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Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A430

BY

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

MAY 1984

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May 29, 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 A430 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 A430.

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

Respectfully submitted,

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RMP:j

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PROFESSIONAL CERTIFICATION

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

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

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Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation and engineering analyses for the A430 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A430 Design Unit consists of the North Hollywood crossover and about two miles of tunnel line extending from the Universal City Station to the North Hollywood crossover structure. The crossover will be constructed by cut-and-cover methods and extend in depth up to 55 feet below the existing ground surface. The line between the Universal City Station and North Hollywood crossover will be constructed by tunnelling methods and will have a variable depth of cover above the crowns of the single track tunnels. Construction will occur predominantly in alluvial type soils having variable ground water conditions. The report defines the subsurface conditions and provides recommendations for design and construction purposes.

1.1 CROSSOVER STRUCTURE

Subsurface materials at the crossover site consist of alluvium extending to a depth of at least 200 feet, the maximum depth penetrated by the exploratory boreholes in Design Unit A430. The actual depth of the alluvial deposits in the San Fernando Valley Ground Water Basin, in which the site is situated, may be as great as 1000 feet in some places. In general, the upper 45 to 50 feet of the alluvium consists primarily of sands, silty sand 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 specifically at the crossover site.

In the vicinity of the Crossover Structure, the ground water table is about Elevation 490 (or about 140 feet below the ground surfaced). Ground water and/or seepage was not encountered in the two large-diameter boreholes drilled in the vicinity of the site.

Construction of the crossover on Lankershim Boulevard will consist of an excavation approximately 425 feet long, 60 feet wide, and up to 55 feet deep. The crossover excavation will occur entirely within alluvial type soils as discussed above. Temporary support of the crossover excavation will be either flexible or rigid type vertical wall systems with internal bracing or external tieback systems. Successful installation of tiebacks will require certain precautions to maintain the stability of such borings in the granular soils. Lateral pressures and other guidelines for design of temporary support systems are provided in the report.

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

1.2 TUNNELS

The average depth of ground cover above the crown of the tunnels in A430 is about 60 feet, varying between a minimum of 20 feet at the Los Angeles River and a maximum of 75 feet near Station 990+00. The tunnel is below the known water level in the alluvium between Universal City Station and about Station 970+00 and above the water level from about Station 970+00 to the south end of the North Hollywood Crossover Structure. Tunnel excavations may encounter gravel/cobbles, sometimes up to 12 inches in diameter although infrequently, and their presence should impact the type of equipment selected and possibly the rate of excavation progress.

Upon leaving the Universal City Station the tunnel line will pass through the Topanga formation bedrock; however, the tunnel crown may encounter some mixed-face conditions for a distance of approximately 3500 feet north of the Universal City Station. The alluvial materials at the mixed face may consist of saturated gravels, sands, silts and clays overlying soft Topanga siltstone, claystone and sandstone materials. The ground water level above the invert varies between 50 feet at the north end of the Universal City Station to zero near Station 970+00. It is anticipated that flowing ground conditions may be encountered at the crown and face of the tunnels if dewatering systems are not in place or operating properly.

The tunnels between Station 970+00 and the Crossover structure will encounter heterogeneous alluvial materials consisting of interbedded horizons of unsaturated cohesive and cohesion-less materials with variable distribution of occurrence over the face of the tunnels. The ground water level is believed to be entirely below invert in this tunnel segment. Therefore, this tunnel segment should not encounter flowing ground conditions.

We believe that the soil and bedrock conditions of Design Unit A430 are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavation equipment. Because of the mixed-face conditions, nature of the soil and ground water conditions, we do not believe that tunnel construction without a shield will be successful in this segment of the tunnel. Shield tunnelling methods will require means for the utilization of fore polling and/or breast boarding techniques to maintain stability of the face, prevent loss of ground and avoid surface settlement along the alignment. The presence of gravel/cobbles up to 12 inches in diameter, although not preeminent, should be anticipated and may well dictate the type of mechanical excavation equipment as well as rate of which excavation can be made through "cobbly" horizons.

Only one, unnamed, postulated fault is known to cross the tunnel line (near Station 987±). It is not known to be active or potentially active nor does it act as a ground water barrier. This fault is expected to have little or no effect on tunneling excavation. However, the contractor should anticipate encountering other small faults and shear zones.

No strong or unusal odors were detected during the drilling and logging of the borings located along the tunnel alignment. Design Unit A430 is not located in an oil-producing area nor near known oil fields.

The cross passages between tunnels may encounter saturated, interlayered horizons of cohesionless-like soils. The cross passages should be excavated by hand and/or mechanical excavation equipment with appropriate support, exercising precautions similar to those noted for tunnel construction.

1.3 UNDERPINNING

Guidelines for assessing the need for underpinning of buildings adjacent to the Station construction and along the tunnel alignment 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.

1.4 SEISMIC CONSIDERATIONS

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the 1984 SCRTD report entitled "Guidelines for Seismic Design of Underground Structures". Seismological conditions which may impact the project and the operating and maximum design earthquakes which may be anticipated in the Los Angeles area are described in the SCRTD report entitled "Seismological Investigations and Design Criteria" dated May, 1983. The 1984 report complements and supplements the 1983 report. Site specific static and dynamic properties for materials in Design Unit A430 are given in the report.

Liquefaction potential at the crossover structure is considered nil due to very deep ground water levels. The potential for liquefaction along the tunnel alignment is considered to be low based on the fact that, within the southern segment which is below the ground water level, the tunnel will pass through bedrock and dense deep alluvium.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A430. The unit consists of about two miles of subsurface track line proceeding north and east from the north end of the Universal City Station to the south end of the North Hollywood Station and includes the crossover structure which adjoins the North Hollywood Station. Also included in this Design Unit is a mid-line vent structure to be located near the Blix Street crossing. The work performed for this report includes borings, laboratory tests, engineering analysis, and the development of recommendations and specifications for design and construction of the crossover structure and the tunnels. This Design Unit is a part of the 18.6-mile long Metro Rail Project (see Drawing 1, Vicinity Map).

Additional geotechnical information on the Metro Rail Project is included in the following reports, some of which may pertain to Design Unit A430.

- "Geotechnical Investigation Report, Metro Rail Project", Volume I - Report, and Volume II - Appendices, prepared by Converse Ward Davis Dixon, Earth Sciences Associates and Geo/Resource Consultants, submitted to RTD in November 1981. This report presents general geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experience and practices in the Los Angeles area.
- "Geotechnical Report, Metro Rail Project, Design Unit A425", prepared by Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in May, 1984. This report presents our results of the findings for the Universal City Station.
- "Geotechnical Report, Metro Rail Project, Design Unit A445", prepared by Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in May, 1984. This report presents our results of the findings for 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 RTD in May 1983. This report presents the results of a seismological investigation.
- "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF866, 1977), prepared by the U.S. Geological Survey in cooperation with the U.S. Department of Transportation. This publication includes a compilation of geotechnical data in the general vicinity of the proposed Metro Rail Project and this Design Unit.

The design concepts discussed in this report are based on Drawings AP-16AAA-C--19 through AP-16AAA-C-21, Definitive Fixed Facilities Plans, Alignment Plan and Profile dated September, 1983; and Drawings AP-16AAA-C-201 and AP-16AAA-C--471, CBD to North Hollywood Line, Plan and Profile both dated July, 1983.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

3.1 GENERAL

The existing ground surface elevations along the A430 alignment vary from about 575 feet at the north end of the Universal City Station to a low point of about 528 feet at the Los Angeles River then rising to the highest elevation of about 628 feet at the north end of the North Hollywood crossover structure. The southern most 1500 feet of the tunnel will pass beneath private property, however, there does not appear to be any major structures along the alignment in that area and the depth of cover over the tunnel crown ranges from 40 to 55 feet. The remainder of the alignment runs beneath Lankershim Boulevard, a major thoroughfare underlain by a variety of utilities and drainage facilities.

The north end of the A430 alignment will include the North Hollywood crossover structure which will be constructed by cut and cover methods. The depth to the crossover structure subgrade is approximately 53 and 55 feet at the southern and northern ends, respectively.

3.2 NORTH HOLLYWOOD CROSSOVER

The North Hollywood crossover site will be located beneath Lankershim Boulevard spanning Weddington Street (see Drawing 5). The development along the west side of Lankershim at the structure location consists of low-rise commercial structures. The structures along the east side of Lankershim are to be removed to provide surface parking for 1180 autos with possible future construction of a 2500-space parking structure. The existing ground surface along Lankershim Boulevard varies from about Elevation 625 at the south end of the structure to about Elevation 628 at the north end.

The North Hollywood Crossover will be a reinforced concrete structure about 425 feet long and 60 feet wide (outside wall dimensions). The top of rail varies from about Elevation 579 feet at the south end to about Elevation 580 feet at the north end of the structure. Assuming the structure will be supported on a 4- to 6-foot thick concrete mat, the station area will require an excavation to approximately 53 feet below the existing grade at the south end of the structure, and 55 feet below the existing grade at the north end of the structure. After the structure is constructed, approximately 4 to 9 feet of fill will be placed above the structure end areas, and about 25 feet of fill will be placed above the middle portion of the structure. Design loads for the crossover structure were not available at the time of this report.

3.3 TUNNEL ALIGNMENT

As shown on Drawings 2, 3, and 5, the tunnel line in Design Unit A430 starts at approximately Station 935+50 and ends at approximately Station 1043+00. The tunnel proceeds in a northeast direction from the north end of the Universal City Station and enters into a northward curve to enter the Lankershim Boulevard right-of-way just north of the Los Angeles River. From that point, the tunnel continues northward directly under Lankershim Boulevard until it reaches the south end of the North Hollywood Crossover structure.

The construction features about two miles of twin bore tunnels, having an outside diameter of approximately 19 feet. The minimum depth of cover is approximately 20 feet and occurs at the Los Angeles River undercrossing. Maximum depth of cover approaches 75 feet. A mid-line vent structure is planned near the Blix Street undercrossing.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The information presented in this report is based primarily on the field and laboratory investigations performed during the period 1980 to 1984. This information was derived from field reconnaissance, borings, geologic reports and maps, ground water measurements, field gas measurements, field geophysical surveys, ground water quality tests, and laboratory tests on soil and rock samples. References listed at the end of this report were utilized to complement and supplement the more recent information.

4.2 BORINGS

For the A430 investigation, 13 borings were drilling along the alignment and at the crossover structure. Ten borings were drilled along the alignment: five rotary wash borings numbered 34-D, 35-A, 36-B, CEG-37 and CEG-38; and five man-size bucket auger borings numbered 34-C, 35-B, 36-A, 37-A and 38-A. Three rotary wash borings numbered 38-1 through 38-3 were drilled within the crossover site. The location of the borings are shown on Drawings 2, 3, 4 and 5 and logs of the borings are provided in Appendix A. Ground water observation wells were installed in Borings 34-D, 34-C, 35-A, CEG-37 and CEG-38. Section 5.3 presents a summary of ground water level measurements in these wells and at other borings within and near Design Unit A430.

Information pertinent to this design unit was also obtained from borings for the Universal City Station (Design Unit A425) and the North Hollywood Station (Design Unit A445). These borings are identified as 34-5 and 38-4 through 38-6. In addition, one boring drilled by Woodward-Clyde Consultants (Boring WC-11) was considered. Logs of these borings are also included in this report, and their locations are presented on "Location of Borings and Geologic Sections", Drawings 2, 3 and 4.

4.3 GEOPHYSICAL MEASUREMENTS

Downhole compression and shear wave velocity surveys were performed in Boring CEG-38 which was drilled during the initial 1981 investigation. The CEG-38 boring was drilled about 550 feet east of the North Hollywood Crossover structure (see Drawing 4). Appendix B summarizes the field survey procedures as well as the results of the velocity measurements.

4.4 GEOTECHNICAL LABORATORY TESTING

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

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

4.5 PUMP TEST

A pump test was performed north of Bluffside Drive as shown on Drawing 2. The well was 12 inches in diameter (I.D.), 63 feet deep, and perforated below the ground water table within the depth interval of 22 to 33 feet. Two 4-inch diameter observation wells were installed to evaluate water level drawdown at distances of 66 and 166 feet east of the test well.

The test well was pumped initially at a discharge rate of 30 gpm for about 11.5 hours. Following a recovery period, a second test was performed also at a discharge rate of 30 gpm for about 8 hours. Appendix D provides a report of the pump test procedures and results.

4.6 WATER QUALITY ANALYSES

Chemical analyses were performed and selected parameters were evaluated for water samples obtained in Borings CEG-35, CEG-36, CEG-37, CEG-38 and 35B. The chemical analyses and results of these tests are presented in Appendix E.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 NORTH HOLLYWOOD CROSSOVER

Drawing 4 shows a general geologic profile through the Crossover site. Drawing 6 shows a more detailed subsurface profile through the Crossover structure and the contiguous North Hollywood Station which is not part of Design Unit A430.

Alluvium extends to a depth of at least 200 feet, the maximum depth penetrated by nearby 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 Ground Water Basin, in which the site is situated, has sediments which reach depths of up to 1000 feet in some places.

Our interpretation of the subsurface conditions at the Crossover site is shown in Drawing 6. In general, the upper 45 to 50 feet of the alluvium consists primarily of sands, silty 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 located north of the station which will be the end of the line for this segment of the SCRTD Metro Rail Project. Specific descriptions of the various soils are as follows:

- Upper Sands: Within this sandy unit, the materials are predominantly silty sands, some clean fine to coarse sands, and gravelly sands. Some of these soils contain scattered cobbles or small boulders. 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 form 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 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 Gravelly Sand and Sandy Gravels: The alluvium below a depth of about 45 to 50 feet consists primarily of gravelly sands and sandy gravels. 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 (Borings 38A and 38B) and on the drilling action noted in the logs of the rotary-wash borings suggest that the soils of this unit graded 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 non-representative. Some minor belling or sloughing occurred in these soils during the drilling of the large-diameter boreholes, but this was due to the relatively high percentages of gravels and cobbles and vibrations caused by the drilling. Based on this observed behavior, the materials which make up this gravelly unit are judge to be medium dense to dense.

During the drilling of the rotary-wash borings at the 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. However, sampling in Borings 38-1 and 38-2, located at the Crossover site, and in Boring 38-7, located north of the Station in the tail track segment, did not experience such problems.

The large-diameter borehole, Boring 38A, which was drilled just south of the Crossover structure, experienced some minor ravelling between the depths of 10 and 14 feet and significant caving below 50 feet. The log of the other large-diameter hole, Boring 38-B, drilled at the extreme northern end of the tail track, indicated that the hole stood up well with no caving from the ground surface to a depth of 50 feet. Based on the above information, it is possible that caving soils will be randomly encountered during excavations required at the Crossover site.

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 (Figure 7-6).

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.

5.2 TUNNEL ALIGNMENT

Only a small percentage of the A430 tunnel line, leaving from the north end of the Universal City Station will occur in weak bedrock of the Topanga Formation. The remainder of the tunnel will be in Alluvium and will usually be above the water table. Section 7.0 describes the geotechnical and tunnelling conditions for this design unit. Geologic units along the tunnel alignment (Drawings 2, 3 and 4) are described below:

- Alluvium: Alluvium consists of a heterogeneous mixture of sand, gravel, silt, clay and cobbles, listed in order of decreasing occurrence. The granular materials are primarily dense with low compressibility. The fine-grained alluvium is generally stiff to very stiff. We believe the alluvium will flow at the face of the tunnel when excavation occurs below the water table (Stations 950 to 970). The discussion of the "upper sands" and "lower gravelly sand and sandy gravels" presented in Section 5.1 are considered applicable to granular soils anticipated along the tunnel segment. In addition to the granular soils there are significant layers of fine-grained materials, as described in Section 7.0 of this report.

- Topanga Formation: Soft bedrock of the Topanga Formation consists of well stratified claystone and siltstone with interbeds of sandstone. The Topanga Formation often is referred to as "bedrock" or "rock" in various other publications and in places within this report, but it has the engineering properties of hard or dense soils with significant cohesive strength. Hence, the Topanga Formation, in Design Unit A430, is classified as "soil-like" bedrock or "soft ground" tunneling material. Based on surface outcrops located in the hillside about 700 to 1,000 feet southeast of the Universal City Station, bedding planes strike northwesterly, with attendant dips of 32° to 60° northward. These dips corresponds to unoriented bedding plane dips recorded in Borings 34-5 and 35-D near the Universal City Station.

