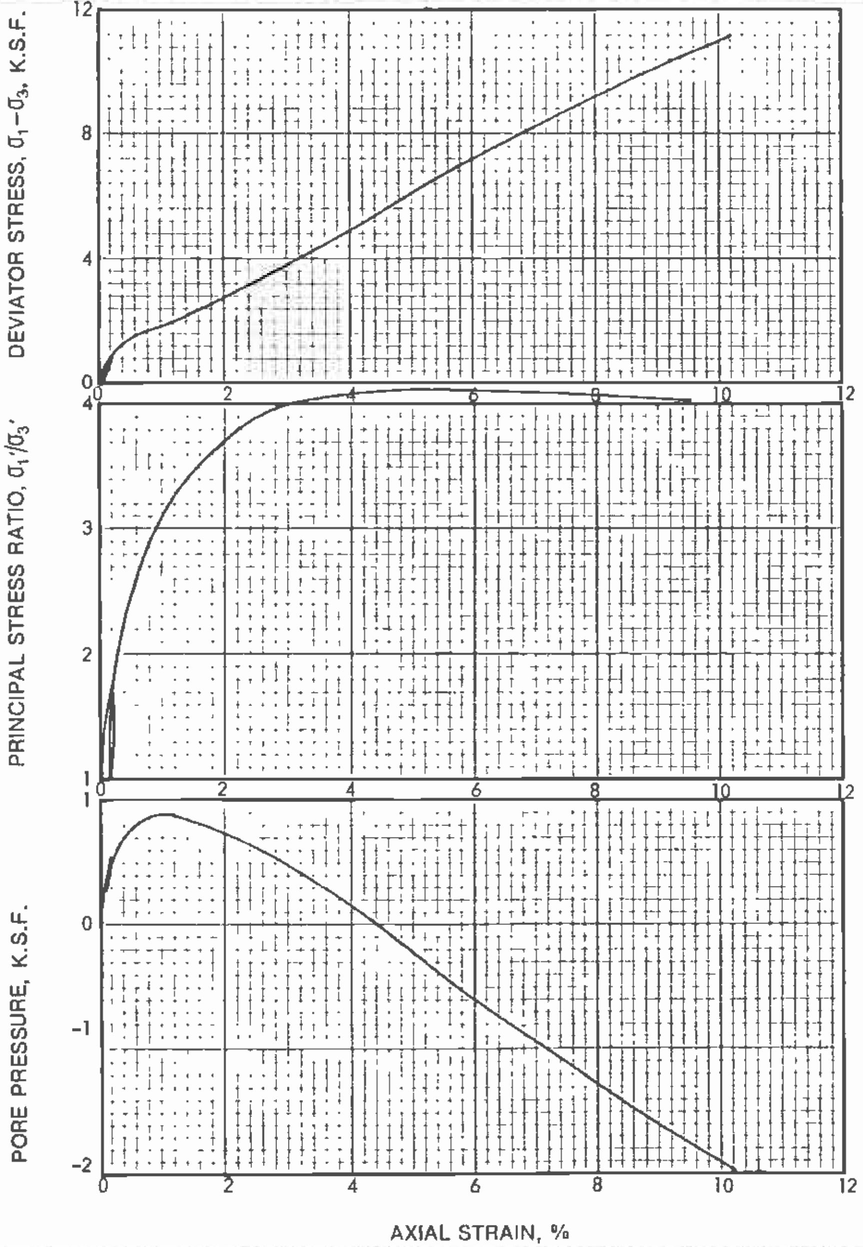

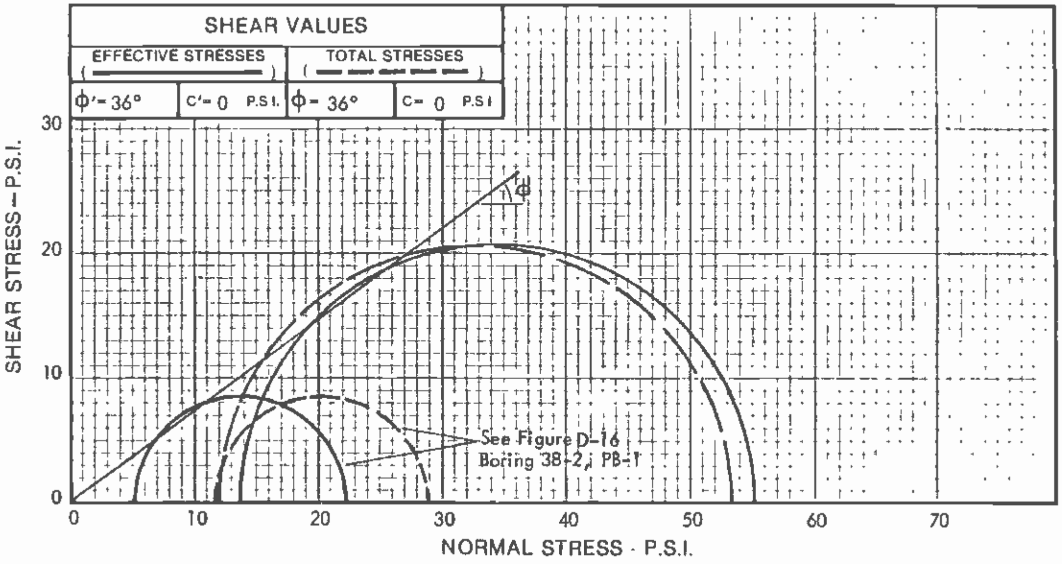

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ'= 36°	C'= 0 P.S.I.	φ= 36°	C= 0 P.S.I.

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	38-1	PB-1	12-14.5	SM	6.50	2.84	98.8	24.3	Pitcher Undisturbed
				Initial	6.50	2.84	98.8	20.2	

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		12	41.8	-1.4	13.4	55.2	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No
D-13

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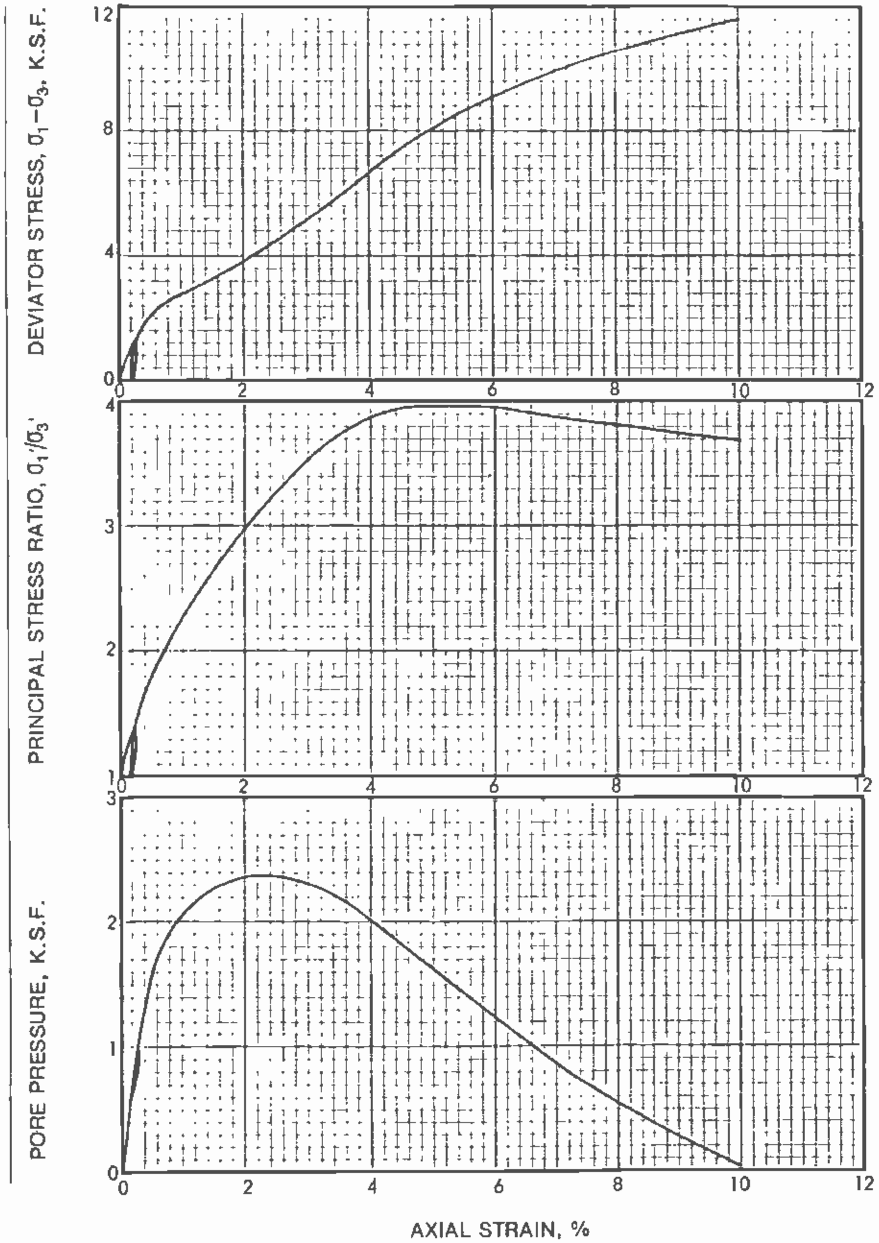

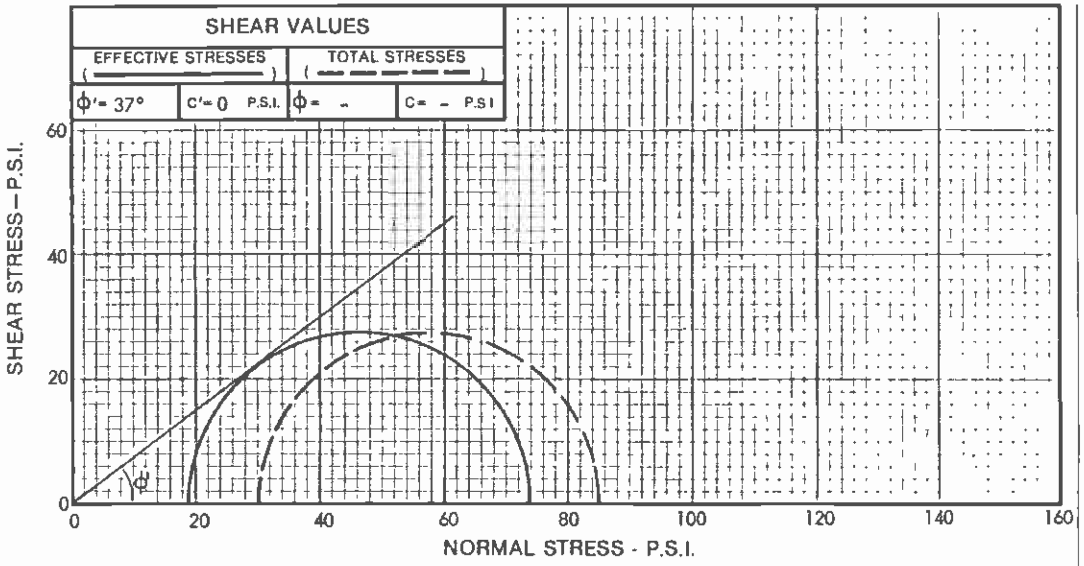

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'= 37°	C'= 0 P.S.I.	ϕ= -	C= - P.S.I

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-3	38-1	PB-3	32-34.2	ML	5.88	2.87	107.5	21.9	
				Initial	6.00	2.85	106.6	19.4	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-3		30	55.3	11.3	18.7	74.0	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No
D-14

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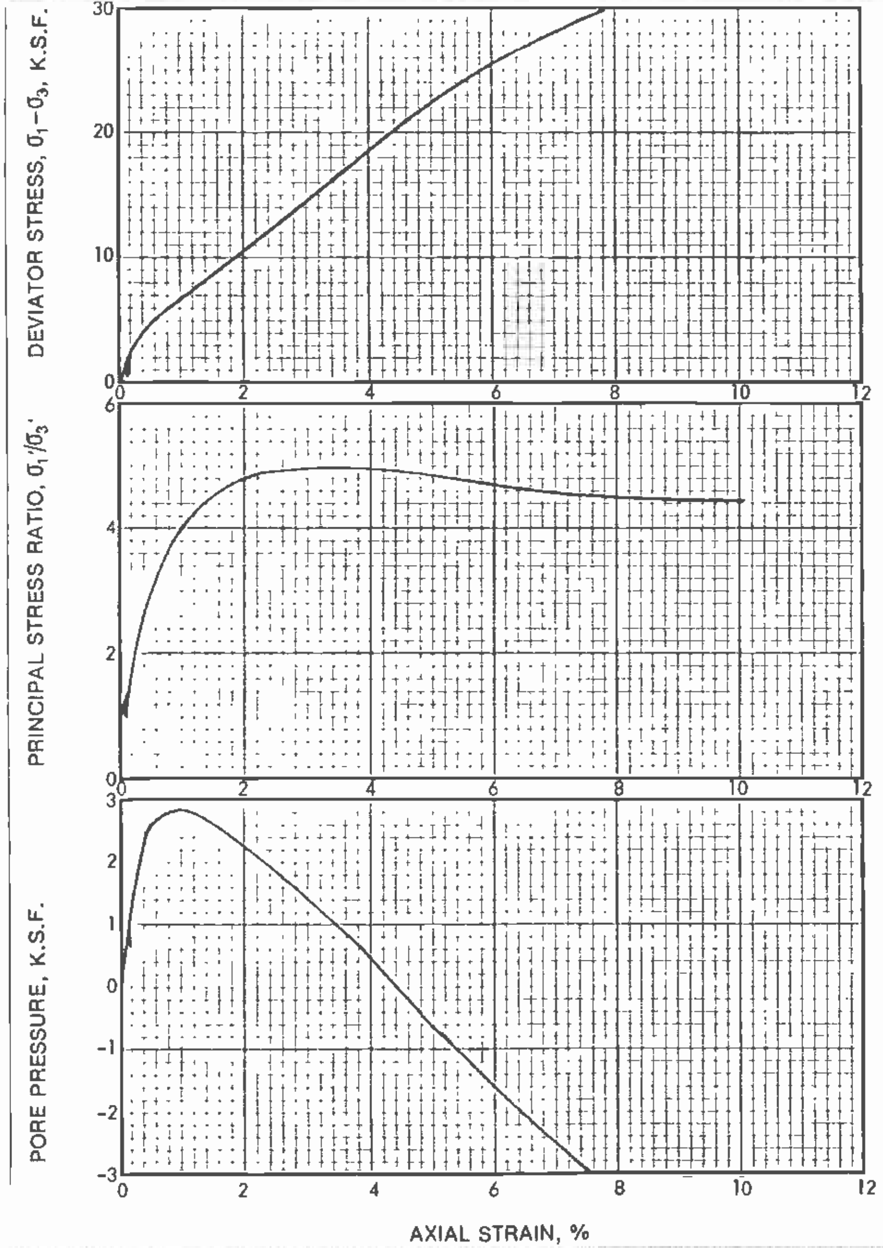