5.3 GROUND WATER

A ground water contour map for the San Fernando Valley Basin by the Los Angeles Flood Control District (LACFCD), 1974 (see Figure 4-13 of the 1981 geotechnical report), indicates that regional ground water flows northwesterly. Table 5-1 presents ground water levels measured in piezometers and man-sized auger borings within the limits of A430.

TABLE 5-1

GROUND WATER OBSERVATION WELL DATA

	GROUND WATER ELEVATION*					
	1977	1981	1982	1983		1984
BORING	DEC.	JUNE	APRIL	JAN.	OCT.	MARCH
34-5						550
34D						539
34C				531		
WC11	518					
35A						515
35B					Dry**	
36B						Dry
36A				Dry**		
37			488			
37A					Dry**	
38A					Dry**	
38		490				
38-4						Dry
38-6						Dry

* Rounded to the nearest foot

** No piezometer installed; water level measured during drilling

5.3.1 Crossover Structure

In the vicinity of the Crossover Structure, the ground water table is about Elevation 490 (or about 140 feet below the ground surface). Ground water 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 clearly corresponds to about Elevation 490 and is in excellent agreement with LACFCD'S reported regional ground water conditions. During the 1981 geotechnical investigation, one water sample was taken in Boring CEG-38 at a depth of about 140 feet and was subjected to chemical analyses. Results of the analyses performed indicate that the ground water is a calcium sulfate-type water (see Appendix C). Total dissolved solids (TDS) of the sample tested was 906 part per million (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 of about 150 ppm is generally regarded to be deleterious to concrete lining, requiring sulfate-resistant concrete. Since the depth to ground water appears to be at least 90 feet deeper than the proposed excavations of Design Unit A430, the ground water should have no influence on the construction operations nor on the design of the planned structures.

5.3.2 Tunnel Alignment

The tunnel invert from the north end of the Universal City Station to Station 970+00 is believed to be below ground water levels based on Borings 34-5, 34-D and 35-A (Figures 7-1 and 7-2). The influence of ground water on tunnelling excavation is discussed in Section 7.1.1 of this report.

The tunnel invert is above LACFCD's ground water Elevation 490 feet from about Station 970+00 to the Crossover Structure. At this elevation, ground water should have no influence in this tunnel segment. If future water levels rise about 10 feet, ground water could influence the tunnelling conditions from about Station 970+00 to about Station 1000+00 (3000± feet). Although no free ground water was recored in man-size Boring 35-B, 1 gpm inflows were observed from sand lenses at depths of 37 and 61 feet below the ground surface (Table 7-1). The seep at 61 feet is about 10 feet above the crown of the tunnel (Figure 7-3).

5.4 OIL OR GAS

No strong or unusal odors were detected during the drilling and logging of the borings located along the tunnel alignment nor at the Crossover Structure. Design Unit A430 is not located in an oil-producing area nor near known oil fields.

5.5 FAULTS

An unnamed, postulated fault crosses the tunnel line near Station 987± (Drawing 3). It is not known to be active or potentially active nor does it act as a ground water barrier. This fault is expected to have little or no effect on design and construction of the tunnels. Additional information regarding this fault is contained in the 1981 geotechnical investigation report (Volume 1, Sections 4.4.2.11 and 4.4.2.12).

Based on a review of published geologic maps and literature, this is the only known fault in Design Unit A430. However, because California is earthquake country, the contractor may encounter other small faults and/or shear zones. Such small faults and shear zones should not impede tunnelling excavation progress to any great extent but they should be reported immediately for further study and evaluation.

5.6 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS AT THE CROSSOVER

5.6.1 General

For purposes of our engineering evaluations, we have grouped the subsurface materials encountered at the Crossover site into two general subsurface units. These two subsurface units were described in detail in Section 5.1, and include the Upper Sand Unit and the Gravelly Sand and 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 Gravelly Sand and 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 A430 and for the sands and gravels that are present at depths up to about 50 feet. The engineering parameters developed for these two soil types are summarized in Table 5-2. 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 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 judgement in selecting appropriate material properties for the gravelly soils present at depths greater than 50 feet.

It is our judgement that the material properties selected for the sands and gravels provide a conservative estimate for the gravelly and sandy soils encountered below a depth of 50 feet. The parameters listed in Table 5-2 were used for engineering analyses, the result of which are presented in Section 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.

The properties which are listed in the first column of Table 5-2 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 ground water level 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 test 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 lense 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 gravelly sand and sandy gravels. 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 Crossover; 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 judgement that the engineering parameters given in Table 5-2 for the sands and gravels are conservative estimates for these very gravelly soils.

Table 5-2

MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

	ALLUVIUM	
MATERIAL PROPERTY	UPPER SANDS[a]	SANDS & GRAVELS[b]
Moist Density (pcf)	115	130
Effective Stress Strength		
ϕ' (degrees)	35	38
c' (psf)	0	0
Compression Modulus (psf)	$300\ \sigma'_v$[c]	$500\ \sigma'_v$[c]
Poisson's Ratio	0.35	0.35

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

[b] Applies to soils between the depth of 15 feet 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.

Section 6.0

Crossover Structure - Geotechnical Evaluation and Design Criteria

6.0 CROSSOVER STRUCTURE - GEOTECHNICAL EVALUATION AND DESIGN CRITERIA

6.1 GENERAL EVALUATION

Geotechnical design criteria for design and construction of the Crossover structure 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 Crossover will be through alluvial deposits which consists predominantly of a mixture of sands and gravels. As discussed in the previous section, the upper soils consist primarily of sands, silty sands and gravelly sands, whereas the deeper soil deposits (at depths greater than about 50 feet) are generally sandy gravel with cobbles and boulders. The depth of the excavation will range from about 54 feet at the south end of the Crossover to about 55 feet at the north end. No ground water was encountered at the Crossover 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 Crossover 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.

The following subsections present more detailed evaluations and recommendations for design and construction of the crossover structure.

6.2 EXCAVATION DEWATERING

6.2.1 General

No ground water was encountered or observed within the depths of the Crossover construction during the 1981 and 1983 field investigations. Thus the only possible source of ground water during excavations would be mainly that due to infiltration of water from the ground due to rainfall, and/or minor seeps. If any dewatering is necessary due to these sources it can probably be accomplished by use of sump pumps within the excavation combined with supplementary ditch drains. No major dewatering problems are expected to be encountered at the location of the proposed structure.

6.3 UNDERPINNING

6.3.1 Common Underpinning/Support Methods

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

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

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

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

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

6.3.2 Underpinning Considerations

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

From an engineering standpoint, the need to underpin is evaluated on the basis of expected ground movements and potential for structural damage. Figure 6-1 presents general guidelines for evaluating if a structure may be within the influence zones of the excavation; however, further evaluation of expected ground movements should be made based upon the type of shoring proposed. Section 6.4.6 discusses the anticipated ground movements in the vicinity of the excavation due to shoring movement. A conservatively designed shoring system (higher design lateral pressures) may be constructed to reduce ground movements due to shoring and thereby reduce the need to underpin.

6.3.3 Design Criteria

Figures 6-2 through 6-5 present design criteria for jacked piles and slant drilled piles. Figure 6-2 illustrates the procedures for determining the geometry of the support zones. No support should be allowed within any existing fill soils encountered or within the "no support" zone shown on Figure 6-2. Figures 6-3 through 6-5 present design capacities for underpinning system based on the expected subsurface conditions at the Crossover structure.

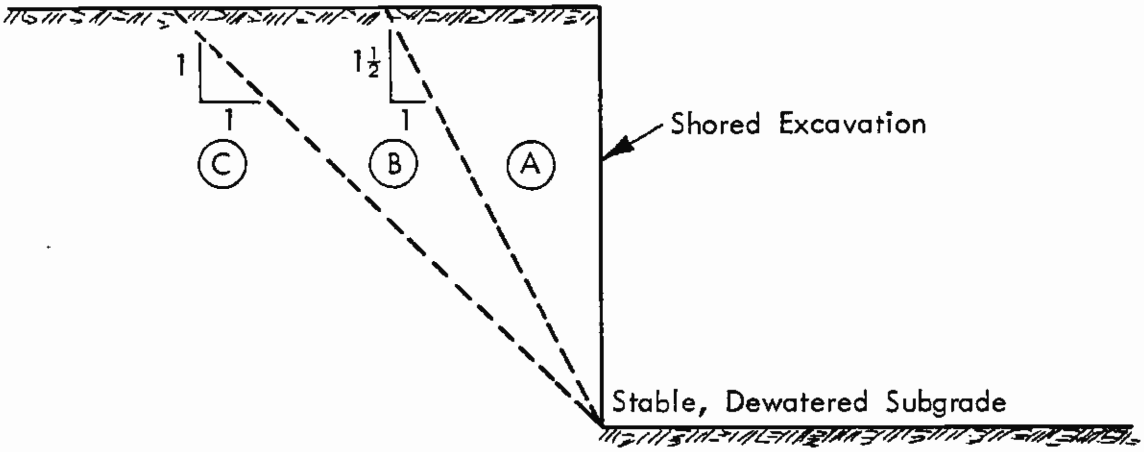
If jetting or other methods which remove soil ahead of the pile are used, no shaft frictional resistance should be allowed. To ensure proper end bearing, jetting must not be used for the final 5 feet of penetration. Group action of piles or piers should be considered and an appropriate reduction factor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

Total capacity of hand-dug, lagged piers should be limited to end bearing only and must extend below the "no support" zone shown on Figure 6-2. All piers are assumed to be 36-inch square or larger in section. For design, an allowable bearing pressure of 20 ksf may be used for piers which bear on undisturbed alluvium and penetrate at least 15 feet below the ground surface.

Surface subsidence due to lateral ground movements adjacent to the excavation are discussed in Section 6.4.5. The capability of the existing structure and underpinning system to sustain these movements should be evaluated.

6.3.4 Underpinning Performance

Underpinning is not a guarantee that the structure will be totally free from either settlement or lateral movement. Some settlement may occur during the underpinning process. Additional vertical and/or lateral movement may occur



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 Ⓐ <u>Special Provisions Required for Important Structures:</u>

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

ZONE Ⓑ <u>Generally No Special Provisions Required:</u>

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

ZONE Ⓒ <u>No Special Provisions</u>

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UNDERPINNING GUIDELINES

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Figure No.
6-1

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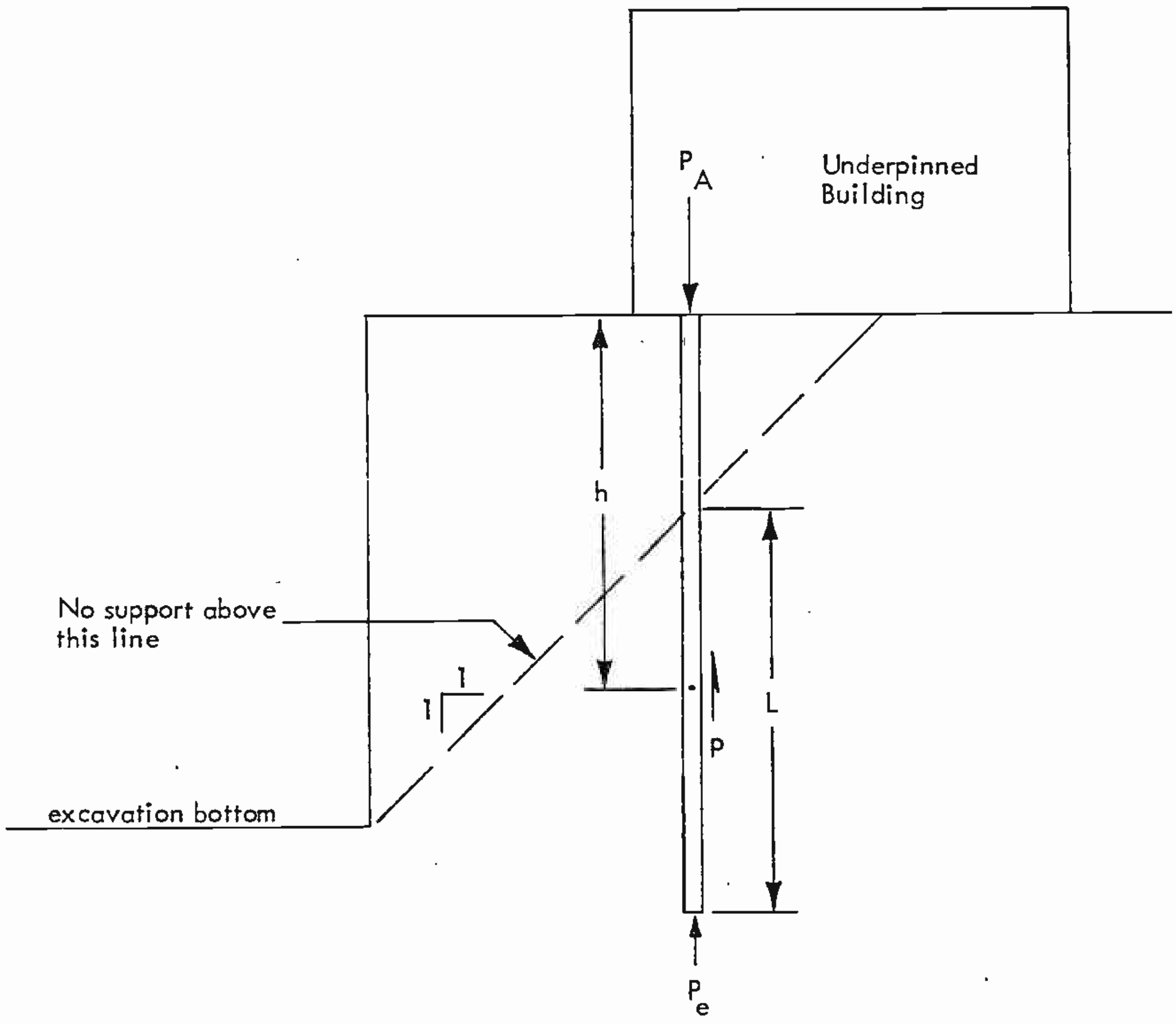

$$P_A = p \cdot L + P_e$$

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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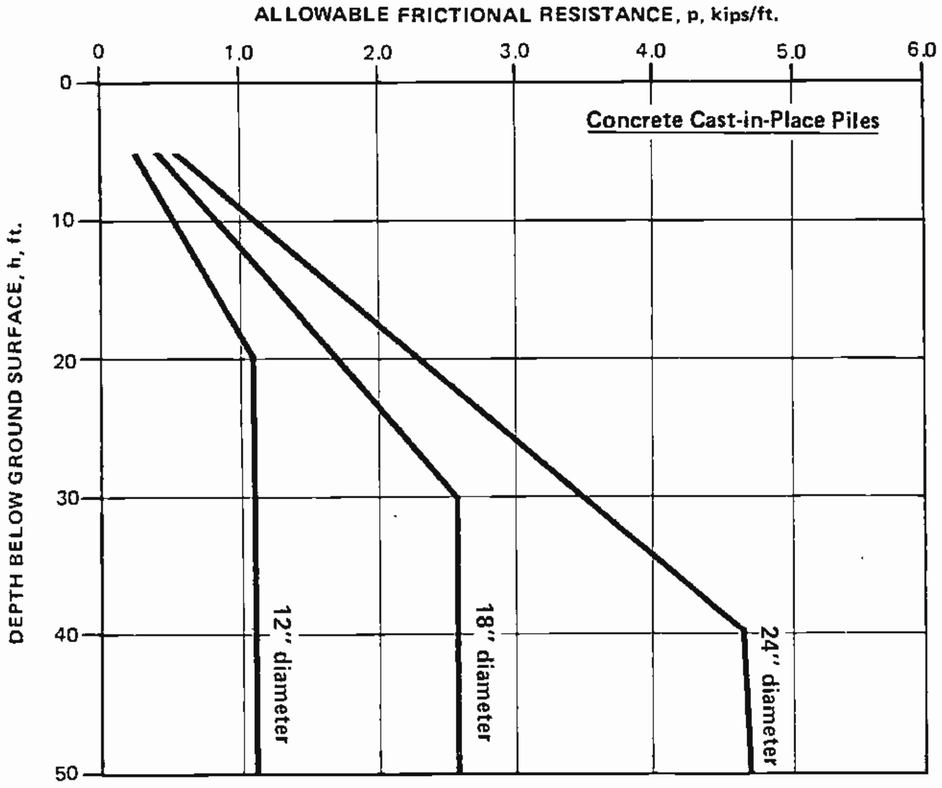

NOTES:
1) See Figure 6-2 for evaluation of total vertical pile capacity. Figure 6-5 should be used to evaluate capacity due to end-bearing support.
2) Maximum pile frictional resistance taken as value corresponding to a depth equal to 20 times diameter of pile.
3) Applicable to drilled cast-in-place piles.