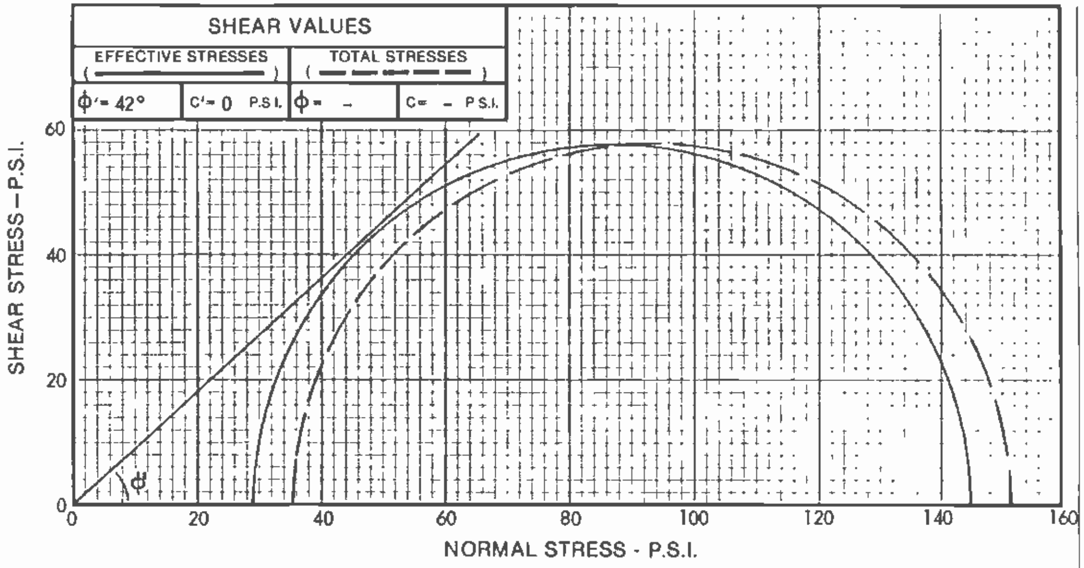

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ' = 42°	C' = 0 P.S.I.	ϕ = –	C = – P.S.I.

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-4	38-1	PB-4	42-43.5	SW	6.24	2.83	116.8	14.2	
				w/gravel					
				Initial	6.25	2.83	116.7	15.6	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 $\sigma_1-\sigma_3$ (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-4		36	115.4	7.0	29.0	144.4	ICU with Pore Pressure Measurements

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Figure No
D-15

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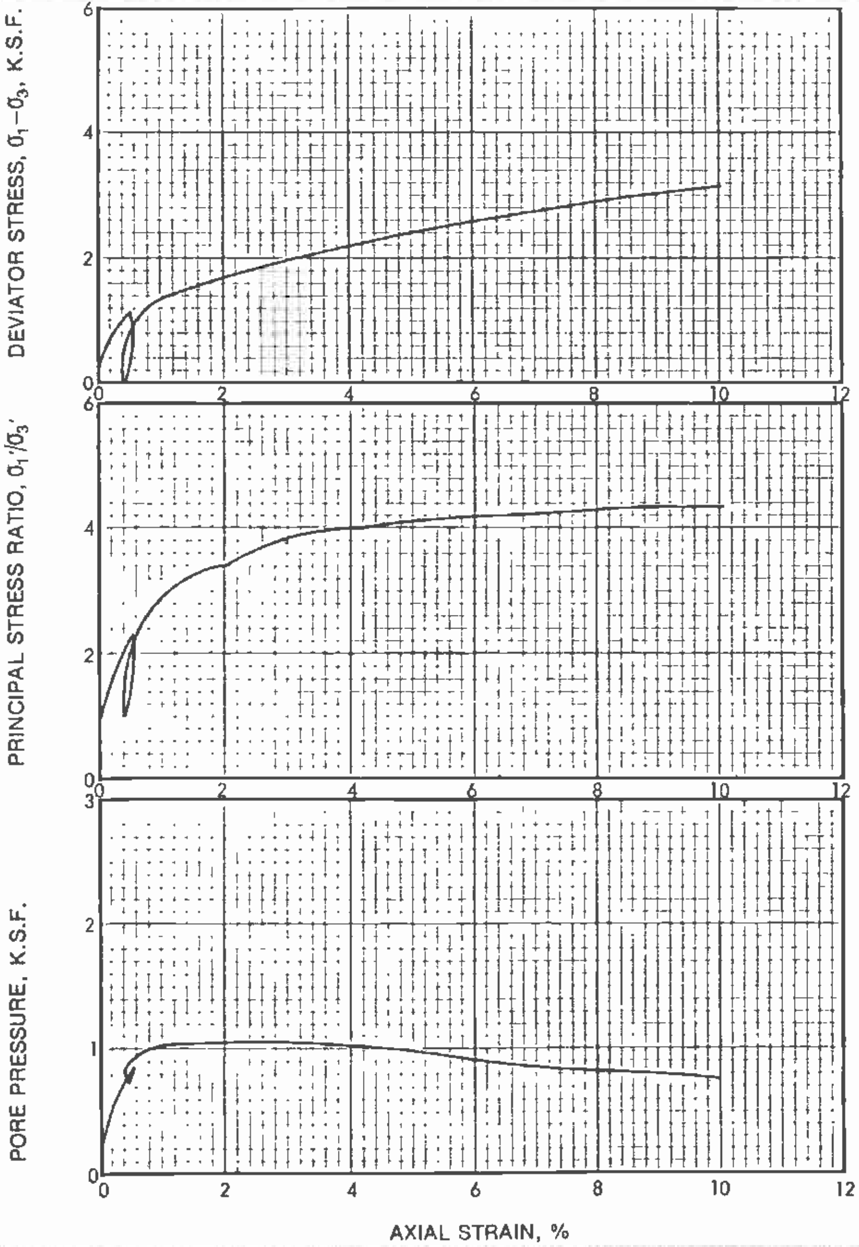

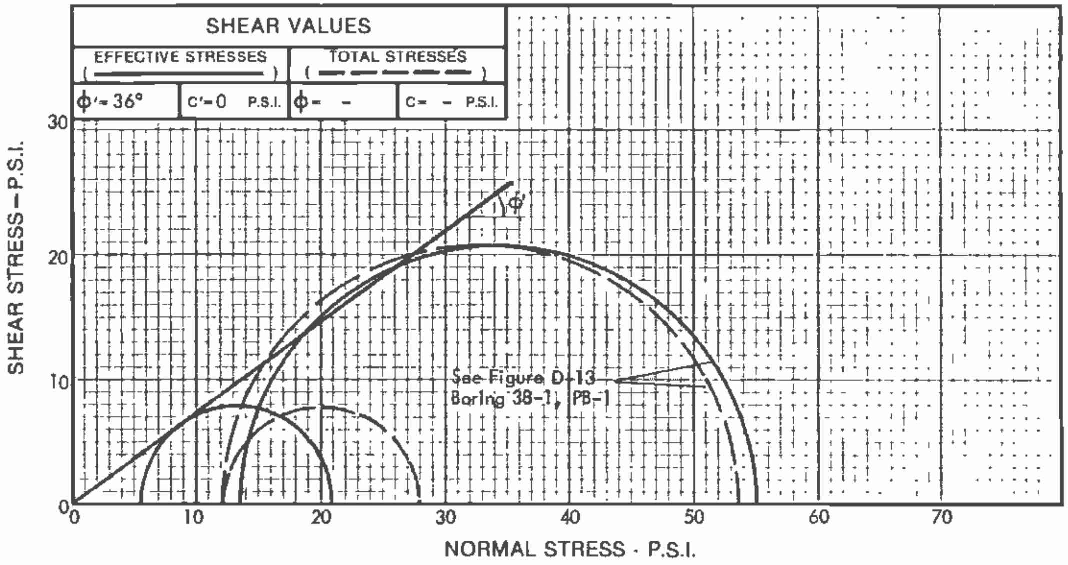

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-1	38-2	PB-1	12-14.2	SM/ML	5.98	2.83	96.7	23.8	
				Initial	6.00	2.83	96.6	25.3	Pitcher 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 $\sigma_1-\sigma_3$ (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-1		12	16.5	6.7	5.3	21.8	ICU with Pore Pressure Measurements

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Figure No
D-16

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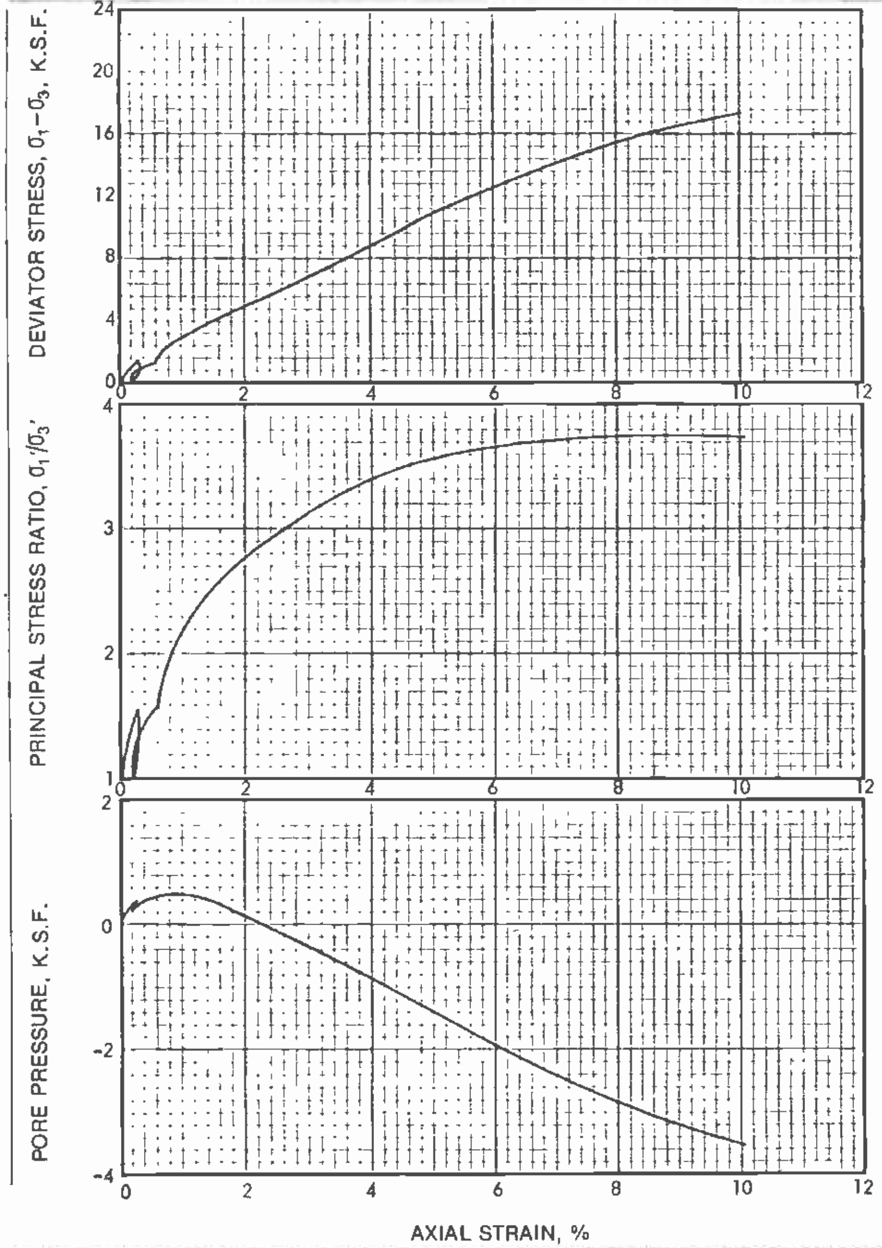

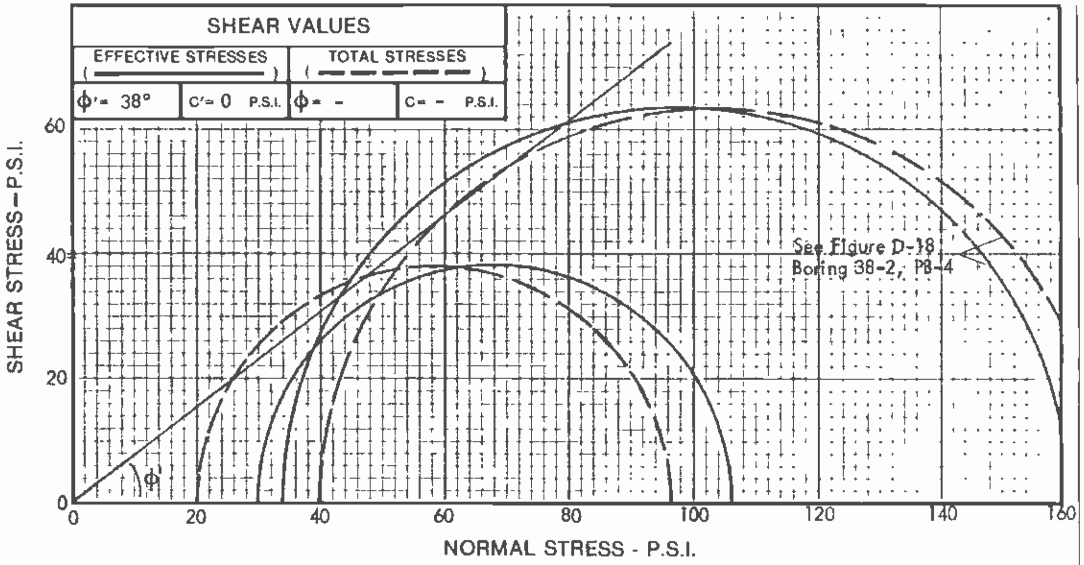

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ' = 38°	c' = 0 P.S.I.	φ = -	c = - P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI-FICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-2	38-2	PB-2	22-24.5	SP	6.49	2.83	105.4	21.1	Pitcher Undisturbed
				Initial	6.50	2.83	105.3	14.4	

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-2		20	76.0	-9.8	29.8	105.8	ICU with Pore Pressure Measurements

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Figure No
D-17

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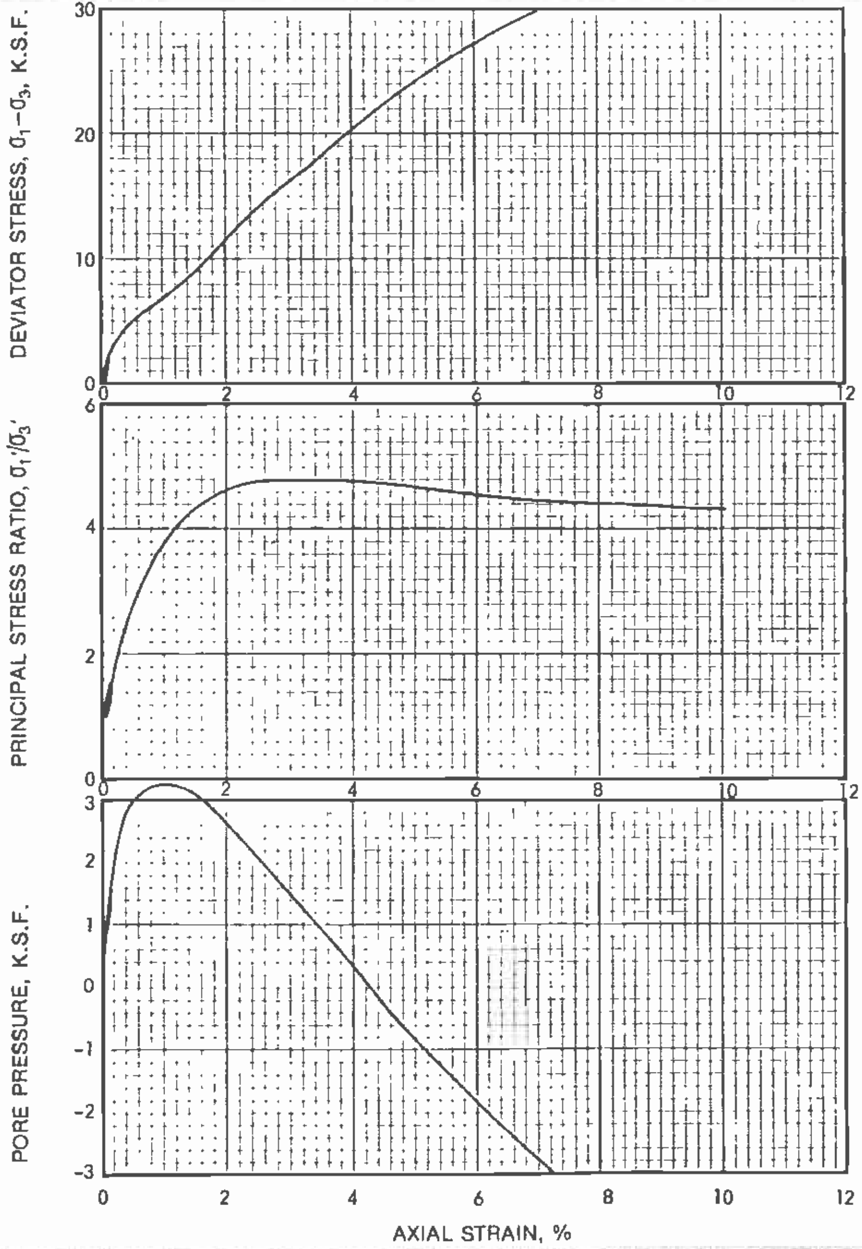

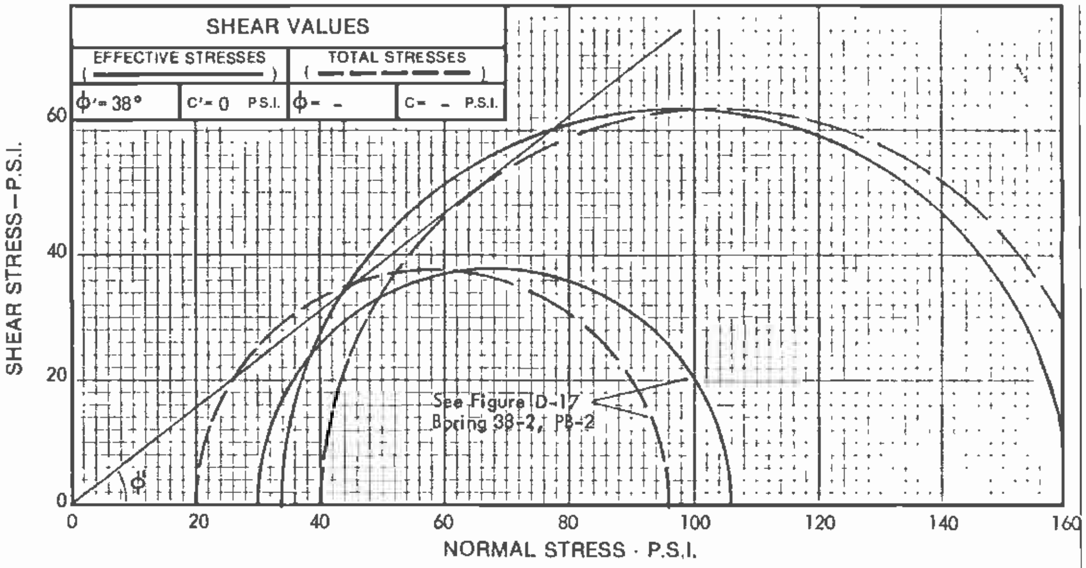

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ' = 38°	C' = 0 P.S.I.	ϕ = -	C = - P.S.I.

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-4	38-2	PB-4	42-44.5	SP/SM	6.08	2.91	108.9	17.1	
				w/gravel					
				Initial	6.13	2.84	108.6	20.2	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-4		40	127.2	6.4	33.6	160.8	ICU with Pore Pressure Measurements

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Figure No
D-18

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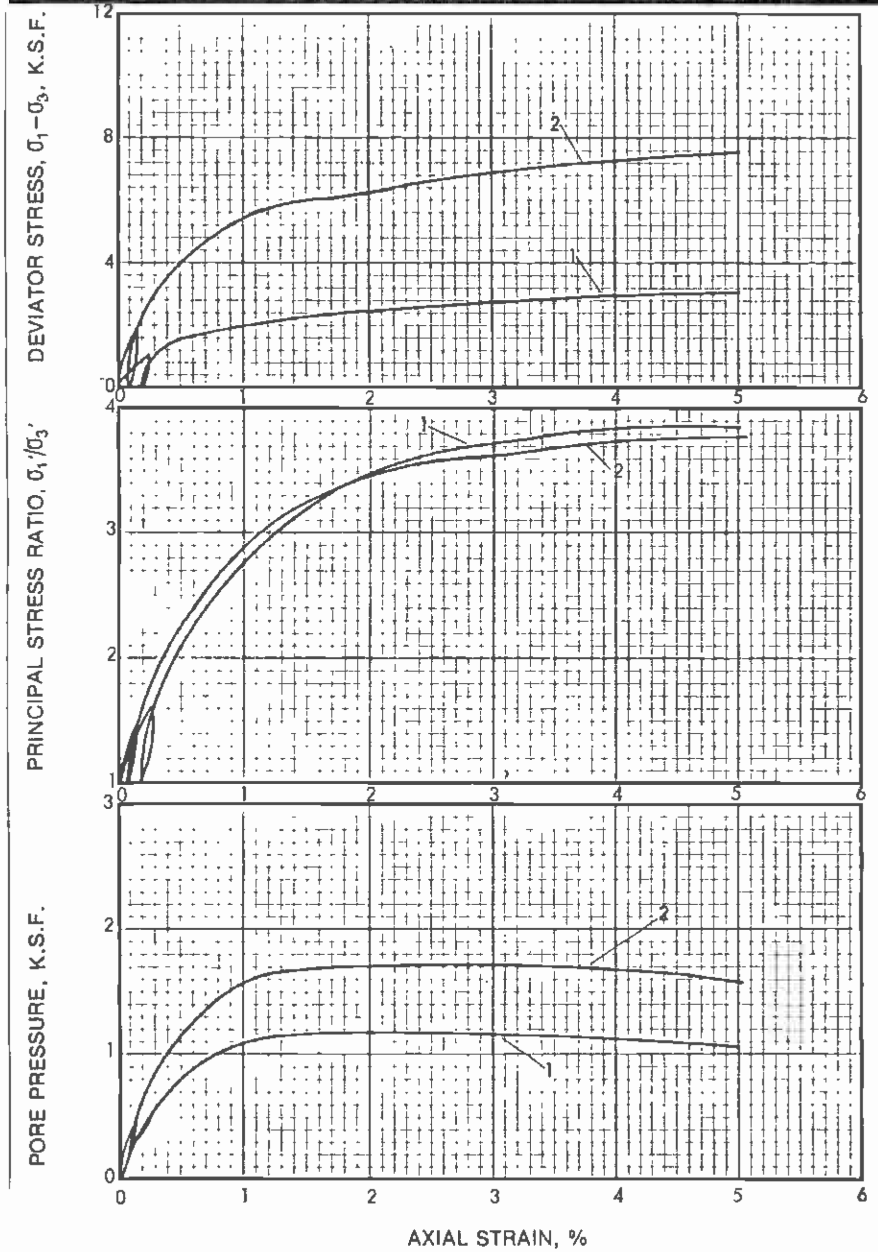

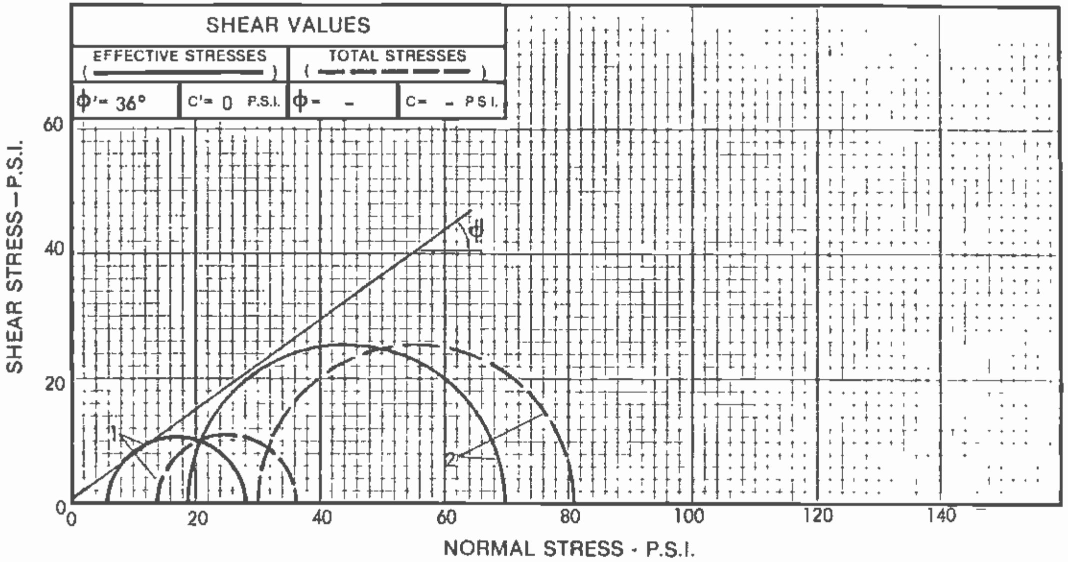

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ'= 36°	C'= 0 P.S.I.	φ= -	C= - P.S.I.

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-1	38-3	PB-1	14.5-17	SM	5.93	2.86	103.5	-	Stage 1
PB-1	38-3	PB-1	14.5-17	SM	5.68	2.90	105.4	18.7	Stage 2
				Initial	6.00	2.85	103.1	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-1	1	15	20.7	7.7	7.3	28.0	Two-Stage ICU with Pore Pressure Measurements
PB-1	2	30	51.1	11.4	18.6	69.7	

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Figure No
D-19

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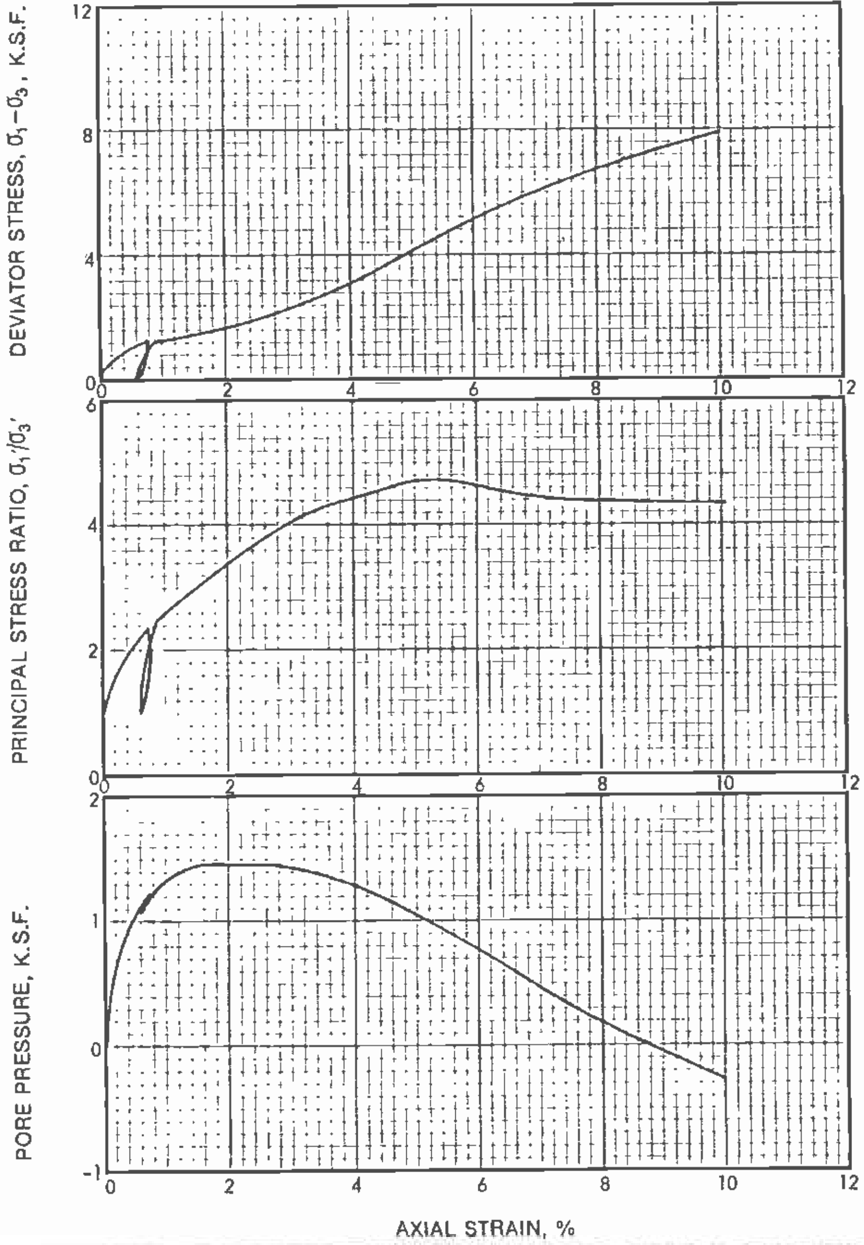