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

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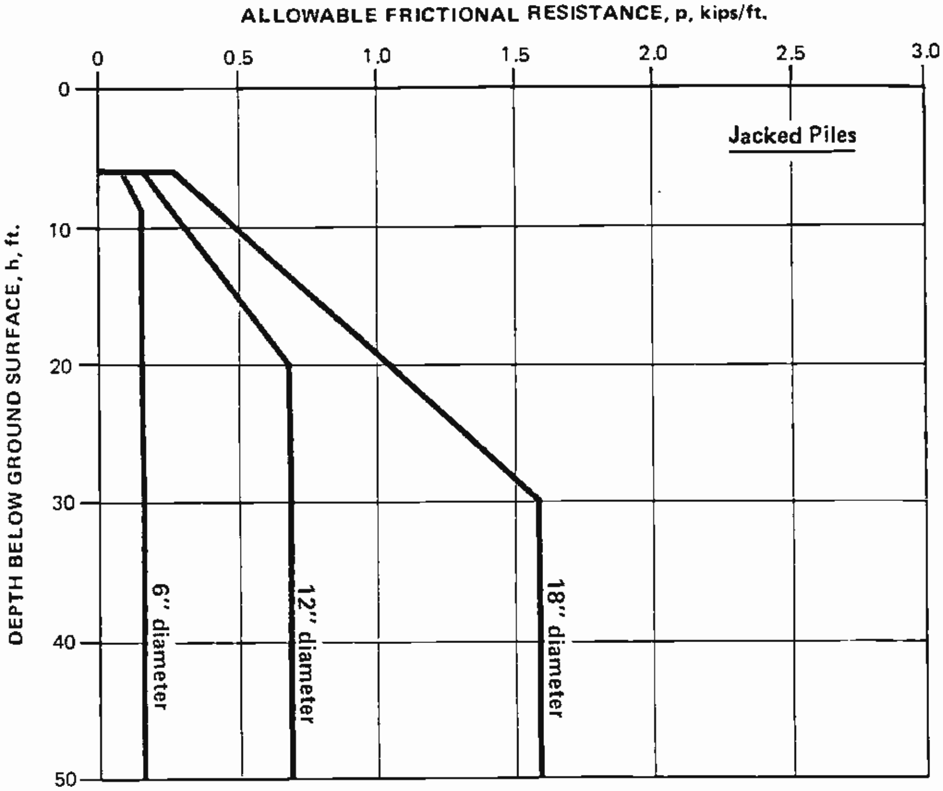
Figure No.
6-3



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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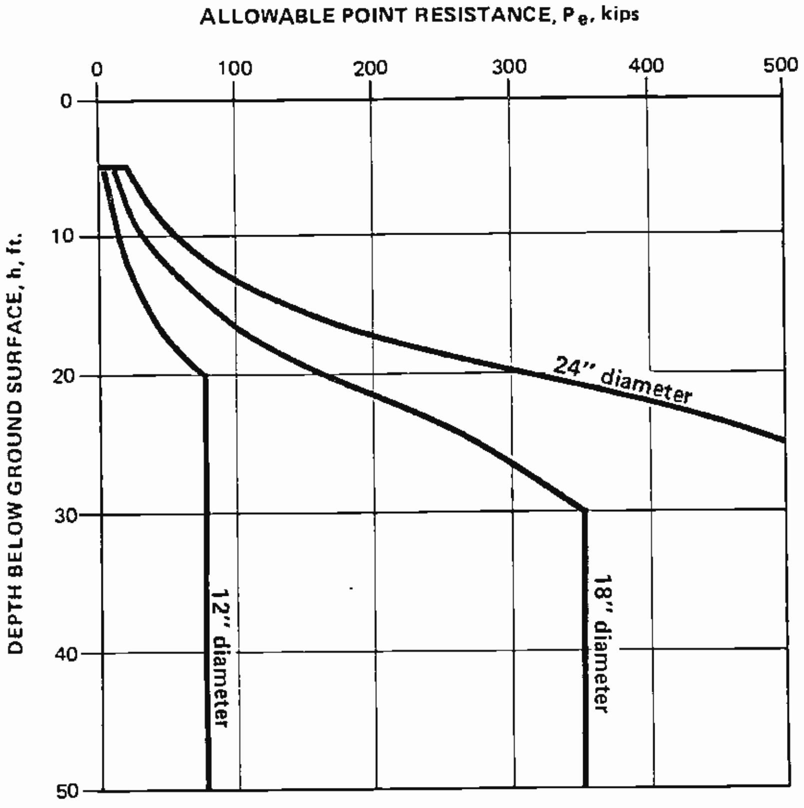
Figure No
6-4



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

Converse Consultants Geotechnical Engineering and Applied Sciences during the construction of the main excavation, depending on the performance of both the shoring and underpinning elements. Effects of subsidence may result in differential settlements between underpinning elements and non-underpinned elements.

6.3.5 Underpinning Instrumentation

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

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

6.4 TEMPORARY SLOPED EXCAVATIONS AND SHORING SUPPORT SYSTEMS

6.4.1 General

The required Crossover structure excavation will extend approximately 55 feet below the existing ground surface. There are several currently used shoring methods for supporting vertical excavations. These methods include soldier piles with lagging, sheet piles, and slurry wall construction. Bracing systems are generally limited to soil/rock anchor tiebacks or internal bracing. We understand that the excavation support system will be chosen and designed by the contractor in accordance with specified criteria and subject to the review and acceptance by the Metro Rail Construction Manager.

Conditions encountered at the site will cause some difficulty in installation of any type of shoring system. Caving of the granular alluvium due to vibrations was experienced during exploration. Caving should be anticipated for pile and tieback excavations in the granular alluvium.

Driven sheet pile shoring does not appear feasible at this site due to the presence of dense gravelly soils.

Both slurry wall and soldier pile systems are considered feasible but both will encounter similar problems from caving; however, caving may be more severe during construction of slurry wall panels than for soldier piles due to the size and shape of the panel excavation. Slurry wall construction would not require unusually deep penetration below the excavation to seal against hydrostatic pressures at this site since ground water was not encountered.

Internal bracing would appear to be preferable over tiebacks from the installation standpoint due to the potential for caving in the granular alluvium.

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

6.4.2 Sloped Excavations

Portions of the shallower cuts could be made with a sloped excavation. The major factors which determine the safe, stable slope include soil conditions, ground water 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 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 Soldier Pile Shoring Systems

Soldier piles have been installed in the Los Angeles area in soils similar to those encountered at the proposed Crossover structure site. Appendix D.1 summarizes several case studies in the Los Angeles area involving soldier pile excavations to depths exceeding 100 feet. In the granular alluvium, caving may be a problem. The contractor should recognize that caving conditions may be encountered during construction of soldier piles or other drilled shaft elements.

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

6.4.4 Shoring Design Criteria

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

Appendix F.1 summarizes the design shoring pressures for nine shoring systems in the Los Angeles vicinity. To our knowledge there are no data on field measurements of actual lateral soil pressures for shored excavations in the Los Angeles area and, therefore, the design pressures of Appendix F.1 have not been strictly verified. However, performance of shoring walls designed on the basis of the indicated values has generally been good.

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 a soldier pile shoring system are presented in Figure 6-6a. Figure 6-6e also includes the case of partial slope cuts. Appendix D.2 provides technical support for the recommended seismic pressures of Figure 6-6f. The full loading diagram above the bottom of excavation should be used to determine the design loads on tieback anchors and the required depth of embedment of the soldier piles. For computing design stresses in the soldier piles, the computed values can be multiplied by 0.8. For sizing lagging, the earth pressures can be reduced by a factor of 0.5.

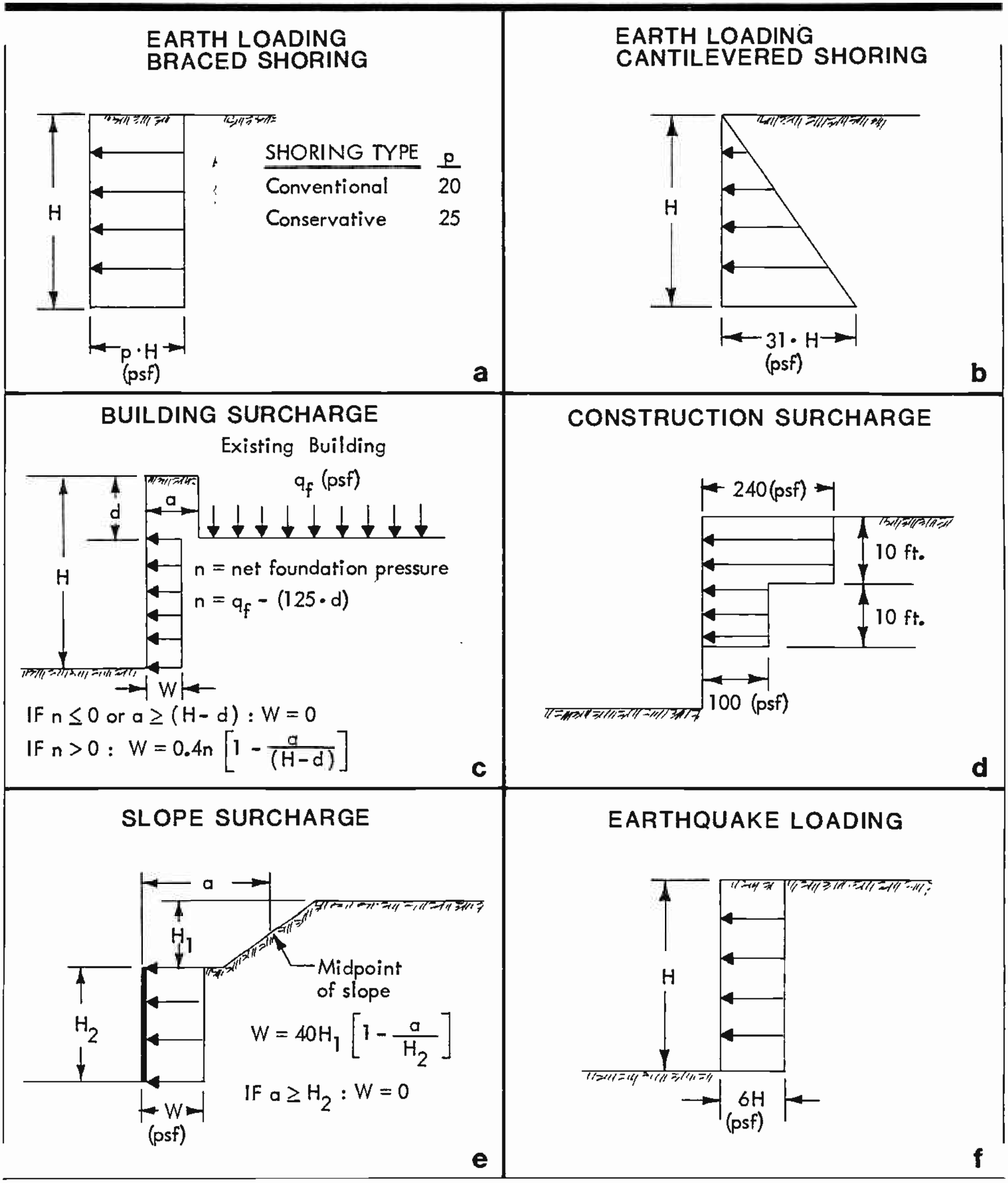
- Depth of Pile Embedment: The embedment depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to satisfy both the lateral and vertical loads under static and dynamic loading conditions.

  The required depth of embedment to satisfy vertical loading should be computed based on allowable vertical loads shown on Figure 6-7 for piles penetrating alluvium.

  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 and/or internal bracing. The required depth of embedment to satisfy lateral loads should be computed based on the net allowable passive resistance (total passive resistance of the soldier pile minus the active earth pressure below the excavation). Due to arching effects, it is recommended that the effective pile diameter be assumed equal to 1.5 pile diameters or half of the pile spacing, whichever is less. Figure 6-8 indicates the recommended method to compute net passive resistance for piles penetrating alluvium.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. At the Crossover structure site the alluvial soils encountered generally were granular soils that would be subject to ravelling and sloughing. Thus, it is recommended that the pile spacing be limited to about 8 feet and that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles. The contractor should limit the temporarily exposed granular soil height to less than 3 feet to control ravelling problems.

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



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No.
6-6

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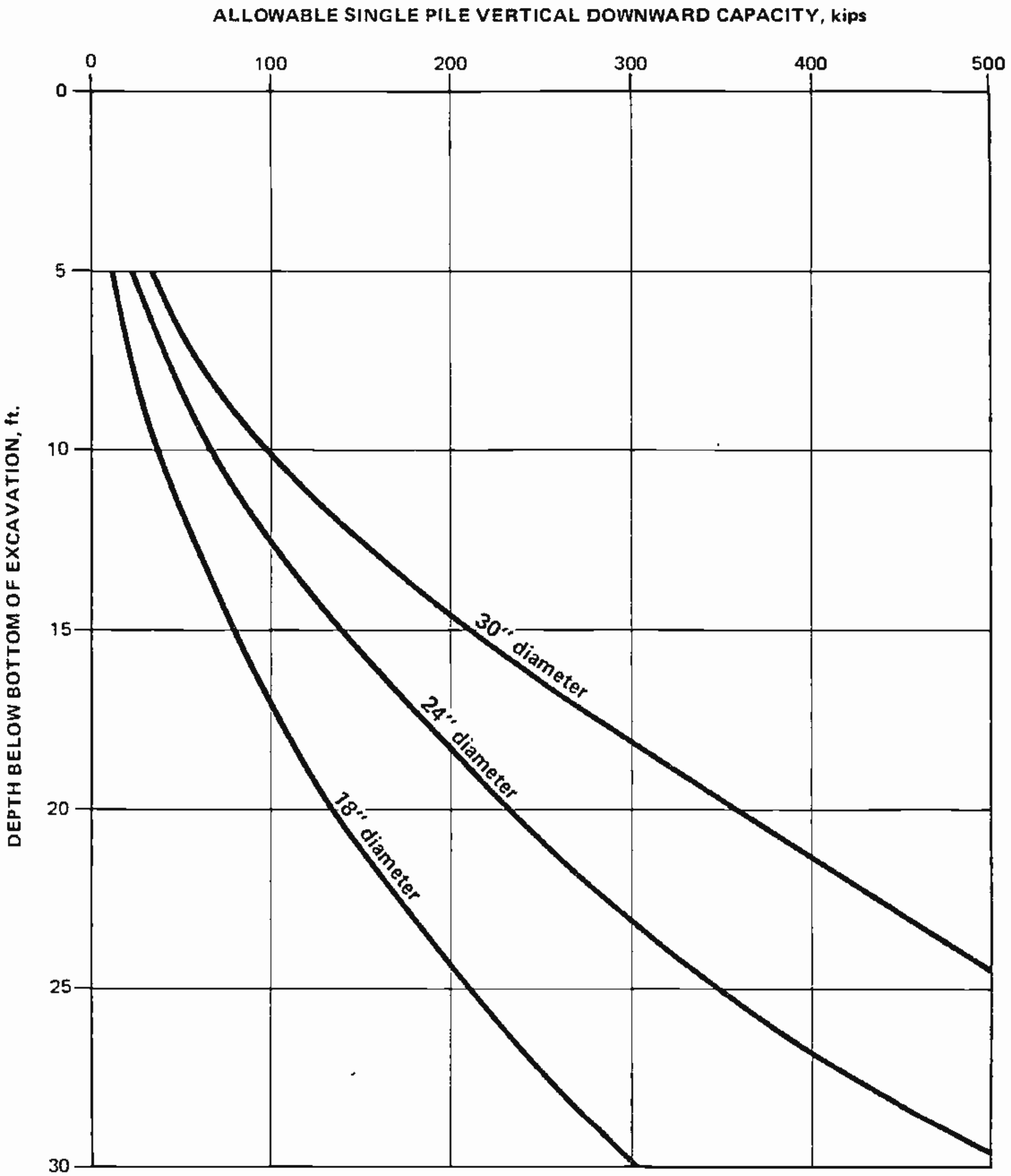

NOTES: 1) Total capacity includes contribution from both shaft frictional resistance and end-bearing.

2) For seismic design, capacities may be increased by one-third.