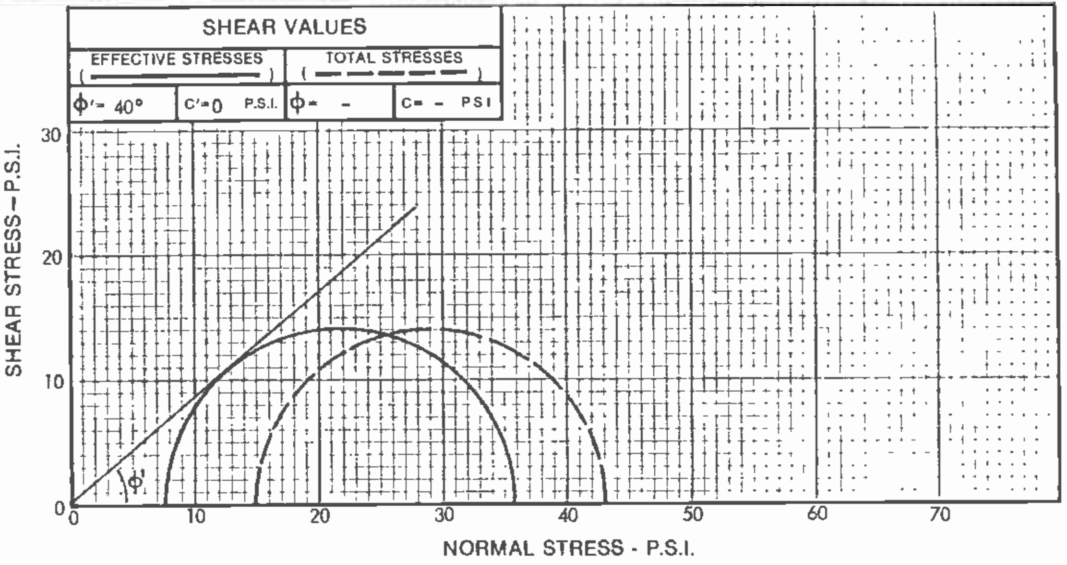

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (– – –)	
ϕ'= 40°	C'=0 P.S.I.	ϕ= -	C= - PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI-FICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-1	38-4	PB-1	14.5-17	SM	5.92	2.84	105.0	20.2	
				Initial	6.00	2.83	104.5	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		15	28.4	7.3	7.7	36.1	ICU with Pore Pressure Measure-ments

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Figure No
D-20

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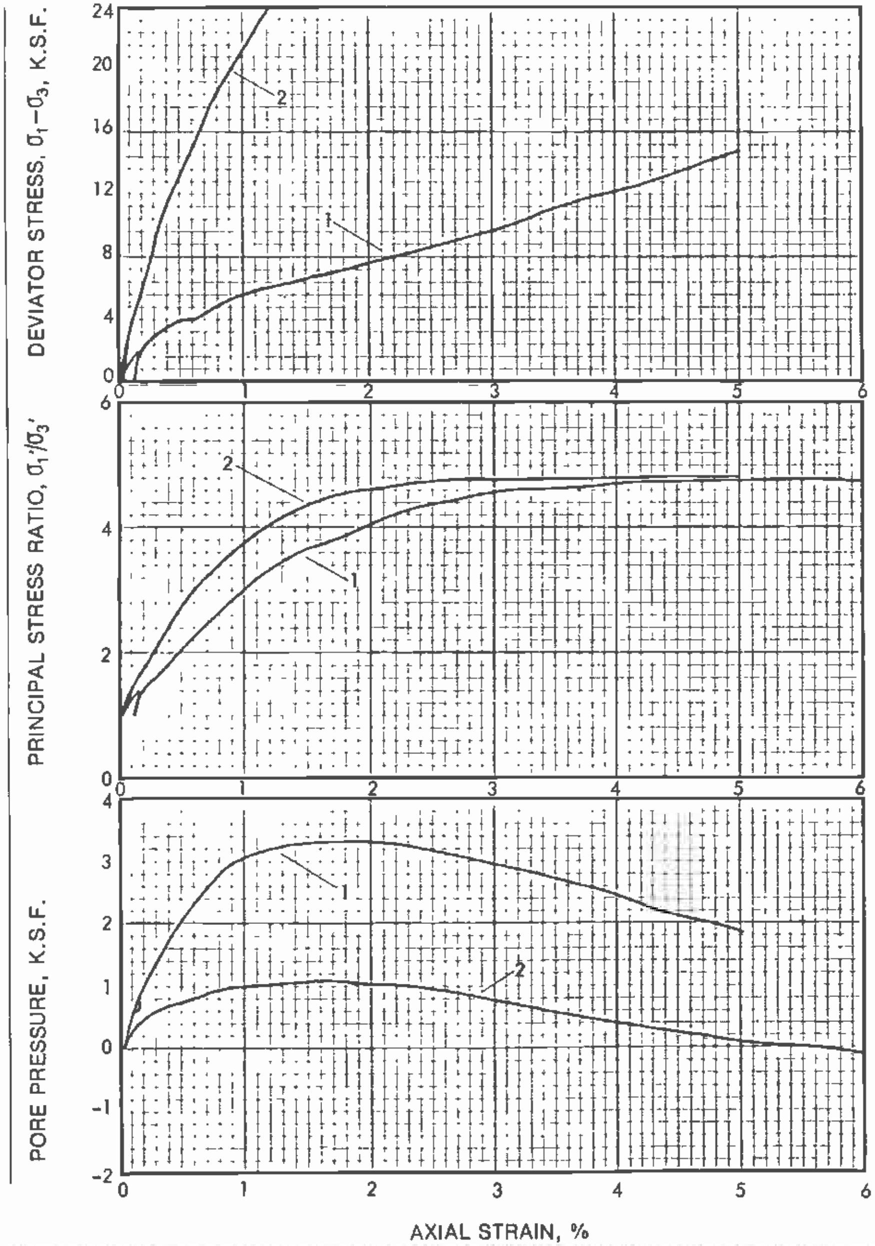

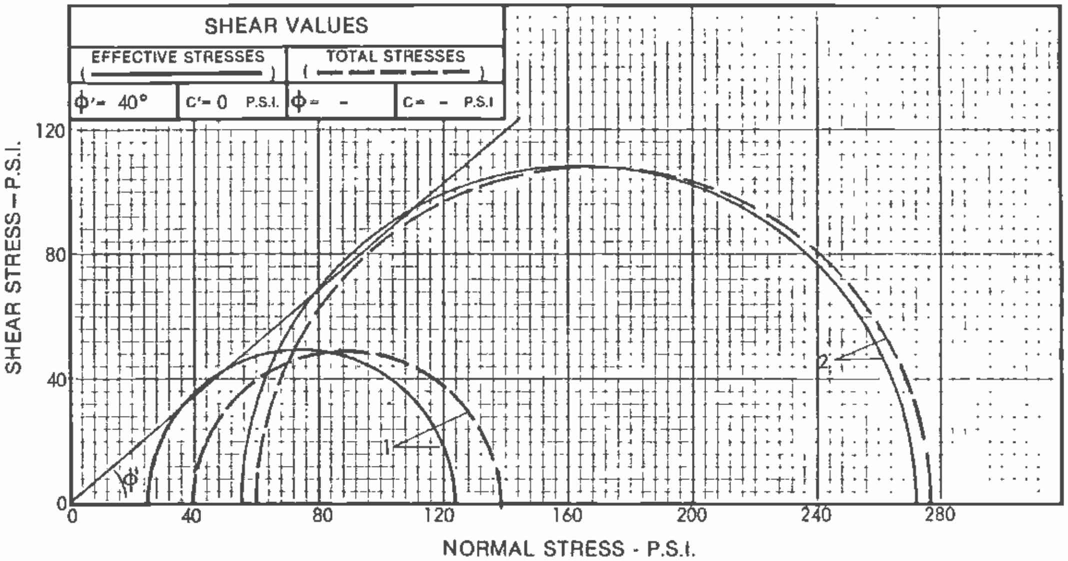

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'= 40°	C'= 0 P.S.I.	ϕ= -	C= - P.S.I

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-4	38-4	PB-4	42-44.5	SP/SM	6.11	2.85	120.2	-	Stage 1
PB-4	38-4	PB-4	42-44.5	w/gravel	5.82	2.89	122.5	13.9	Stage 2
				Initial	6.19	2.84	119.6	10.2	

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-4	1	40	98.7	13.9	26.1	124.8	Two-Stage ICU with Pore Pressure Measurements
PB-4	2	60	216.1	3.0	57.0	273.1	

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Figure No
D-21

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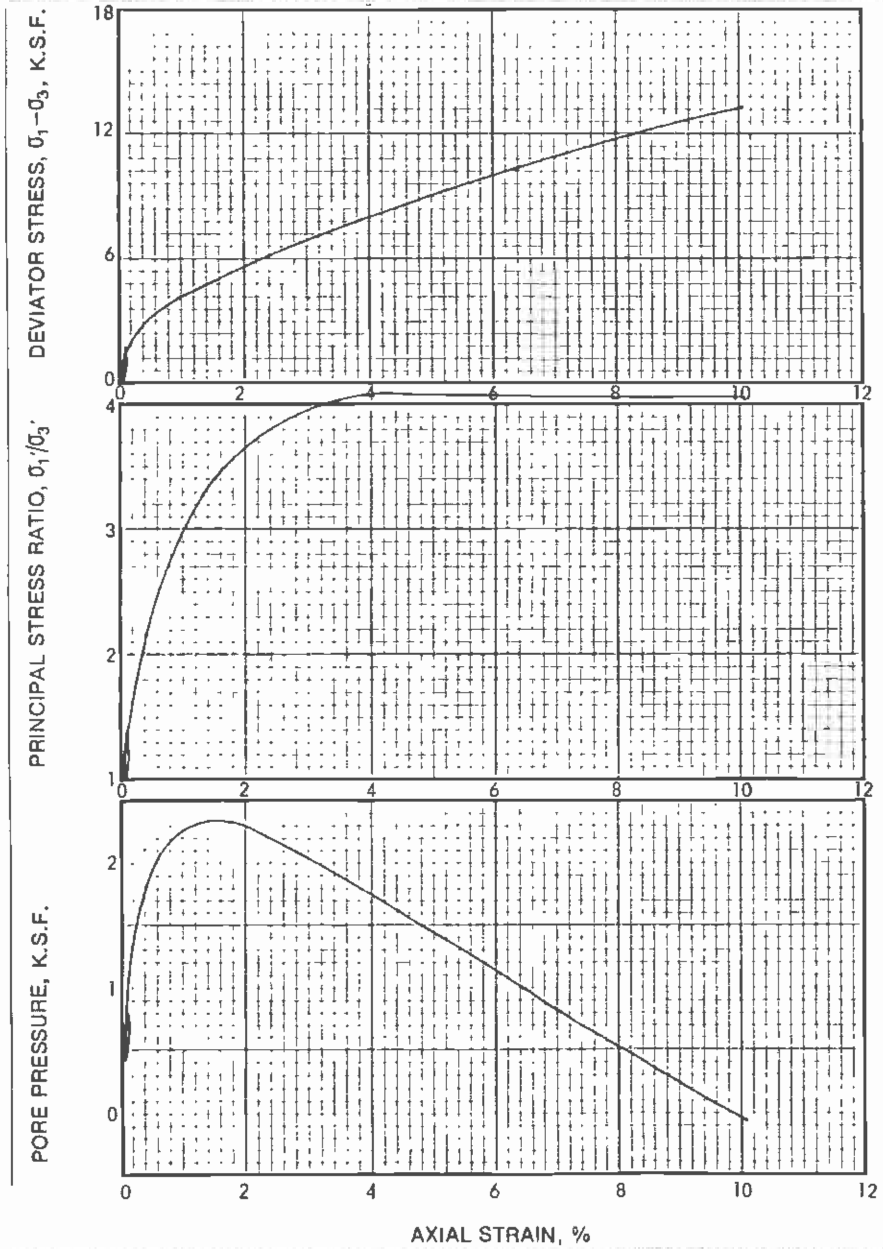

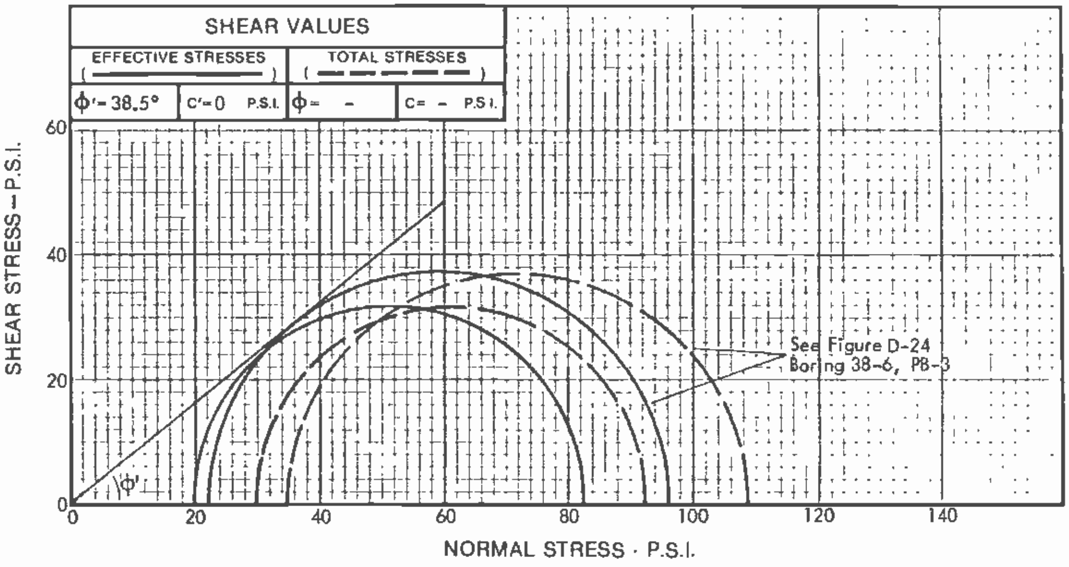

SHEAR VALUES			
EFFECTIVE STRESSES (——————)		TOTAL STRESSES (— — — — —)	
ϕ'- 38.5°	C'-0 P.S.I.	ϕ= -	C= - P.S.I.

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-2	38-5	PB-2	34.5-35.9	SM	5.95	2.87	110.8	17.7	
				Initial	6.00	2.86	110.4	19.1	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-2		30	63.0	9.8	20.2	83.2	ICU with Pore Pressure Measurements

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Figure No
D-22

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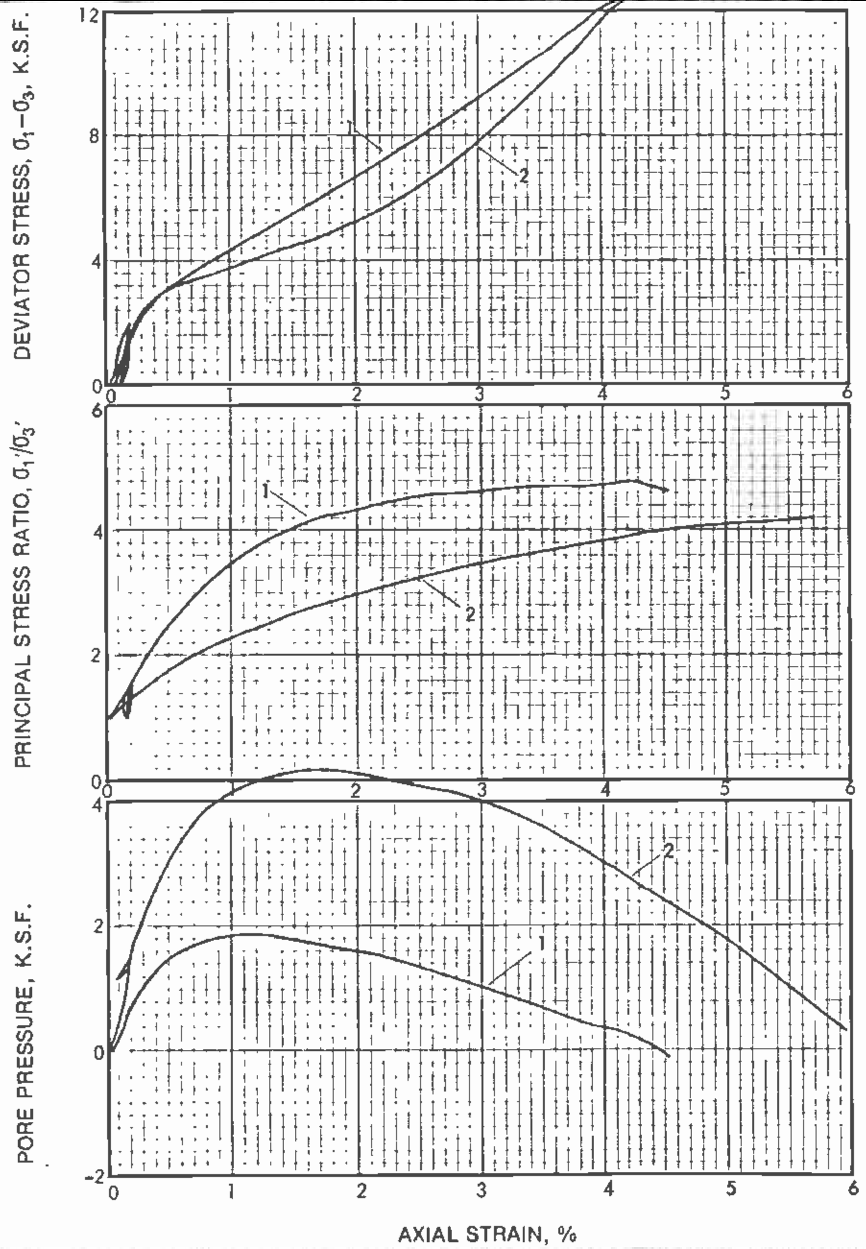

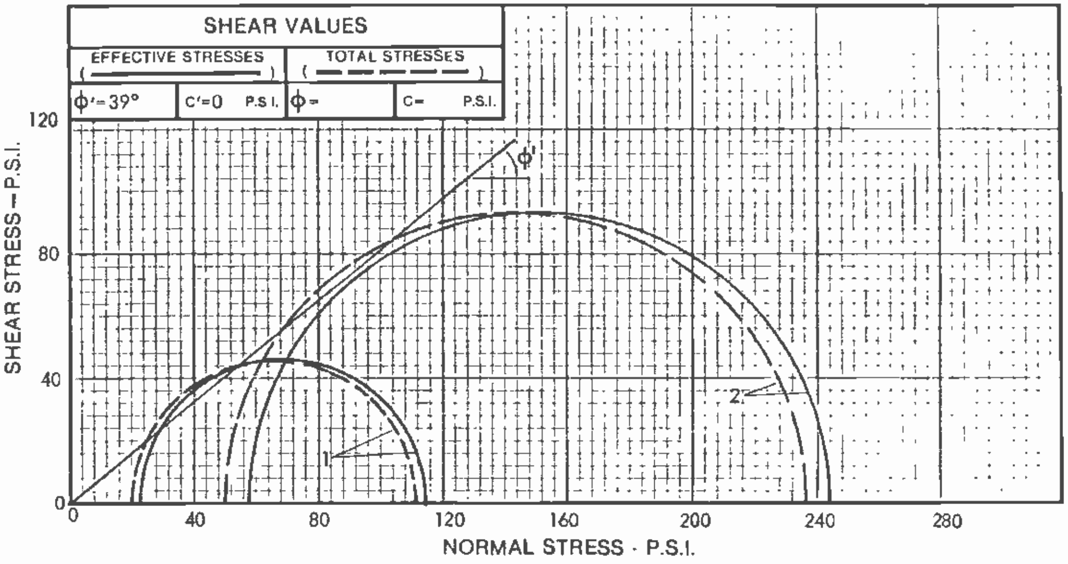

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'=39°	C′=0 P.S.I.	ϕ=	C= P.S.I.

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-1	38-6	PB-1	21-23	SP with	6.07	2.85	109.9	-	Stage 1
PB-1	38-6	PB-1	21-23	gravel	5.66	2.91	113.0	-	Stage 2
				Initial	6.09	2.84	109.7	11.5	

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-1	1	25	87.7	1.9	23.1	110.8	Two Stage ICU with Pore Pressure measurements
PB-1	2	50	187.3	-7.3	57.3	244.6	

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Figure No
D-23

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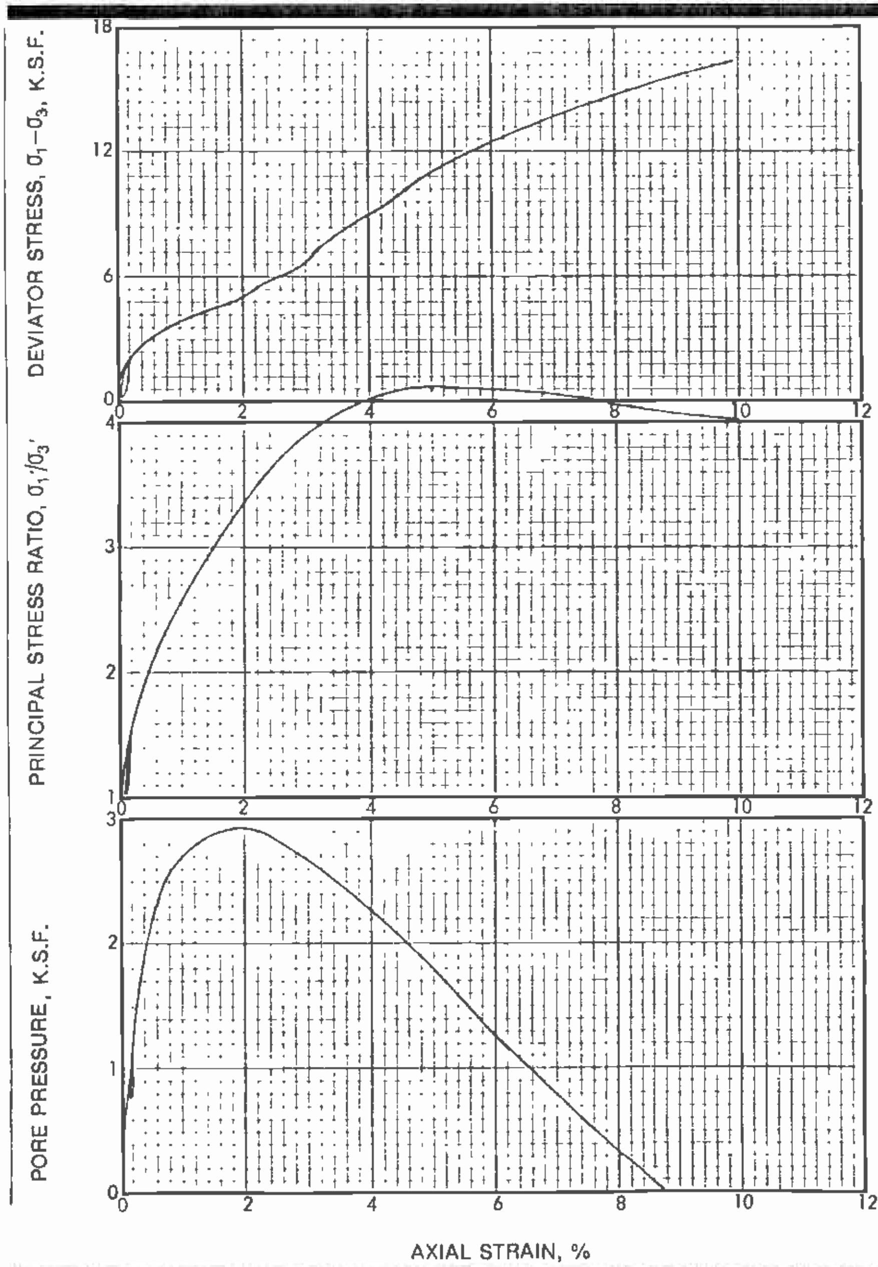

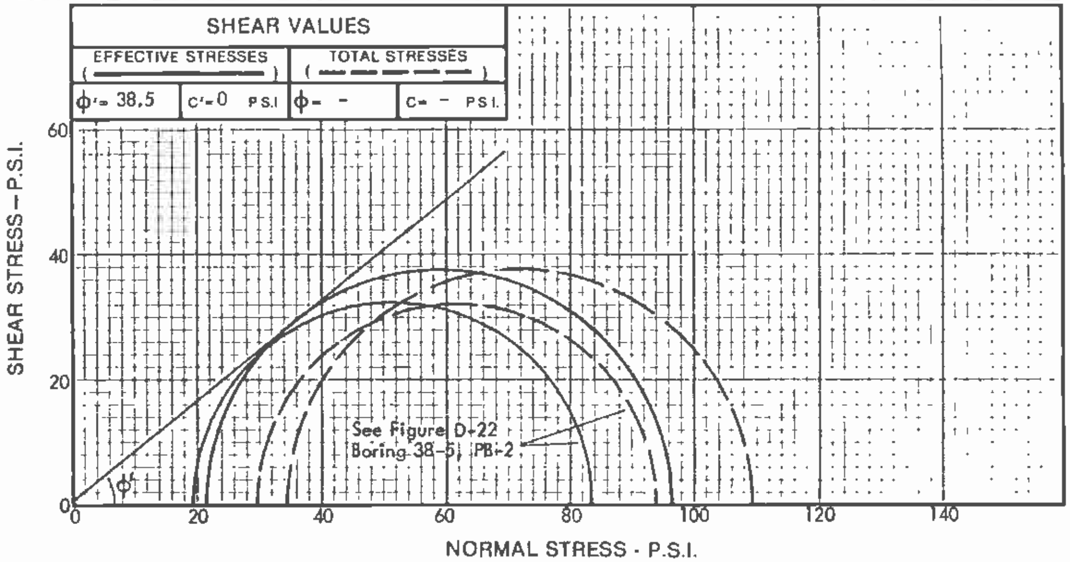

See Figure D-22
Boring 38-5, PB-2

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
φ'= 38.5	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-3	38-6	PB-3	37-39.5	SM/ML	5.92	2.85	105.7	18.4	
				Initial	6.00	2.84	105.1	21.4	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 $\sigma_1-\sigma_3$ (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-3		35	73.9	12.6	22.4	96.3	ICU with Pore Pressure Measurements