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ALLOWABLE VERTICAL PILE CAPACITY IN ALLUVIUM FOR SHORING AND DECKING

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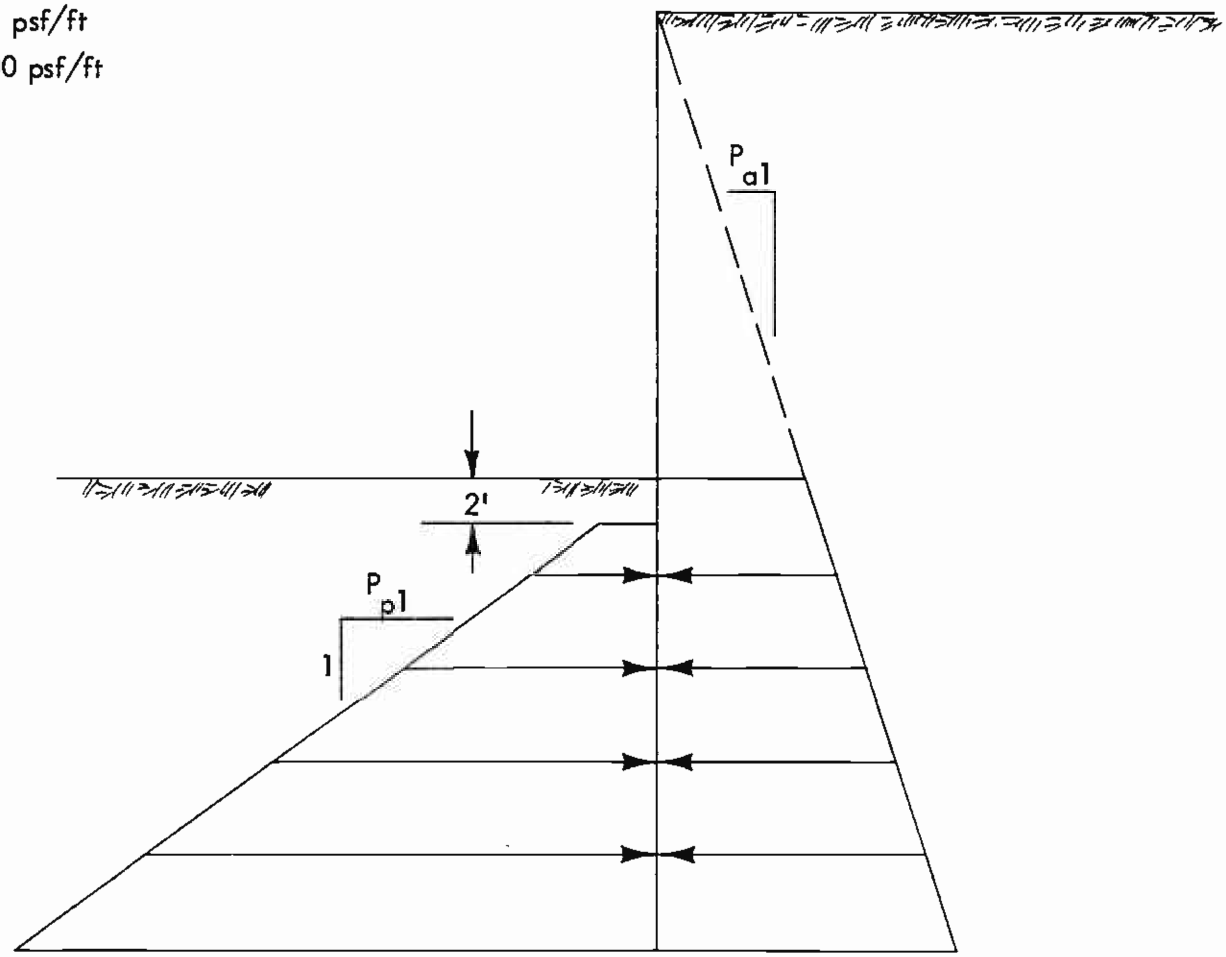
Figure No.
6-7

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 $\frac{1}{2}$ 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

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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. The economics of tiebacks versus internal bracing is normally controlled by excavation width. The critical width is generally on the order of 30 to 40 feet. However, at this site, installation of tiebacks may be difficult in the granular alluvium due to the potential for caving. Obtaining permission to install tiebacks under adjacent properties and encountering obstructions from adjacent below grade structures (such as basements) can also affect the economics and feasibility of tiebacks.

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

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

- Conventional Soldier Pile Wall: 12 feet
- Conservative Soldier Pile Wall: 8 feet

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

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

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

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

$$P = \pi DLq$$

Where:

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

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

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

Where:

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

Allowable anchor capacity/length relationships for tieback types other than straight shaft friction anchors cannot be generalized. Design parameters for anchors such as high pressure grouted anchors and high pressure regroutable anchors must be based on experience in the field and on the results of test anchors.

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

The anchors may be installed at angles generally between 20° to 50° below the horizontal. Based on specific site conditions, these limits could be expanded to avoid underground obstructions. Structural concrete should be placed in the lower portion of the anchor up to the limit of the no-load zone. Placement of the anchor grout should be done by pumping the concrete through a tremie or pipe extending to the bottom 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. Caving of the granular alluvium is expected to occur due to vibration from the drilling equipment and other effects. Uncontrolled caving not only causes installation problems but could result in surface subsidence and settlement of overlying buildings. To minimize caving, casing could be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, a hollow stem auger could be used.

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 excavations combined with our engineering judgement, we estimate that the ground movements associated with properly designed and carefully constructed soldier pile shoring systems will be as follows:

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

° Conventional Wall With Internal Bracing: The maximum horizontal and vertical ground movements should 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 the for the conservative tieback supported wall.

6.5 SUPPORT OF TEMPORARY DECKING

We understand that temporary street decking for the Weddington Street crossing will require center support piles. These piles would have to extend below the maximum proposed excavation level for support. Piles would be founded within the granular alluvium which is suitable for supporting pile loads.

We evaluated allowable loads on cast-in-place concrete piles 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 Crossover structure excavation should be instrumented to reduce liability (by having documentation of performance), to validate design and construction requirements, to identify problems before they become critical, and to obtain data valuable for future designs.

We recommend the following instrumentation program:

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

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

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

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

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

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

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

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

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

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

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

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

6.7 EXCAVATION HEAVE AND SETTLEMENT OF THE CROSSOVER STRUCTURE

The proposed excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed 55-foot excavation at the Crossover structure will decrease the vertical effective ground stresses by about 6500 psf. Stress reduction caused by the excavation will result in an elastic rebound or heave of the alluvium below the excavation. The structure and subsequent backfilling will reload the soil. We estimate that the net loads will be about 4500 to 5000 psf. These loads will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil exceeds the weight of the final structure, the structure will experience some settlement due to recompression of the elastic heave.

We estimate that the maximum heave at the center of the Crossover structure excavation will range from 1-1/2 to 3 inches. We believe that the majority of this will occur while the excavation is being made. These estimates are based on computations of elastic shear deformation (elastic rebound) within the alluvium underlying the proposed excavation.

Settlement calculations for the Crossover structure were performed based on the elastic properties of the subgrade materials and based on the estimated imposed loads due to the structure and backfill given above. Total elastic settlement of the alluvium-supported structure was estimated to range from 2 to 3 inches.

Due to the long narrow shape of the imposed load, the calculated differential settlement between the edge and center of the structure is 1/2 inch.

These calculated settlement values are based on a uniform foundation bearing pressure which could result only from a uniformly loaded and perfectly flexible structure. We understand that the Crossover structure will be structurally quite stiff. Thus the actual differential settlement may be less than that calculated for the assumed theoretical flexible foundation. Anticipated differential settlements and distribution of the bottom slab bearing pressures could be estimated based on a soil-structure interaction analysis. However, such an analysis is beyond the scope of this study.

6.8 FOUNDATION SYSTEMS

6.8.1 Crossover Structure

It is understood that the proposed Crossover structure will be supported on a thick base slab which will function as a massive mat foundation. We estimate that the net mat foundation bearing pressures will be about 4500 to 5000 psf. In our opinion the structure can be adequately supported on mat foundations.

6.8.2 Support of Surface Structures

Surface structures generally can be supported on conventional spread footings founded on undisturbed stiff or dense natural soils. It should be noted that the upper 15 feet of soil at this site was found to include zones of loose to medium dense soils and therefore may not be suitable for support of structures. If suitable natural soils do not exist at the particular site, footings may be founded on a zone of properly compacted structural fill (see Appendix E). Allowable bearing pressures and estimated total settlements of spread footings bearing on the natural alluvium or compacted fill can be determined based on Figure 6-9. This figure is based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed information on structural loadings and site conditions at specific surface structure locations. Detailed site specific studies should be performed to provide final design recommendations for individual structures.

All spread footing foundations should be founded at least 2 feet below the lowest adjacent final grade and should be at least 2 feet wide. The bearing values shown on 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 33%. Differential settlements between adjacent footings should be estimated as 1/2 of the average total settlements or the difference in the estimated total settlements shown on 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 undisturbed alluvium or properly compacted fill. Frictional resistance at the base of foundations should be determined using a frictional coefficient of 0.4 with dead load forces.

6.9 LOADS ON SLAB AND WALLS

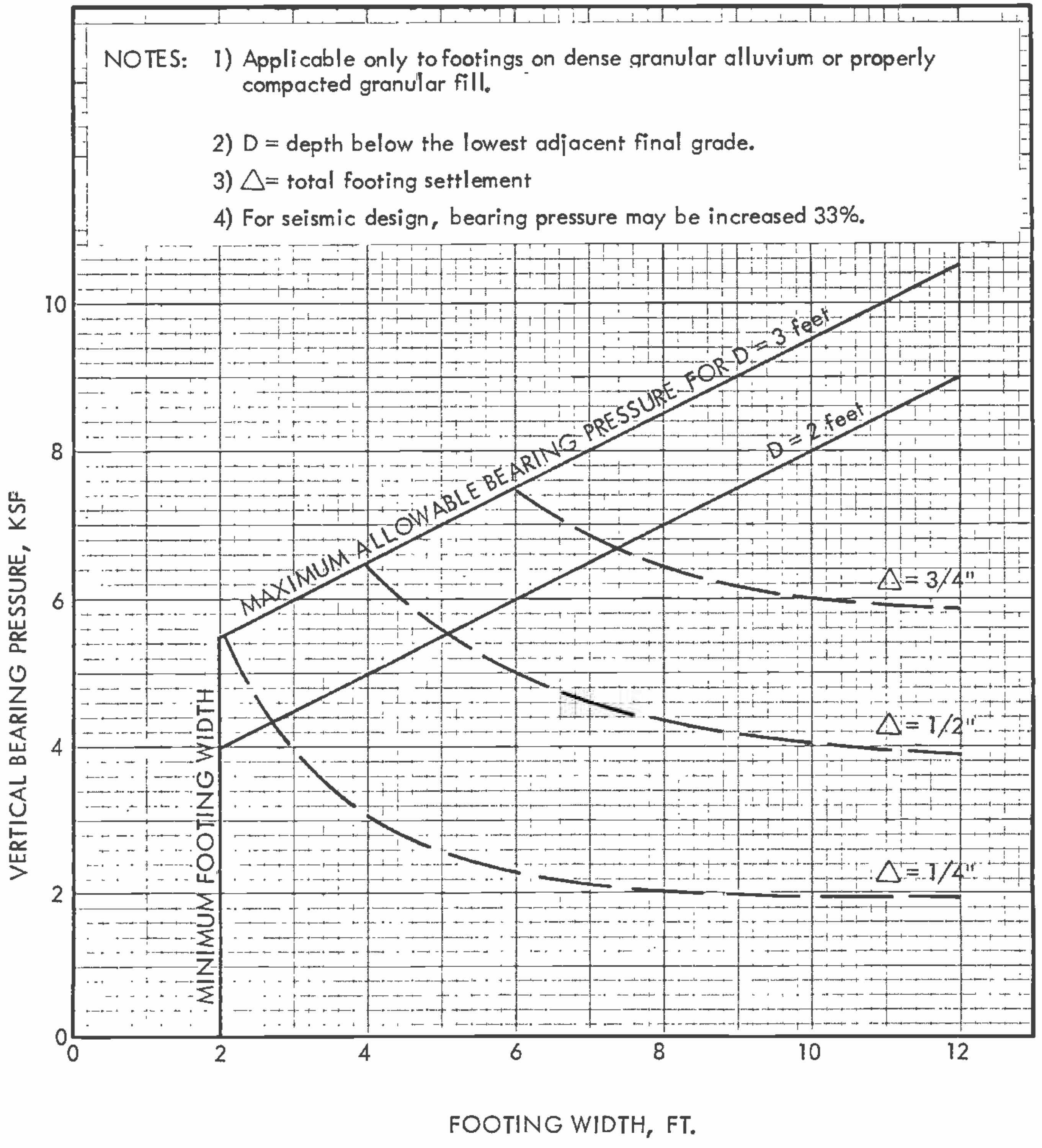
6.9.1 Hydrostatic Pressures

Ground water was not encountered within the borings drilled at the Crossover site in 1983. It is recommended that for design the maximum ground water levels be assumed to be below the base of the foundation slab.

6.9.2 Permanent Static Earth Pressures

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

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



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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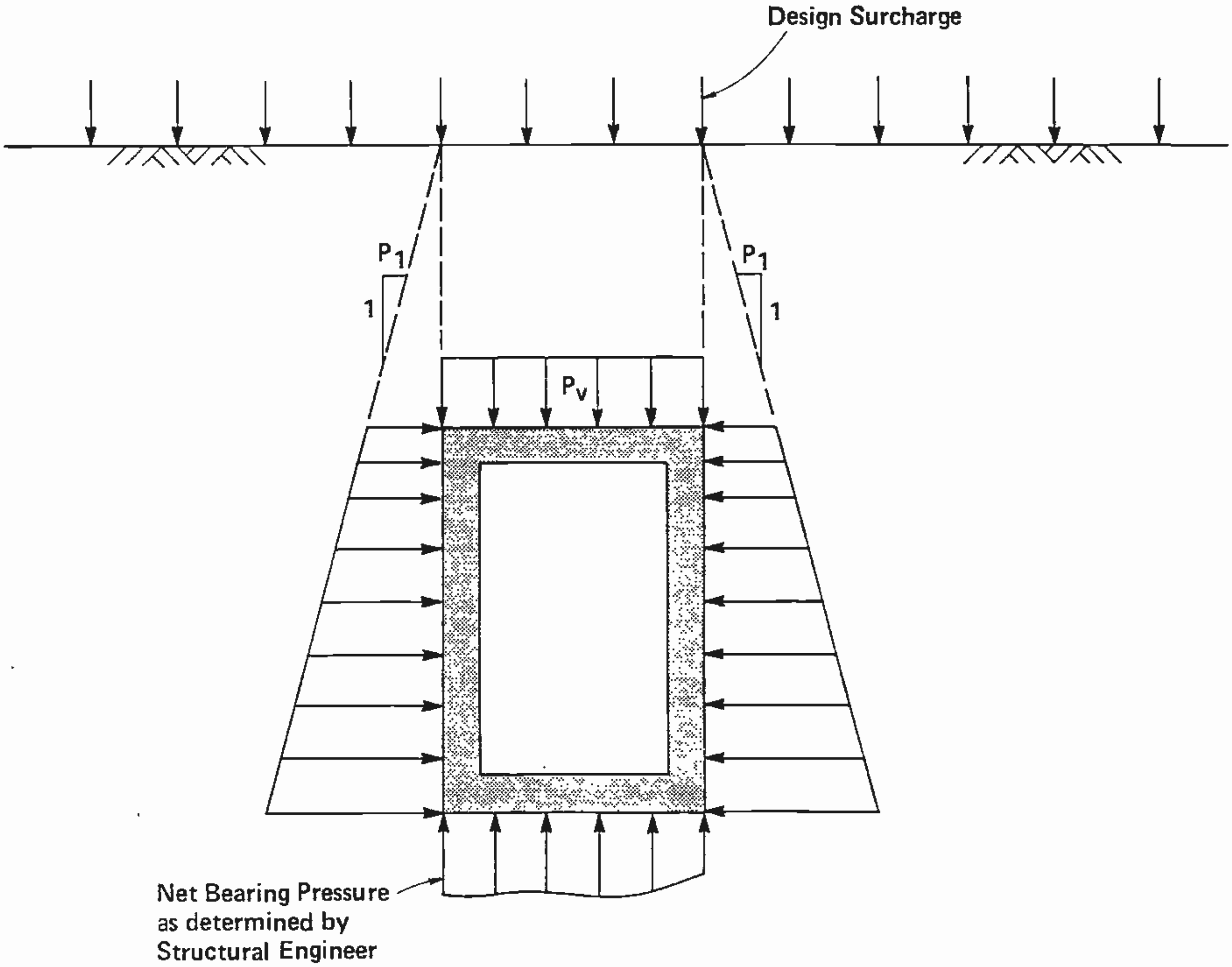

LOADING CONDITION	DESIGN LOAD PARAMETERS, psf	
	P_1	P_v*
End of Construction	31	—
Long Term	50	—
Side Sway	31/50	—

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

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LOADS ON PERMANENT WALLS

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Figure No.
6-10

6.9.3 Surcharge Loads

Lateral surcharge loads from existing or proposed buildings above the structure must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-10. Vertical surcharge loads due to possible future structures, surface traffic, etc. should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.