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Figure No
D-24

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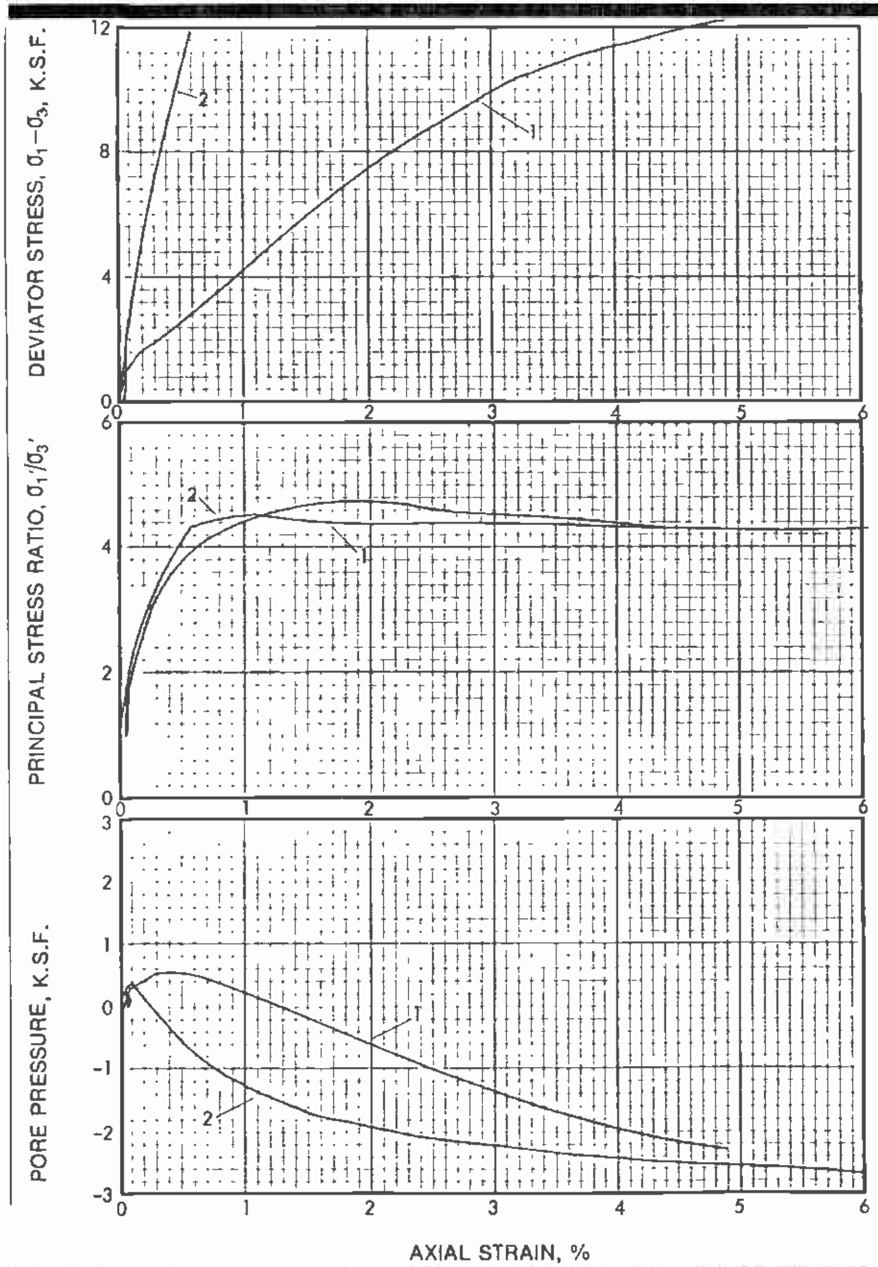

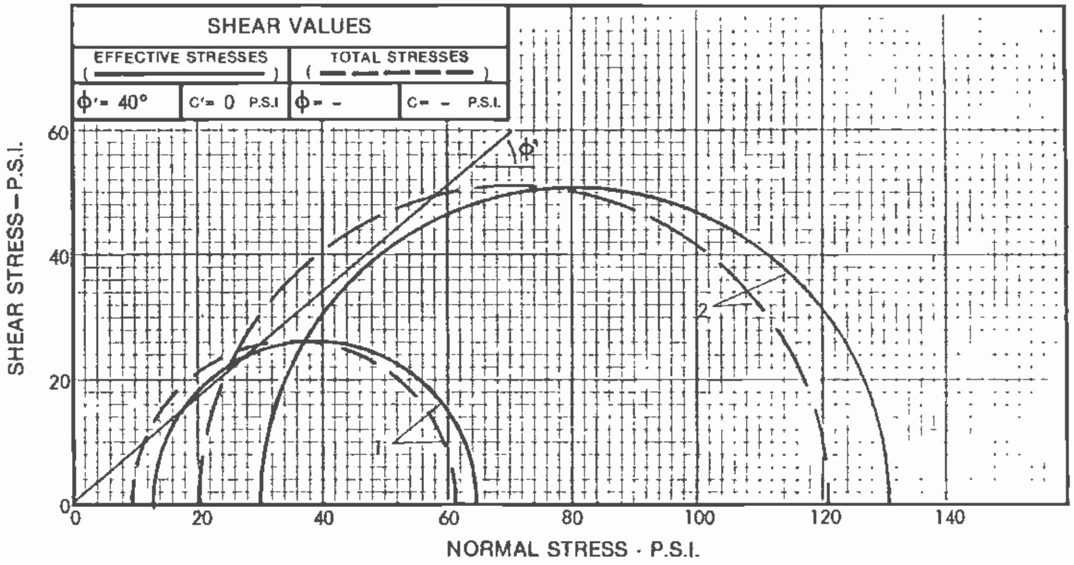

SHEAR VALUES			
EFFECTIVE STRESSES (————)		TOTAL STRESSES (— — — —)	
φ'= 40°	C'= 0 P.S.I	φ= -	C= - P.S.I.

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-1	38-7	PB-1	12.0-14.5	SP/SM	6.12	2.84	102.6	-	Pitcher Undisturbed
PB-1	38-7	PB-1	12.0-14.5	SP/SM	5.84	2.88	104.5	22.1	
				Initial	6.13	2.84	102.5	9.9	

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	1	10	50.6	-3.7	13.7	64.3	Two Stage ICU with Pore Pressure measurements
PB-1	2	20	101.5	-9.6	29.6	131.1	

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Figure No
D-25

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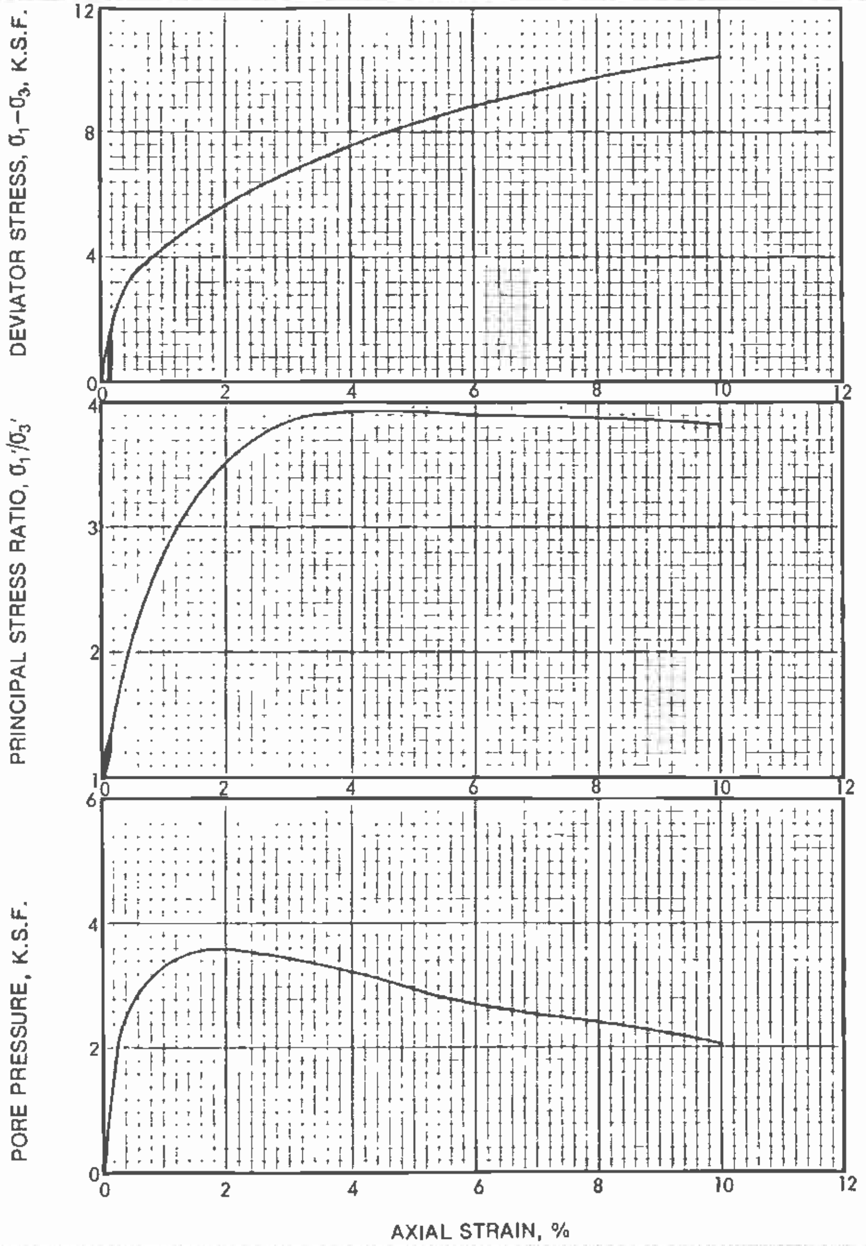
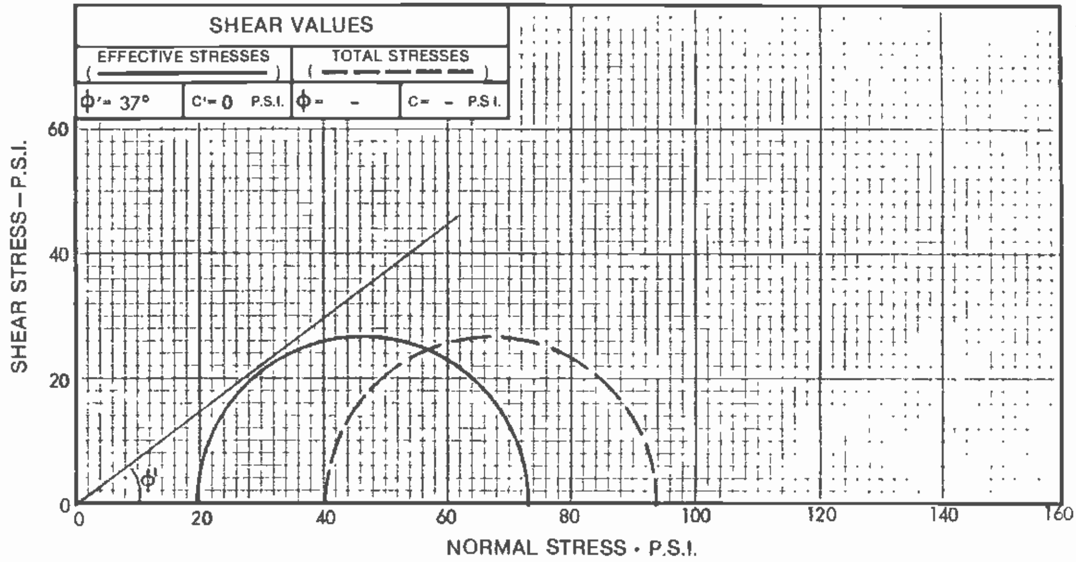




SHEAR VALUES			
EFFECTIVE STRESSES (————)		TOTAL STRESSES (— — — — —)	
ϕ' = 37°	C' = 0 P.S.I.	ϕ = –	C = – P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI FICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-4	38-7	PB-4	41.5-43.6	SM	6.29	2.84	111.4	16.5	Pitcher Undisturbed
				Initial	6.34	2.84	110.8	16.4	

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-4		40	55.0	21.3	18.7	73.7	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

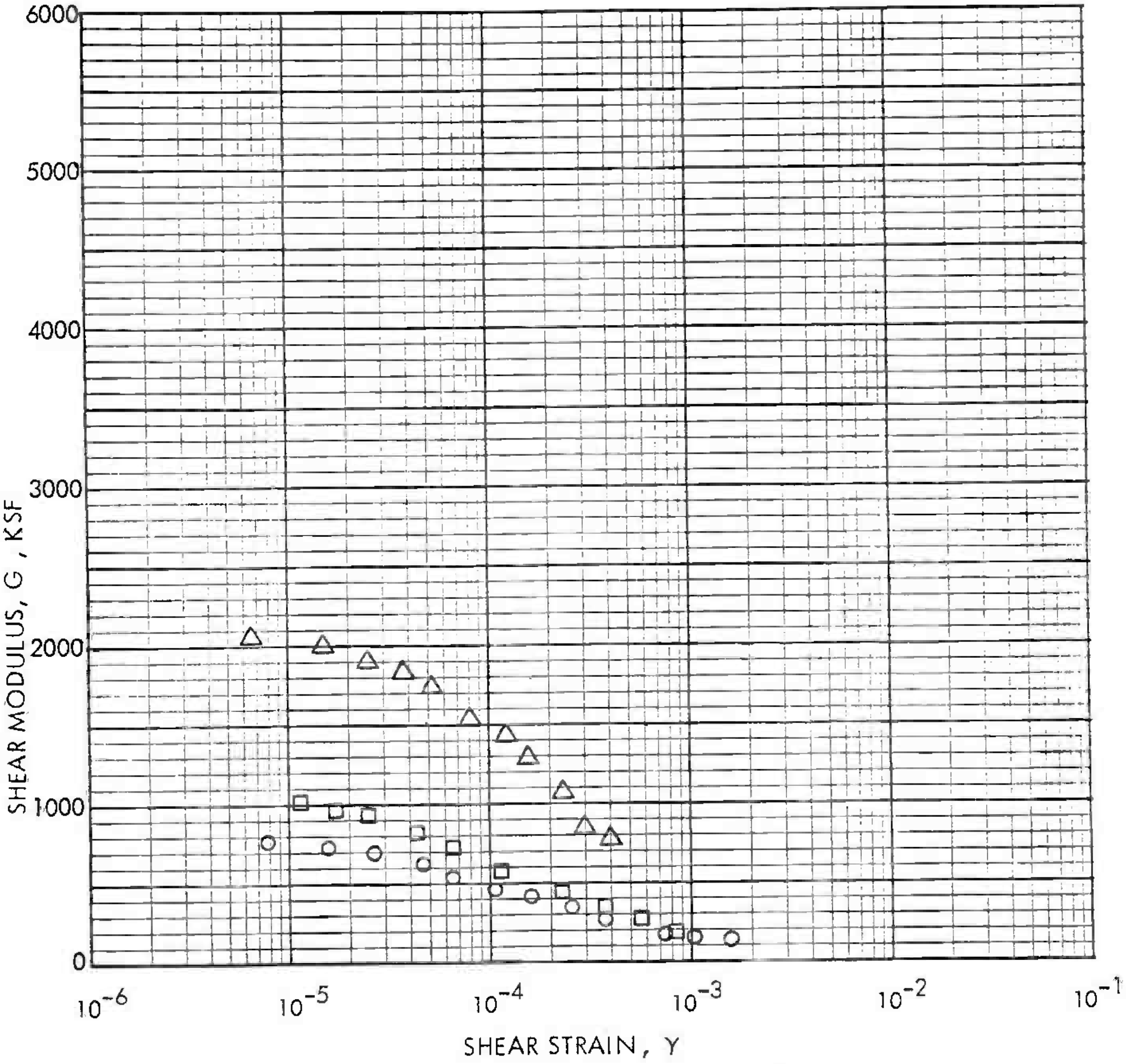
DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No
D-26