6.10 SEISMIC CONSIDERATIONS

6.10.1 General

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. 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 SCRTD 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 were be presented in the referenced 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 constrained modulus, and dynamic shear modulus at low strain levels. In addition, certain types of equivalent linear analyses required 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 downhole geophysical surveys performed in Boring CEG-38 during the 1981 investigation are presented at the top of Table 6-1. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish appropriate modulus values at low strain levels. Computed moduli values for the alluvium are tabulated in Table 6-1.

TABLE 6-1

RECOMMENDED DYNAMIC PROPERTIES FOR ALLUVIAL MATERIALS

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.

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.

6.11 EARTHWORK CRITERIA

Site development is expected to consist primarily of excavation for the subterranean structure but will also include general site preparation, foundation preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations are presented in Section 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill are presented in Appendix G. Recommended specifications for compaction of fill are also presented in Appendix G. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. If the granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the geotechnical engineer.

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

6.12 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

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

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

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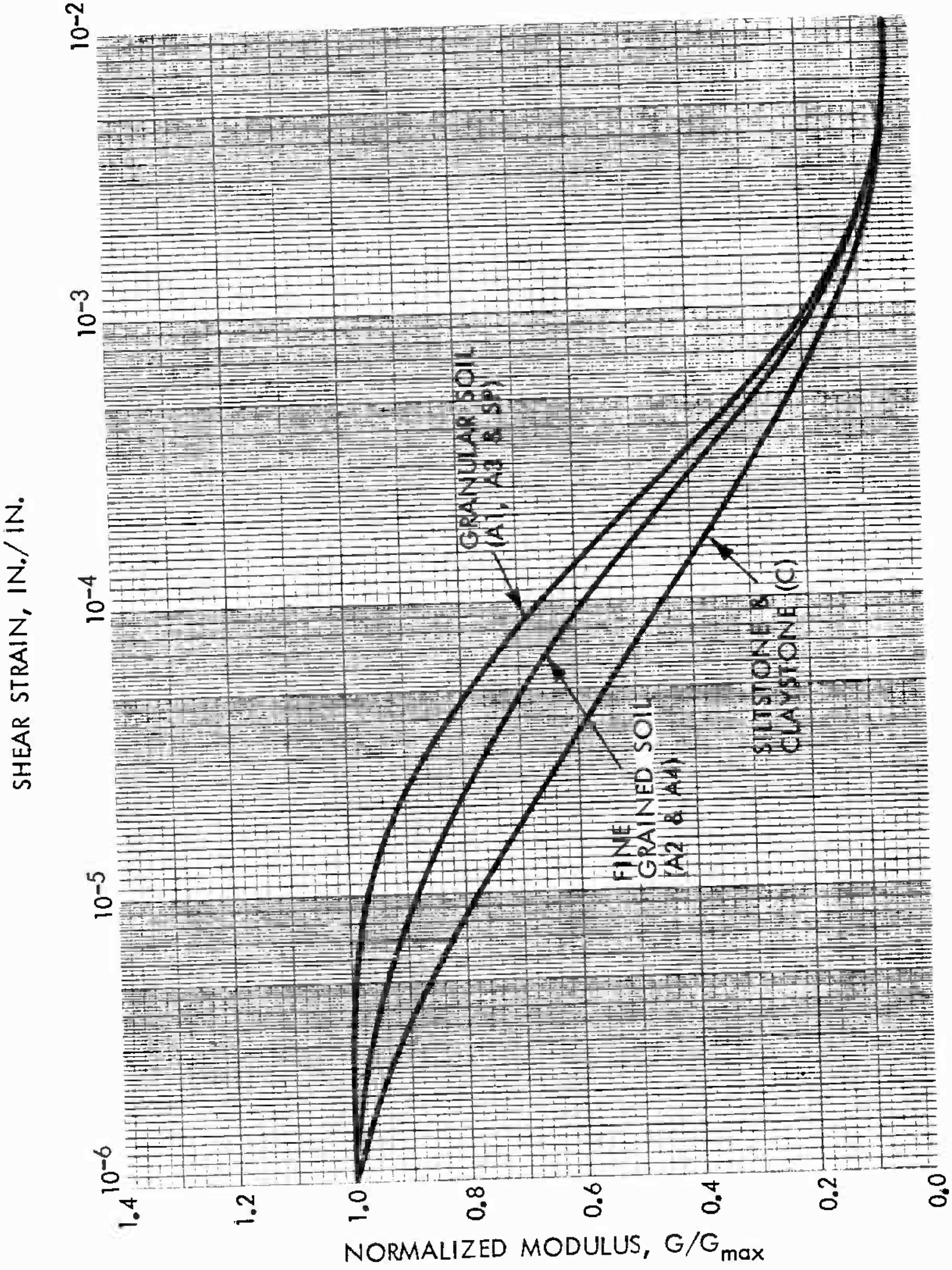

RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
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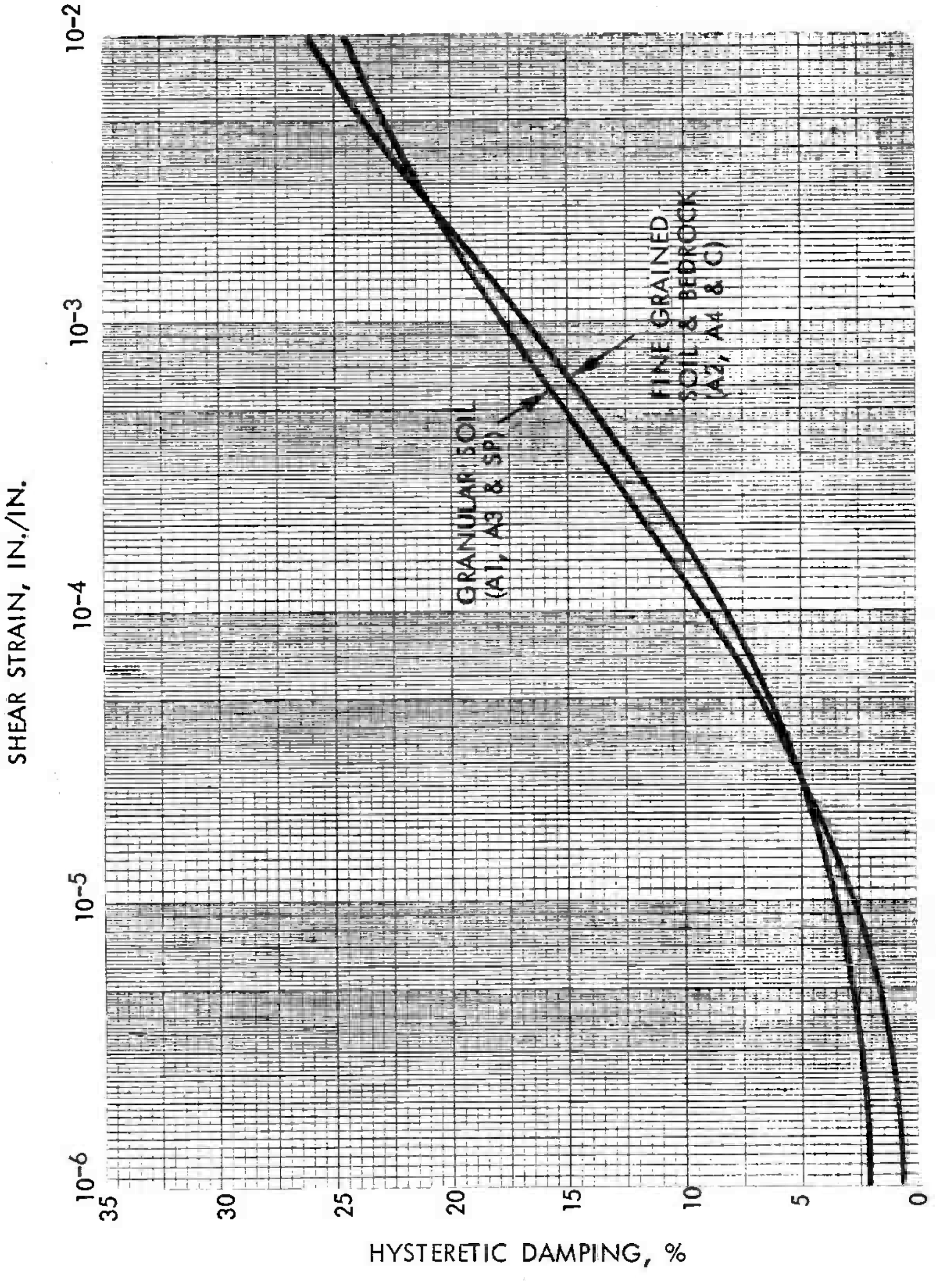

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RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No.
6-12

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

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

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

Section 7.0

Tunnel Alignment - Geotechnical Evaluation and Tunnelling Conditions

7.0 TUNNEL ALIGNMENT - GEOTECHNICAL EVALUATION AND TUNNELLING CONDITIONS

The general geologic stratigraphy along Design Unit A430 tunnel alignment is shown on Drawings 2, 3 and 4. The tunnels occur between about Station 935+50 and Station 1043+00, a distance of about 2 miles.

The average depth of ground cover above the crown of the tunnels is 60 feet, varying between a minimum of 35 feet near Station 1043+00 and a maximum of 75 feet near Station 990+00. The tunnel is below the known water level in the alluvium between about Station 935+50 and 970+00 and above the water level from about Station 970+00 to the south end of the North Hollywood Station's Crossover Structure. Although gravel/cobbles, sometimes up to 12-inches in diameter are infrequent, their presence may impact the type of equipment selected and possibly the rate of excavation progress.

7.1 STRATIGRAPHY, GROUND WATER AND TUNNELLING CONDITIONS

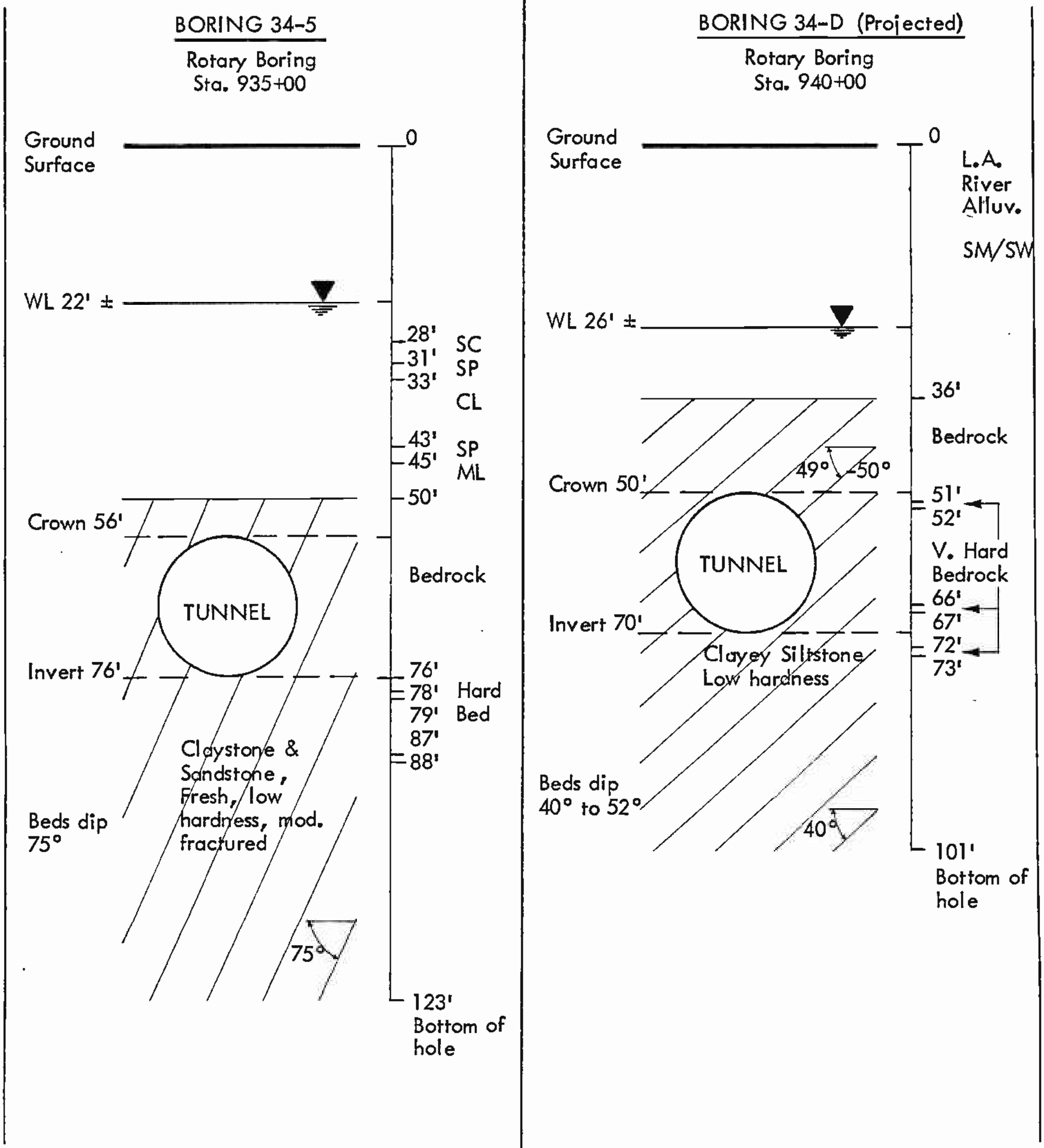
The geologic units existing along the tunnel alignment consist of cohesionless and cohesive alluvium and bedrock-type materials of the Topanga Formation. These units are described in Sections 5.1, 5.2 and 5.3 of this report. The following descriptions define ground water conditions and the soft ground tunnelling conditions between the north end of the Universal City Station and the Crossover structure at the south end of North Hollywood Station and at significant changes in subsurface stratigraphy and/or conditions.

7.1.1 Station 935+50 and Station 970+00 (3450 feet - Drawings 2 and 3)

The tunnel segment leaving the Universal City Station passes through the Topanga bedrock formation. Mixed-face conditions may be encountered immediately adjacent to the Universal City Station and persist to Station 970+00. The rock-alluvium interface may vary locally from that shown on Drawings 2 and 3, and the crown may pass in and out of mixed-face conditions locally over this length. The alluvial materials at the mixed-face can consist of saturated gravels, sands, silts and clays overlying soft Topanga siltstone, claystone and sandstone materials. The ground water level above the crown varies between 30 feet at the north end of the Universal City Station to zero near Station 960+00. Water levels do not pass below tunnel invert until about Station 970+00. It is anticipated that flowing ground conditions may be encountered at the crown and face of the tunnels assuming that dewatering systems are not in place or operating properly. Below the zone of weathering, the remaining perimeters of the tunnel are expected to pass through impervious, competent stable siltstones and claystones of the Topanga formation.

The water level in Boring WC-11 represents a 1977 level, following two drought seasons, therefore, the water level in 1984 may be considerably higher than shown on Drawing 2 (in the vicinity of the Los Angeles River).

Examples of stratigraphic variations in bedrock, soil and water level conditions are graphically represented by records from Borings 34-5, 34-D, 34-C, WC-11 and 35-A (Figures 7-1, 7-2 and 7-3). At these locations the maximum and minimum bedrock recorded above the crown range from 0 at Boring 35-A to 14 feet at Boring 34-D.



See Table A-1 of Appendix A for description of soil symbols.