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	S-1	108	112	19	15	○
					30	□
					50	△

Sample Description: Brown Fine Sand and Silt; moist, dense

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RESONANT COLUMN TEST

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Project No.
83-1140

Figure No.
D-27

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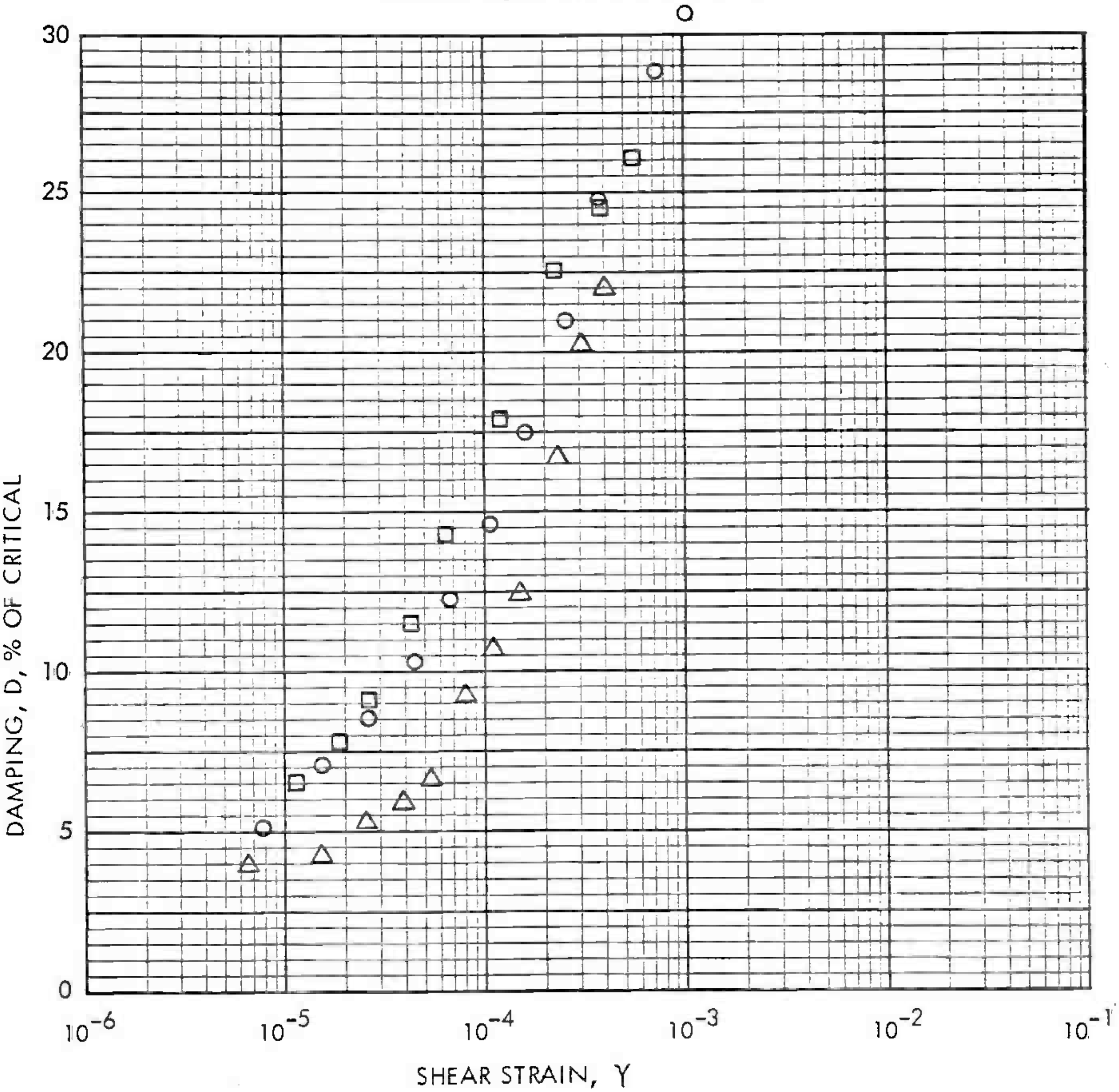
STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	S-1	108	112	19	15	○
					30	□
					50	△

Sample Description: Brown Fine Sand and Silt; moist, dense

RESONANT COLUMN TEST

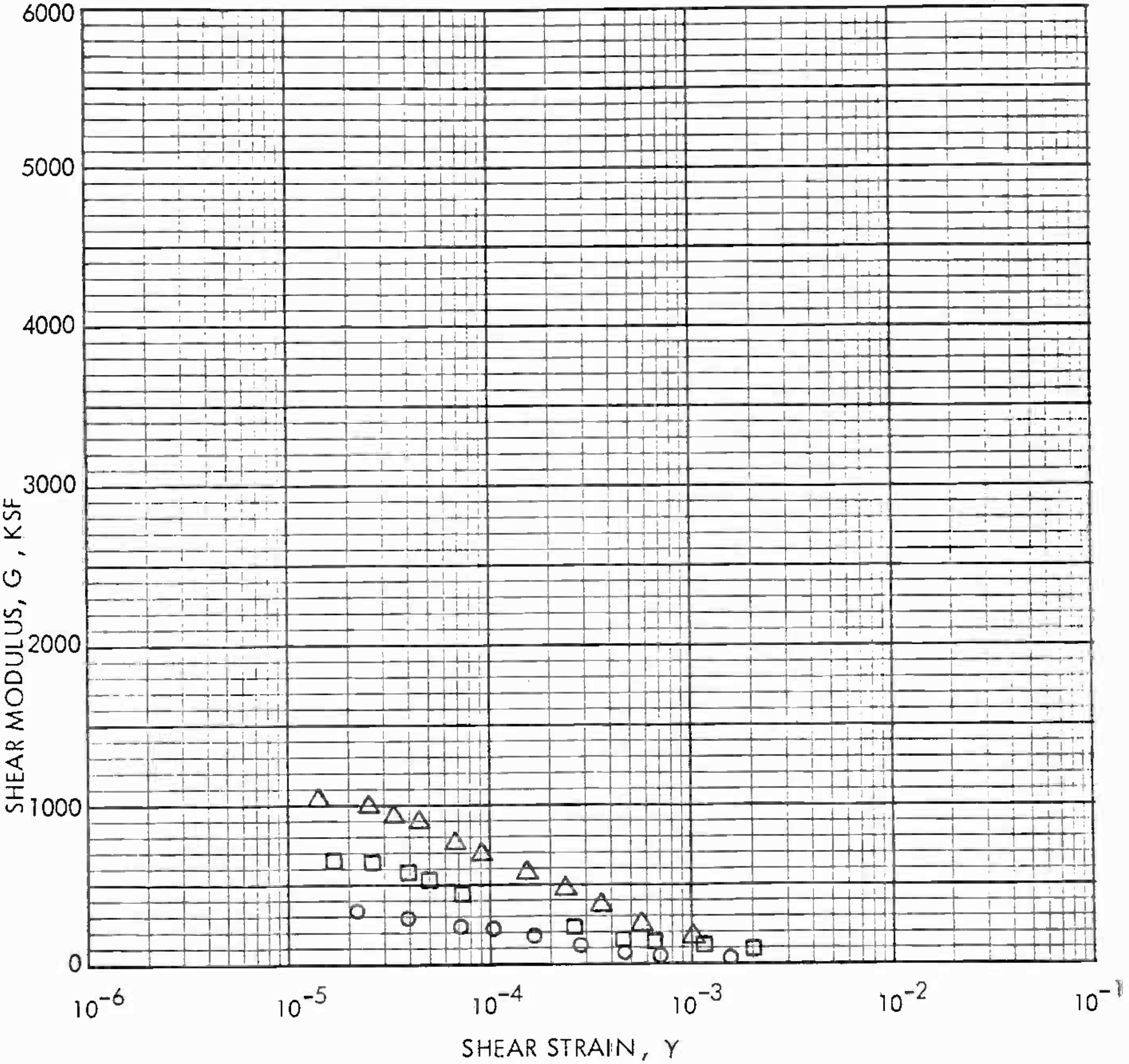
DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
D-28

Converse Consultants Geotechnical Engineering and Applied Sciences

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	S-4	161	115	17	15	○
					30	□
					50	△

Sample Description: Brown Silty Fine Sand; moist, medium dense

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RESONANT COLUMN TEST

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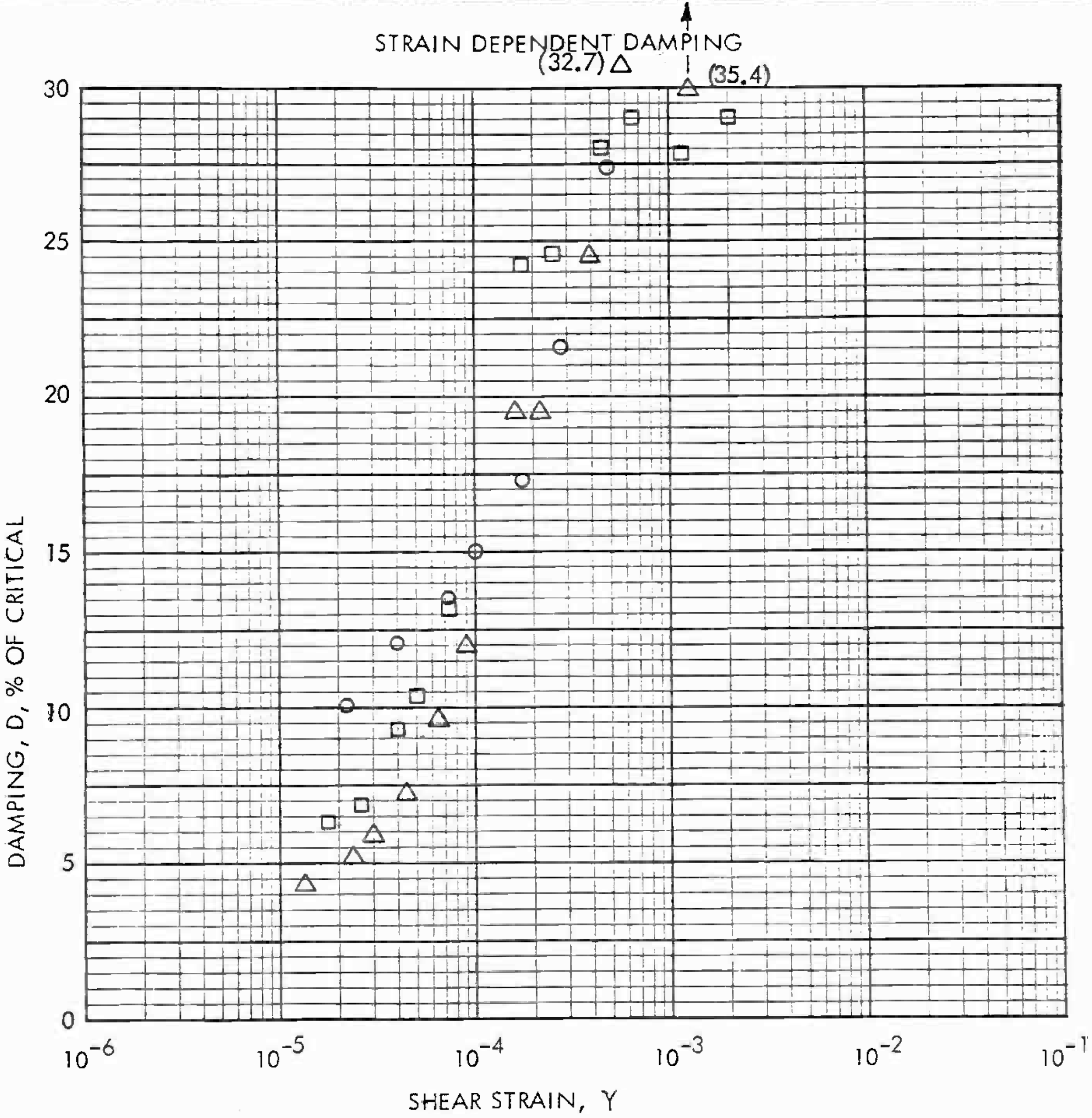
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
D-29



STRAIN DEPENDENT DAMPING

BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	S-4	161	115	17	15	○
					30	□
					50	△