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-1

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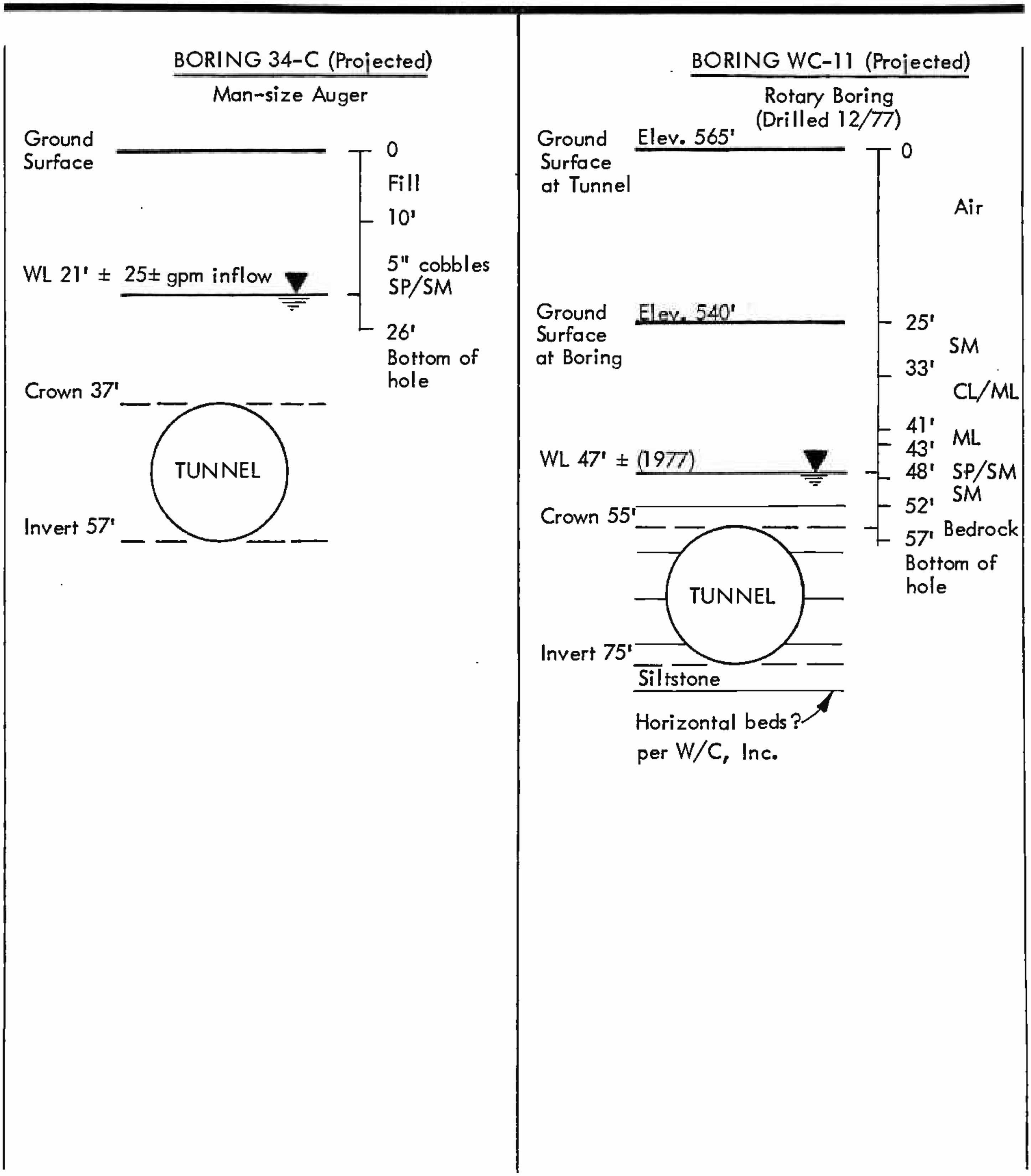

See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-2

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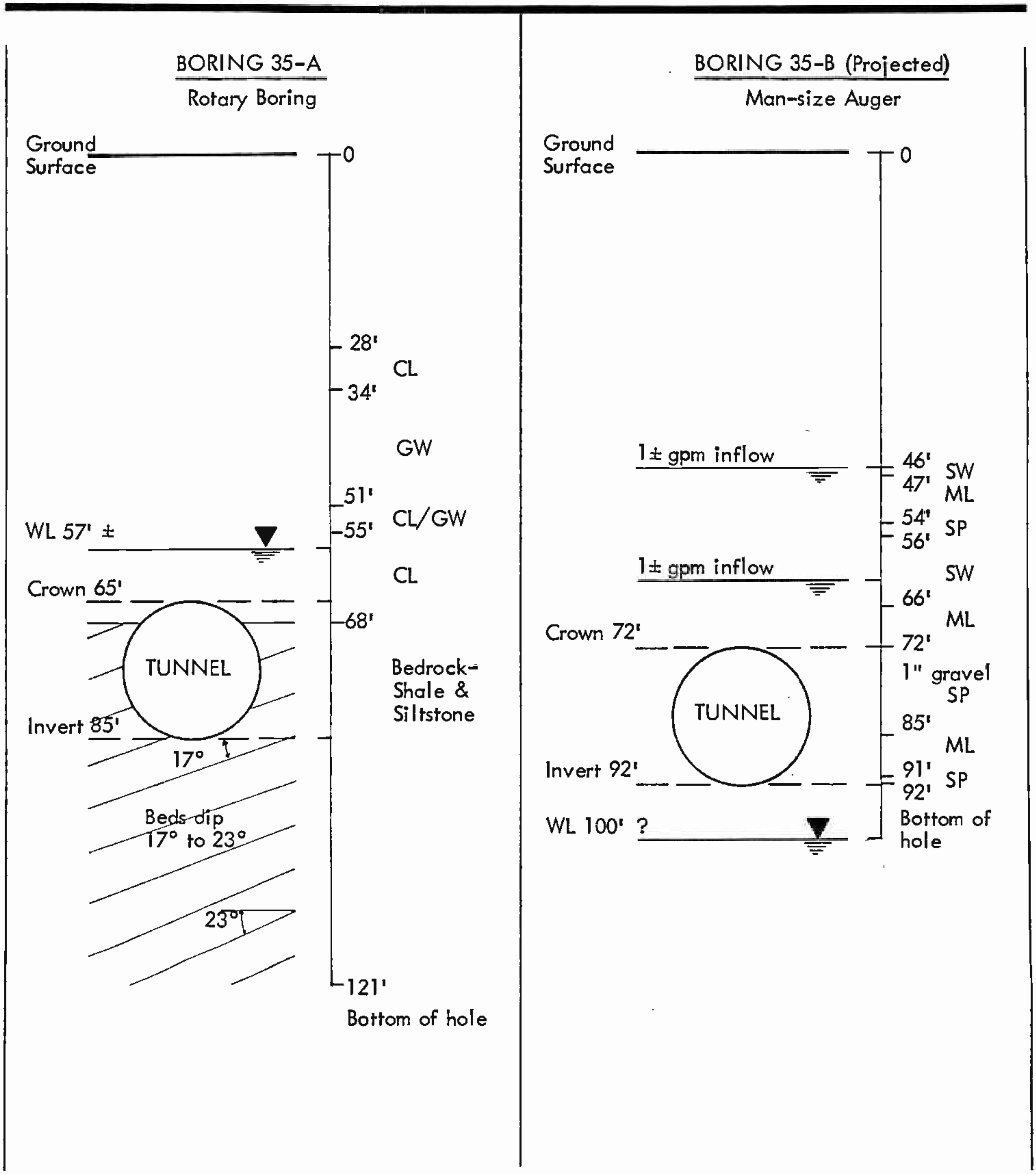

See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-3

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Similar variations in soil stratigraphy and ground water conditions are judged appropriate north of Boring 35A, although the depths to bedrock and ground water levels are unknown.

We believe that the soil conditions between the Universal City Station and Station 970+00 are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavation equipment. Because of the mixed-face conditions, nature of the soil and ground water conditions, we do not believe that methods of tunnel construction not employing a shield will be successful in this segment of the tunnel. Construction shield tunnelling methods will require means for the utilization of fore polling and/or breast boarding techniques to maintain stability of the face, prevent loss of ground and avoid surface settlement along the alignment. Excessive hydrostatic uplift pressures below tunnel invert are not anticipated.

7.1.2 Station 970+00 and Station 1043+00 (7300 feet, Drawings 3 and 4)

The tunnels between Station 970+00 and the Crossover structure will encounter heterogeneous alluvial materials consisting of interbedded horizons of unsaturated cohesive and cohesionless-like materials with variable distributions over the face of the tunnels.

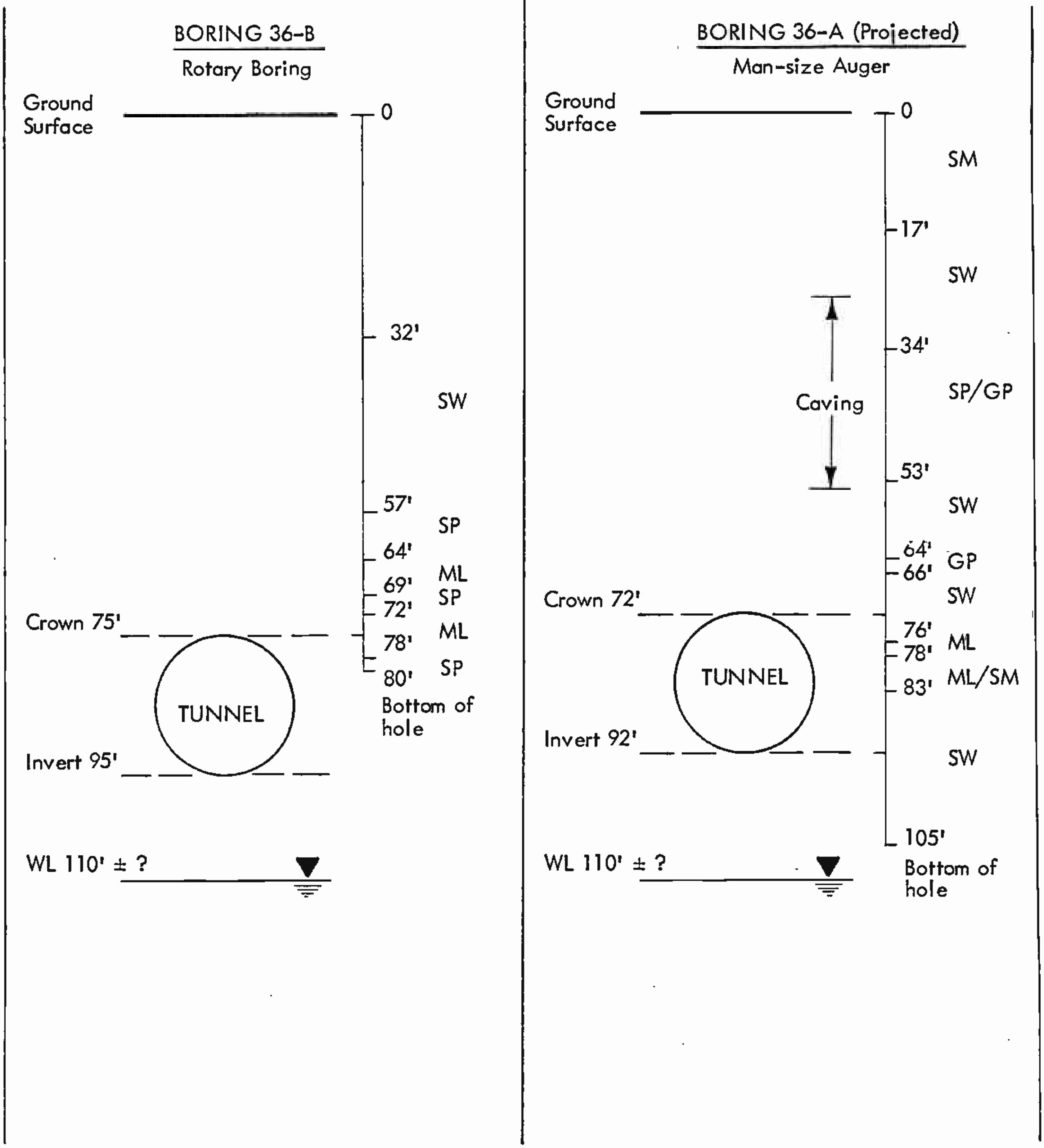
The ground water level as measured is believed to be entirely below invert in this tunnel segment. Therefore, this tunnel segment should not encounter flowing ground conditions.

Typical examples of stratigraphic variations and soil conditions which may be encountered by the tunnel construction along this segment are graphically represented by records from Borings 35-B, 36-B, 36-A, 37, 37-A, 38-A, 38-1, 38-2 and 38-3 (Figures 7-3, 7-4, 7-5, 7-6, 7-7, and 7-8). The coarse-grained cohesionless fractions are expected to predominate in the face of the tunnel excavation (Figures 7-5, 7-6, 7-7 and 7-8). Interbedded horizons of cohesive silts and clays should also be anticipated at the face of the tunnel excavation (Figures 7-3 and 7-4). Caving of sand and gravel near tunnel grade was recorded in man-size Borings 37-A and 38-A (Figures 7-5 and 7-6). Caving that occurred above the water table was caused by vibrations and the mechanical action of the drilling rig (Table 7-1).

Had the man-sized borings been left open for a day or two, we believe the cohesionless fractions may have caved naturally, due to a reduction in moisture content. Also, a horizontal tunnel bore will be more susceptible to caving than the vertical boring.

Gravel and cobbles, 1" to 12" in diameter were observed at tunnel grades in Borings 35-B, 37-A and 38-A (Figures 7-3, 7-5 and 7-6) as follows:

Boring 35-B	Trace of 1" gravel, 72' to 85'
Boring 37-A	40% cobbles to 8", 47' to 60'
Boring 38-A	40% cobbles to 12", 47' to 60'



See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-4

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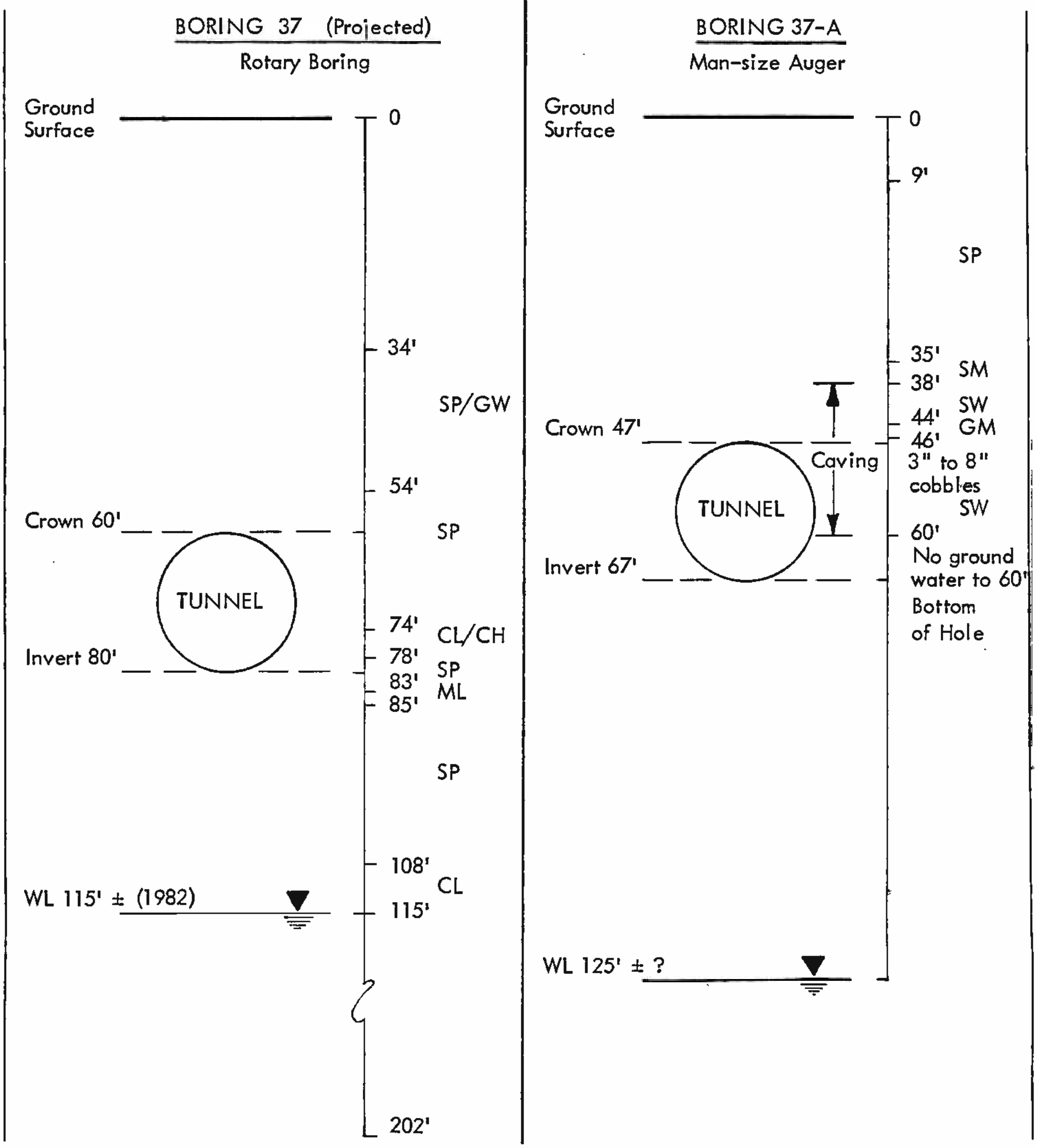