Sample Description: Brown Silty Fine Sand; moist, medium dense

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RESONANT COLUMN TEST

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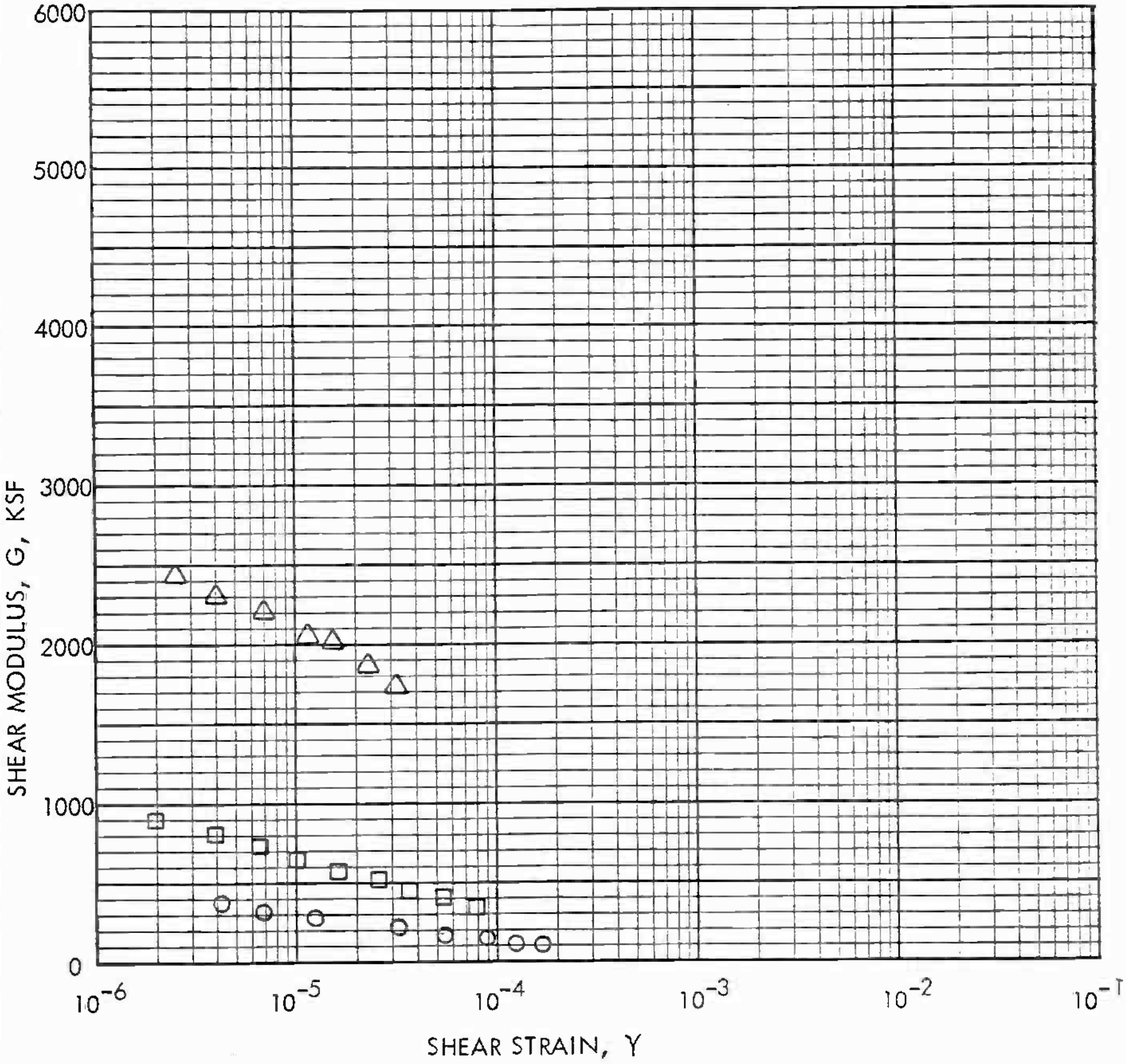
Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
D-30

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	C-16	201	111	14	15	○
					30	□
					50	△

Sample Description: Brown fine to medium Sand, medium dense

RESONANT COLUMN TEST

DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
D-31

Converse Consultants Geotechnical Engineering and Applied Sciences

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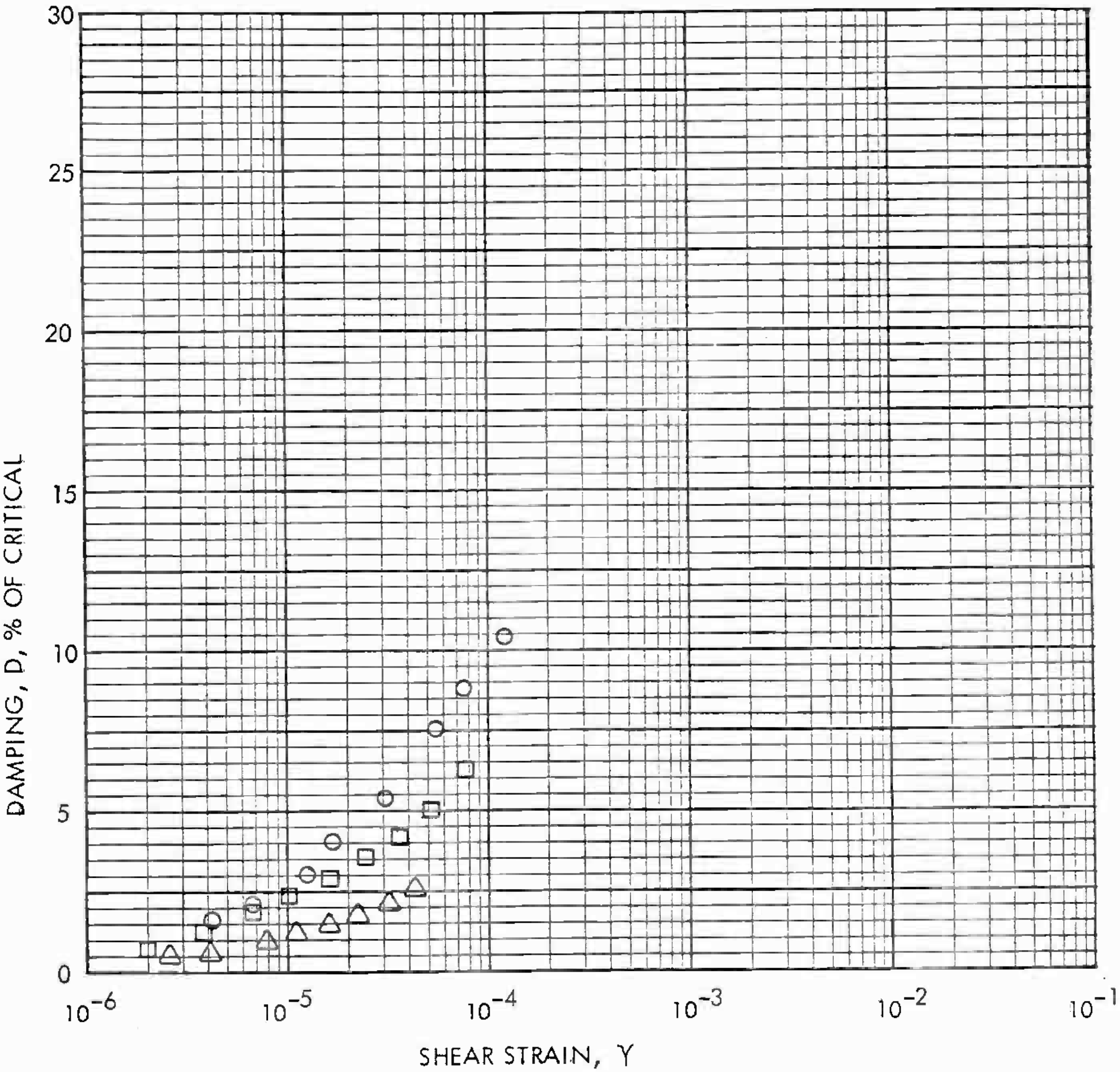
STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
38	C-16	201	111	14	15	○
					30	☐
					50	△

Sample Description: Brown fine to medium Sand, medium dense

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RESONANT COLUMN TEST

DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
D-32

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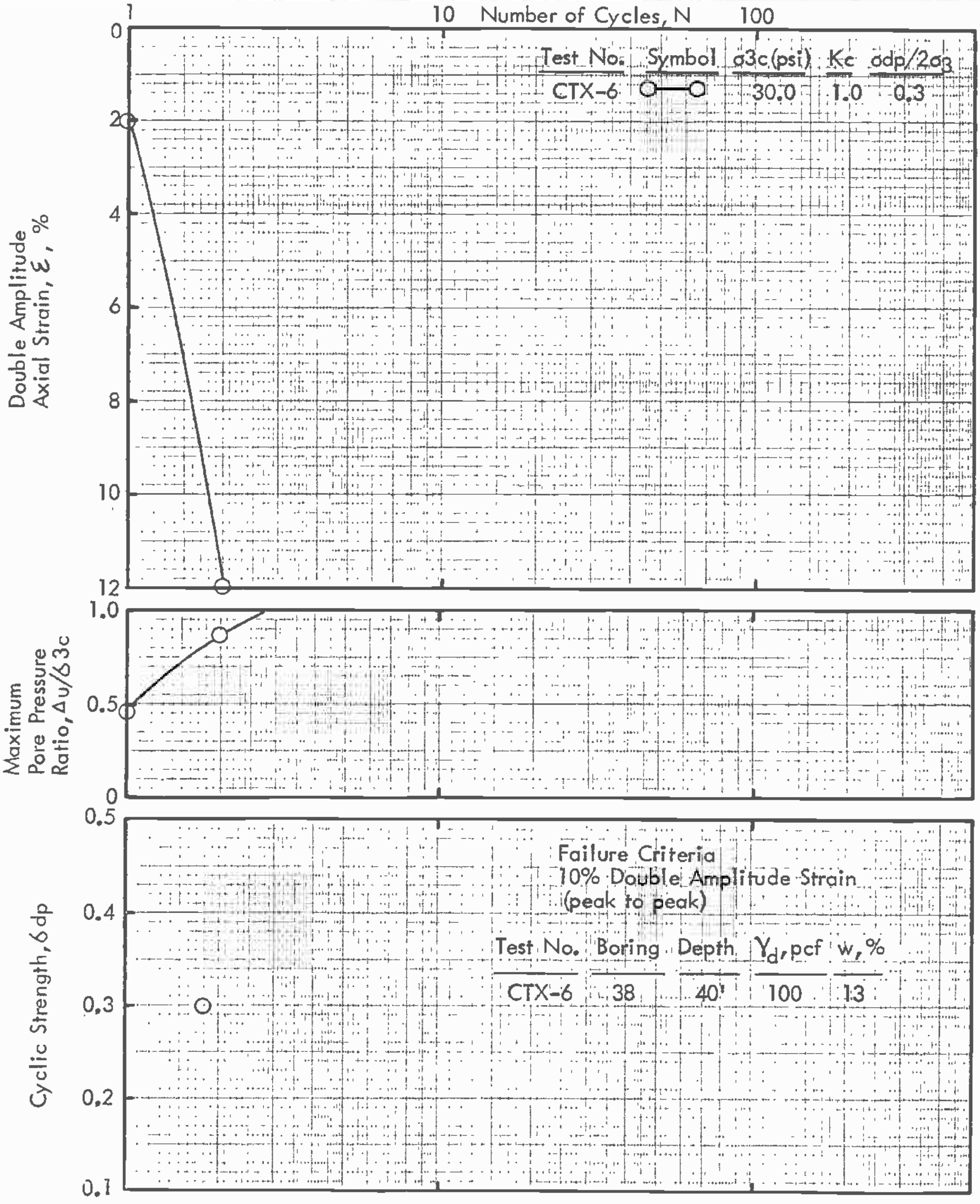

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CYCLIC TRIAXIAL COMPRESSION TEST RESULTS

DESIGN UNIT A445
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
D-33

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Appendix E

Technical Considerations

APPENDIX E: TECHNICAL CONSIDERATIONS

E.1 SHORING PRACTICES IN THE LOS ANGELES AREA

E.1.1 General

Deep excavations for building basements in the Los Angeles area are commonly supported with soldier piles with tieback anchors. Three case studies involving deep excavations in the Los Angeles area are presented below.

E.1.2 Atlantic Richfield Project (Nelson, 1973)

This project involved three separate shored excavations up to 112 feet in depth in the siltstones of the Fernando Formation. The project is located just north of Boring CEG 9, and the proposed location of the 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.

E.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.

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

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

- Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20^{o} to 40^{o}. 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.

E.1.5 Design Lateral Load Practices

Table E-1 summarizes the design lateral loads used for eight shored excavations in the Los Angeles area. 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.

E.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. 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.

Table E-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.

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

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

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

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

where:

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

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

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

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

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

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

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 F

Earthwork Recommendations

APPENDIX F: EARTHWORK RECOMMENDATIONS

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

- <u>Site Preparation (Surface Structures)</u>:

  Existing vegetation, debris, and soft or loose soils should be stripped from the areas that are to be graded. 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.

- <u>Minor Construction Excavations</u>:

  Temporary dry excavations for foundations or utilities may be made vertically to depths up to 5 feet. For deeper dry excavations in existing fill or natural materials up to 15 feet, excavations should be sloped no steeper than 1.5:1 (horizontal to vertical).

- <u>Structural Fill and Backfill</u>:

  Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite 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.

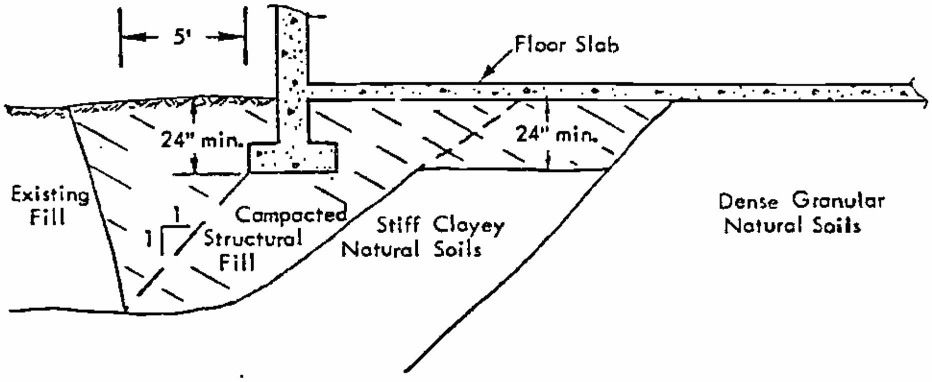
  Where space limitations do not allow for conventional backfill compaction operations, special backfill materials and procedures may be required. Sand-cement slurry, pea gravel or other selected backfill can be used in limited space areas. Sand-cement slurry should contain at least 1-1/2 sacks cement 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.