See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-5

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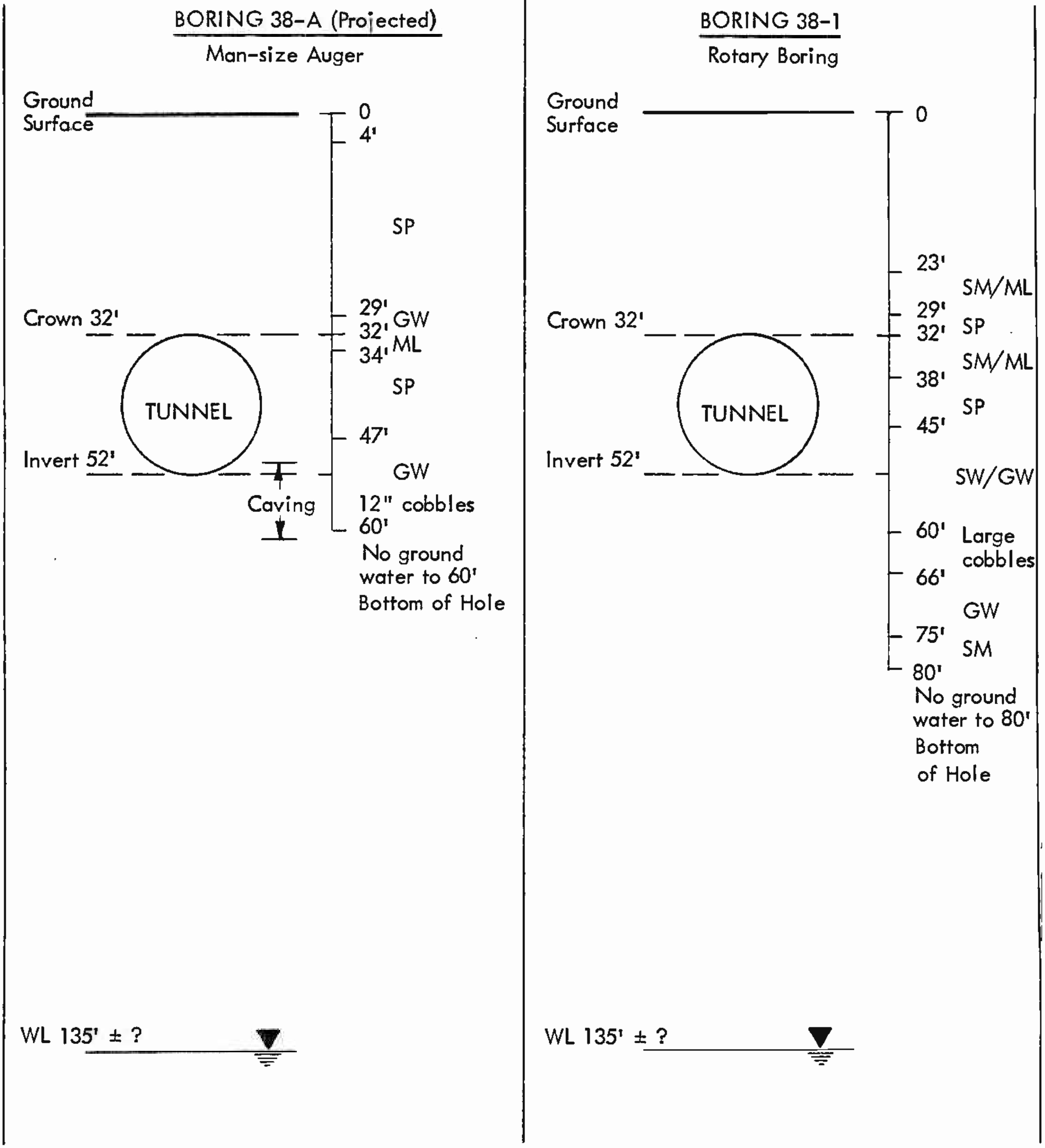

See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-6

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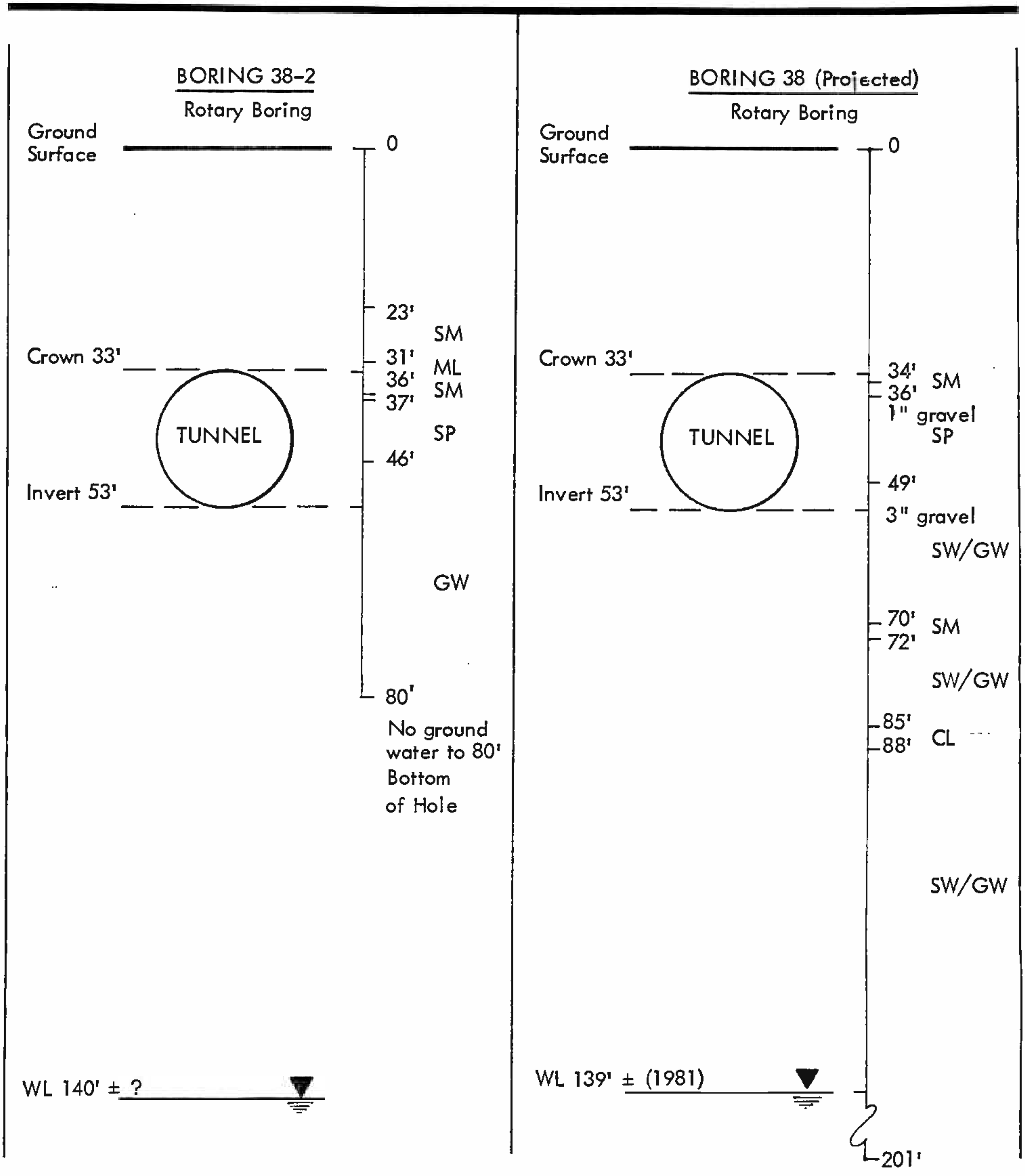

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-7

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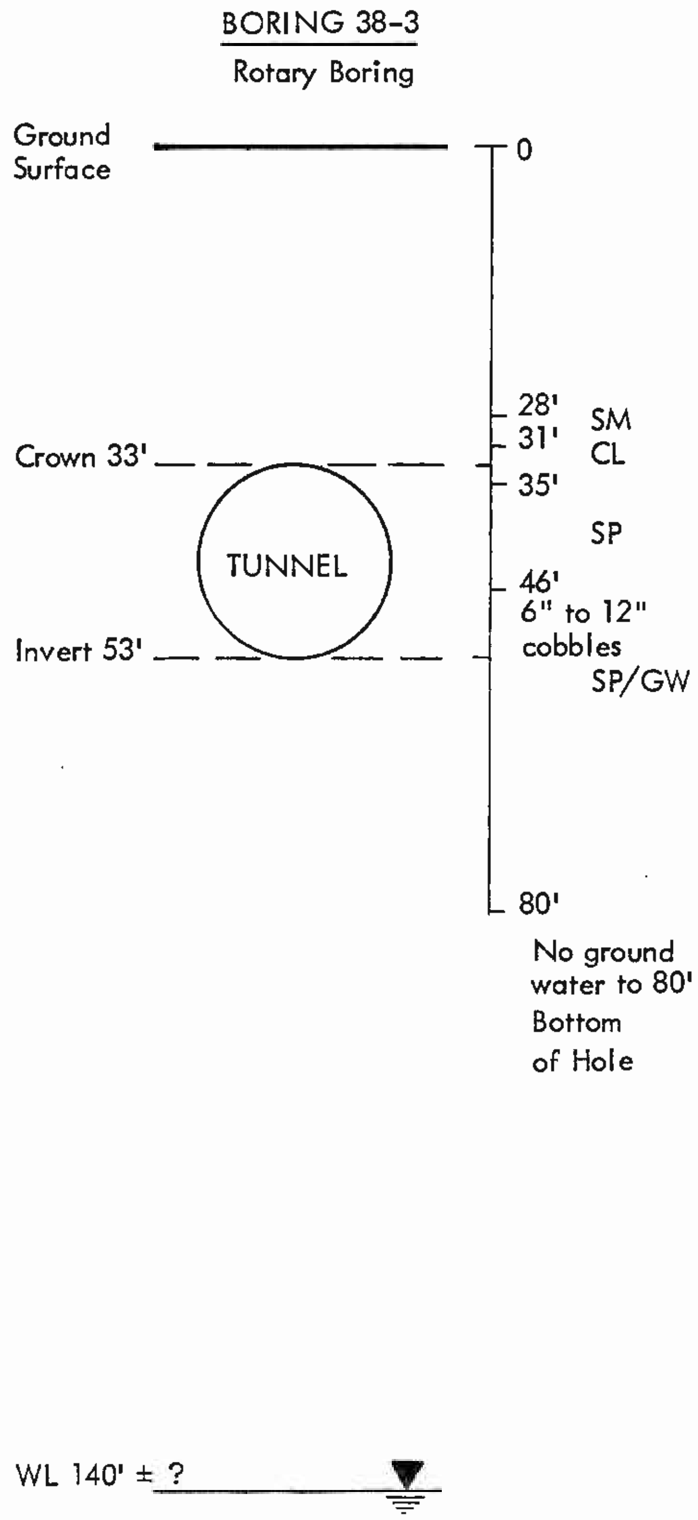

See Table A-1 of Appendix A for description of soil symbols

EXAMPLES OF STRATIGRAPHIC VARIATIONS

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Figure No.
7-8

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TABLE 7-1

GROUND WATER INFLOWS AND CAVING CONDITIONS

BORING No.	APPROXIMATE TUNNEL STATION	DEPTH TO CROWN-INVERT (ft)	CAVING DEPTH (ft)	DEPTH TO WATER LEVEL (ft)	WATER CHEMISTRY (TDS/pH)	GAS/OIL	REMARKS
34C	943	37 - 57	21 - 26	21	N/A	none	No caving 0 to 21 ft; excessive caving 21 to 26 ft; due to 25± gpm inflow at 21 ft
35B	976	72 - 92	37 & 61	none	760/7.7	none	Inflows 1± gpm at 37 and 61 ft
36A	1004	75 - 95	26 - 54	none	N/A	none	No ground water encountered; moderate caving from 26 to 54 feet due to mechanical action of drilling rig
37A	1028	47 - 67	38 - 60	none	N/A	none	No ground water encountered; minor caving 38 to 60 ft due to mechanical action of drilling rig
38A	1044	32 - 52	50 - 60	none	N/A	none	No ground water encountered; moderate caving 50 to 60 ft due to mechanical action of drilling rig

It is pointed out here that the elevation of the surface of the water level (Drawings 3 and 4) may vary and may well be deeper than that shown north of Boring 35-A. However, if it is near the elevation shown, and the Los Angeles Department of Water and Power stops pumping wells and/or there are exceptionally wet winters prior to construction, these levels could rise to the tunnel grades shown between about Stations 970+00 and 1000+00.

We believe that the soil conditions between Station 970+00 and the Crossover structure are suitable for the use of soft ground tunnelling techniques utilizing an open-face shield with hand and/or mechanical excavating equipment. We do not believe that tunnelling without a shield would be successful in these predominantly cohesionless-like alluvial soils described in this tunnel segment. Locally, shield tunnelling methods are expected to require means by which the face of the tunnel excavations can be supported to prevent running ground.

7.2 GROUND WATER - INFLOWS AND MINERAL ANALYSES

We believe that water seepage into the tunnel excavation from fresh, unfaulted, slightly fractured, fine-grained bedrock of the Topanga Formation will likely be of small amounts; i.e., dripping conditions.

Ground water inflows into the tunnel excavation from saturated alluvial materials between the Universal City Station and about Station 970+00 are likely to be significant with attendant caving problems, based on the records of man-sized auger Boring 34-C. The ground water inflows/caving conditions are summarized in Table 7-1.

The hydraulic properties of the Los Angeles River alluvium were investigated by performing a pump test in a well located about 700 feet west of the proposed Universal City Station (Drawing 2) in Weddington Park. The testing procedures and test results are presented in Appendix D, "Pump Test Results." The general hydraulic characteristics of the alluvium determined from the pump test are as follows:

- Transmissivity: approximately 24,000 gpd/ft
- Storage Coefficient: 0.008
- Specific Yield: 0.20 to 0.25
- Pump Test Discharge: 30 gpm for 470 minutes
- Saturated Thickness of Aquifer: 15 feet of clean sands and gravels
- Average Formation Permeability: Computed to be 1,900 gpd/ft^2 (~8.5 x 10^{-2} cm/sec)

We would like to point out that the saturated thickness of clean sands and gravels is considerably greater at the Universal City Station and the tunnel segment north of the Station. Therefore, appropriately designed dewatering wells are judged more applicable than a well point system. Ground water and caving problems associated with driving MWD's San Fernando Tunnel in alluvial deposits are discussed in the 1981 "Geotechnical Investigation Report, Metro Rail Project", Volume I, Section 6.1.

Dewatering performed in the area south of Bluffside Drive (approximately Station 941+00) may result in significant ground subsidence due to the nature of the soils and the greater depth of drawdown required. Estimated subsidence values presented in the A425 (Universal City Station) Geotechnical Report ranged from 1 to 3 inches. Dewatering north of Bluffside Drive is not expected to cause significant subsidence because required drawdown will likely be less than 25 feet and the saturated alluvium in this reach is generally dense and stiff.

Mineral analyses of the alluvial ground water from Boring 35-B indicate the total dissolved solids (TDS) are 760 parts per million (ppm) with a pH of 7.7. This is considered good quality water. We did not study the effect of corrosion. For details on corrosion, refer to the "Corrosion Control Final Report" dated June 20, 1983 performed for SCRTD by Professional Services Group, Inc., Waters Consultants Division, San Diego, California. Water quality analysis is provided in Appendix E.

7.3 ENGINEERING PROPERTIES OF TUNNELLING MATERIALS

The engineering properties of alluvium, and Topanga bedrock formation, as applied to tunnelling, are similar to those described in Section 5.6 and in Table 5-2, "Material Properties Selected for Static Design".

Squeezing of the Topanga formation (Tt) should not be a particular stability problem in normal shield tunnel construction operations because the average unconfined compressive strength is 70 psi. The alluvial material should not squeeze.

7.4 GAS AND OIL

In our judgement, the tunnel line segment in Design Unit A430 should be classified non-gassy. This classification is from the California Administrative Code, Title 8, page 684.18.

The entire tunnel segment is considered devoid of oil according to boring records along this segment.

7.5 CROSS PASSAGES

Southern California Rapid Transit District Drawings CSK-9 (Sheets 6 of 7 and 7 of 7) dated January 12, 1984, indicate 13 cross passages are planned at tunnel line stations 942+90, 950+40, 957+90, 965+40, 972+90, 980+40, 987+90, 995+40, 1005+60, 1013+10, 1020+60, 1028+10, and 1035+60. Based on SCRTD tunnel standard Drawings SD-053 and SD-054, the cross passage dimensions are about 20 feet long, 10 feet wide, and 12 feet high. The plans also indicate the finished opening will be supported by a 2-foot thick concrete liner.

The cross passages at Stations 942+90, 950+40, 957+90, 965+40 and possibly 972+90 should encounter similar stratigraphic, ground water and tunneling conditions described in Section 7.1.1. We believe mining of cross passages with hand and/or mechanical excavating equipment will require full support breast boarding and ground water control to maintain stability of the passage and to prevent loss of ground and settlement of the ground surface.

All other cross passages will be excavated above ground water levels (Drawings 3 and 4) in primarily cohesionless-like alluvium (A_1/A_2). Mining between twin-bore tunnels will be as described for the tunnel in Section 7.1.2.

7.6 VENT STRUCTURE

A vent structure is planned near the intersection of Lankershim Boulevard and Blix Street. Based on discussion with the Section Designer on April 18, 1984 it is understood that the vent structure will have four levels and a rectangular configuration. The total depth of the structure will be about 96 feet. The lower two levels will have plan dimensions of about 60 feet by 30 feet. The third level will have plan dimensions of 60 feet by about 95 feet and the uppermost level will be 60 feet by about 110 feet.

The subsurface conditions at Boring 36-B located about 130 feet east of the vent structure site are generally the same as those encountered at the crossover structure. Subsoils encountered at Boring 36-B consisted of predominately stiff sandy silt soils with layers of dense silty sand to a depth of 26 feet underlain by predominately dense to very dense sand and silty sand with gravel with thin interlayers of stiff sandy silt to the depth of Boring 36-B (80 feet). A standpipe piezometer was installed in Boring 36-B and all readings to dated have been "dry". Ground water levels are estimated to be below the tunnel grade at this location.

Based on the general similarity of conditions encountered at the vent structure to those encountered at the crossover structure it is our opinion that the vent structure construction excavation, and permanent wall design may be based on the recommendations presented in Section 6.4 and 6.9.

The vent structure may be satisfactorily supported on either spread footing foundations or mat foundations bearing on undisturbed dense or stiff natural alluvial soils. Foundation design should be in accordance with recommendations presented in Section 6.8. Settlements of the structure should occur as the structure is constructed due to the granular nature of the supporting soils. Assuming a uniform distribution of load at each level, differential settlements between the "overhanging" portions of the upper two levels and the remaining portion of those levels should not be a problem provided that the structure is constructed from bottom to top.

7.7 SPECIAL TUNNELLING PROBLEM AREAS

The presence of gravel/cobbles up to 12 inches in diameter, although not preeminent, should be anticipated and may well dictate the type of mechanical excavation equipment as well as rate of which excavation can be made through "cobbly" horizons.

Due to the relatively shallow ground cover over the tunnel as it enters the Crossover structure, underground conditions should be established prior to start of construction for such items as tiebacks and/or foundation along Lankershim Boulevard.

7.8 SEISMIC CONSIDERATIONS

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

In general, there appears to be a low potential for liquefaction along the A430 alignment. Based on measured ground water levels and the proposed tunnel grades, the only portion of the tunnel at or below current ground water levels will be from Universal City Station to about Station 970+00. Although a high potential for liquefaction of the upper alluvium was identified at Universal City Station, the lower alluvium and Topanga Formation bedrock at the tunnel grade near the station are considered to have a very low potential for liquefaction. Data on the saturated alluvium north of Universal City Station where the tunnel emerges from the Topanga Formation is very limited due to the required wide spacing of borings. The boring in this area (35-A) indicates the granular alluvium to be dense to very dense and interlayered with stiff clayey soils. Based on this limited information combined with the fact that planned tunnel grades are generally below a depth of 65 feet, the potential for liquefaction is considered low.

7.9 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- Additional Field Exploration: Due to the lack of data on ground water conditions along the tunnel alignment, we recommend drilling five piezometer borings (ground water observations wells) to firm up water levels between tunnel Stations 900+00 and 1040+00. These borings should be located at about Stations 965+00, 983+50, 997+00, 1021+50 and 1035+00. The borings at Stations 965+00 and 983+50 would also help define the depth to bedrock.

- Pump Test: It is recommended that a pumping test in the thick saturated Los Angeles River alluvium be performed at the junction of Design Unit A425/A430 to evaluate the pumping and dewatering characteristics. The test well should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

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

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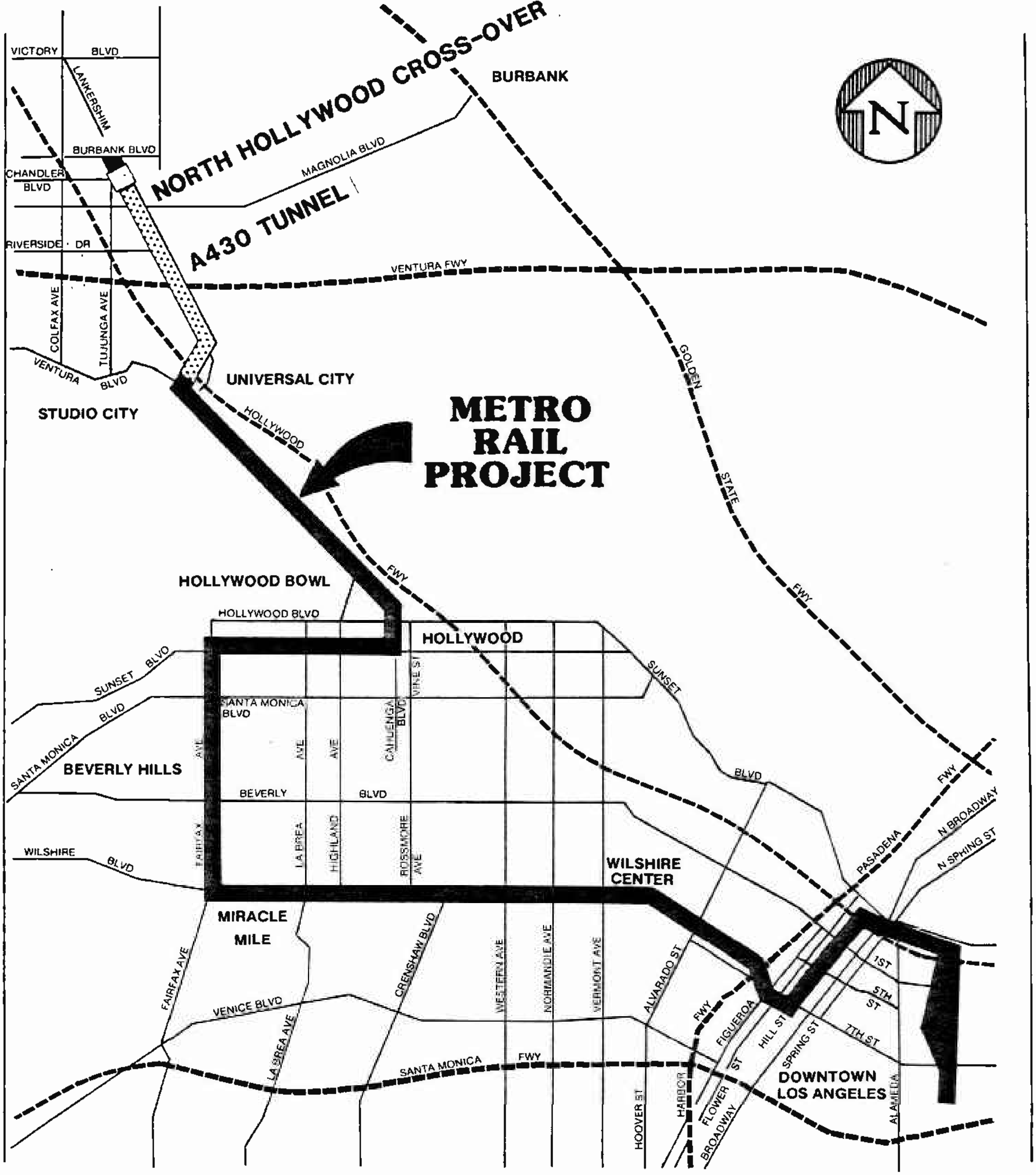
Drawings

CCI/ESA/GRC



VICINITY MAP

DESIGN UNIT A430

Southern California Rapid Transit District

METRO RAIL PROJECT

Project No.

83-1140

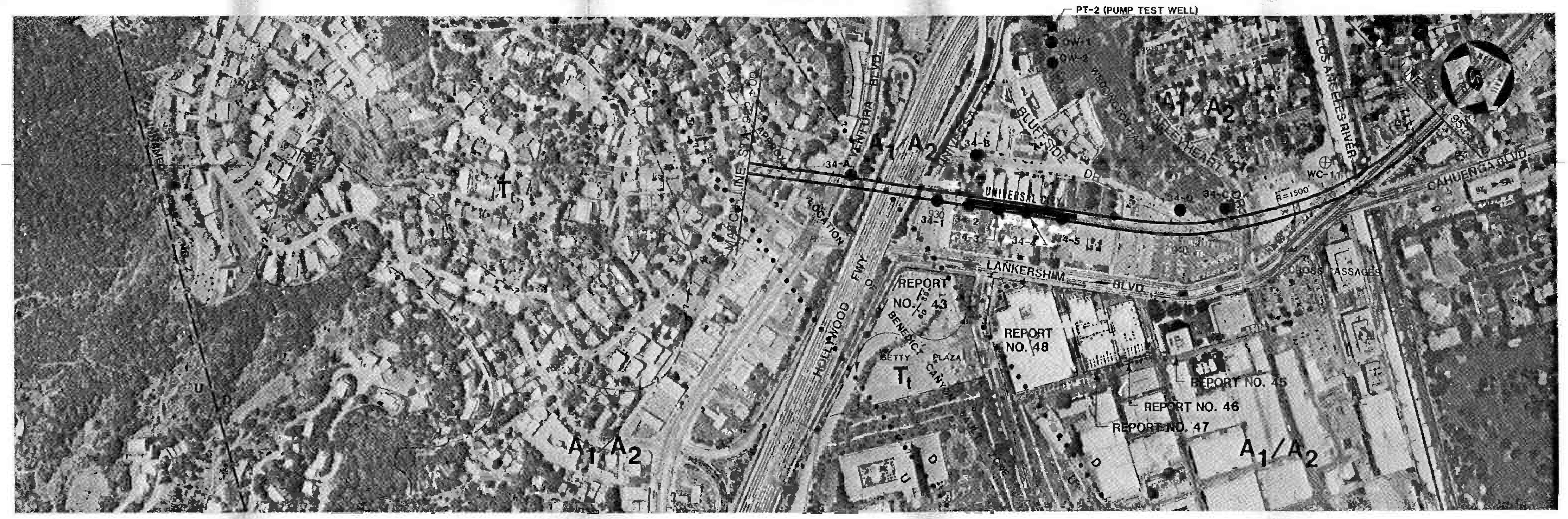
Drawing No

1

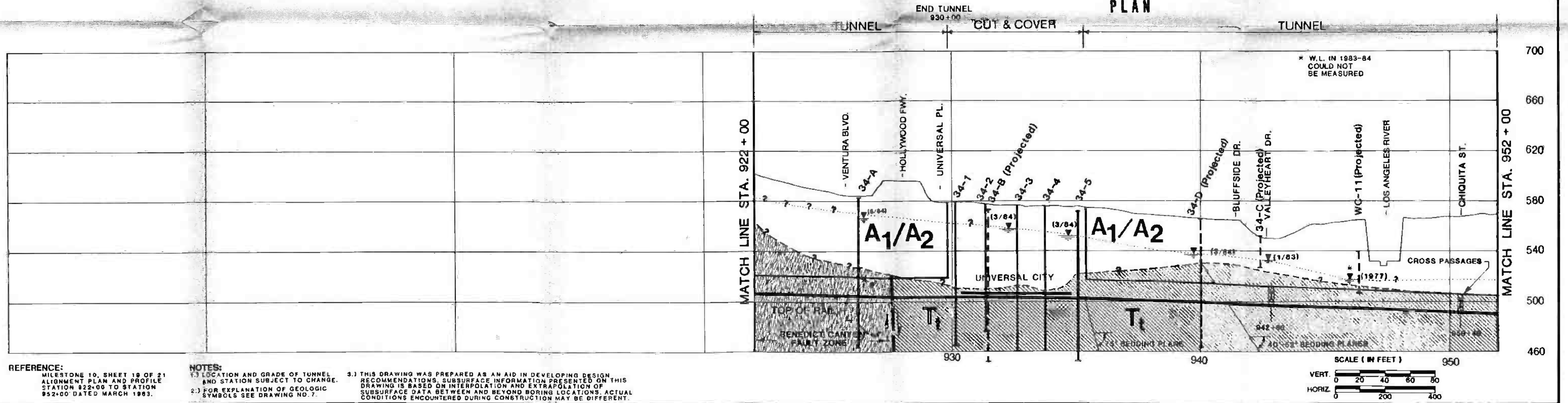
Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication 5/5/4 by

D-1



PLAN



REFERENCE:
MILESTONE 10, SHEET 19 OF 21
ALIGNMENT PLAN AND PROFILE
STATION 922+00 TO STATION
952+00 DATED MARCH 1983.

NOTES:

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

2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 7.

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

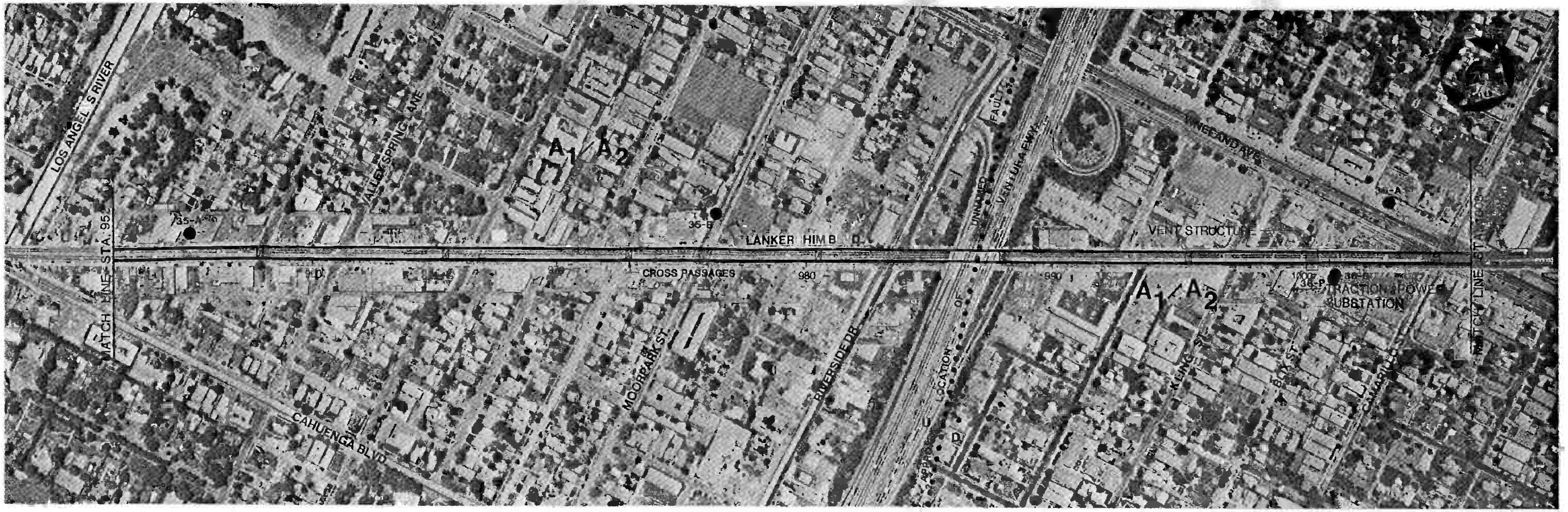
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

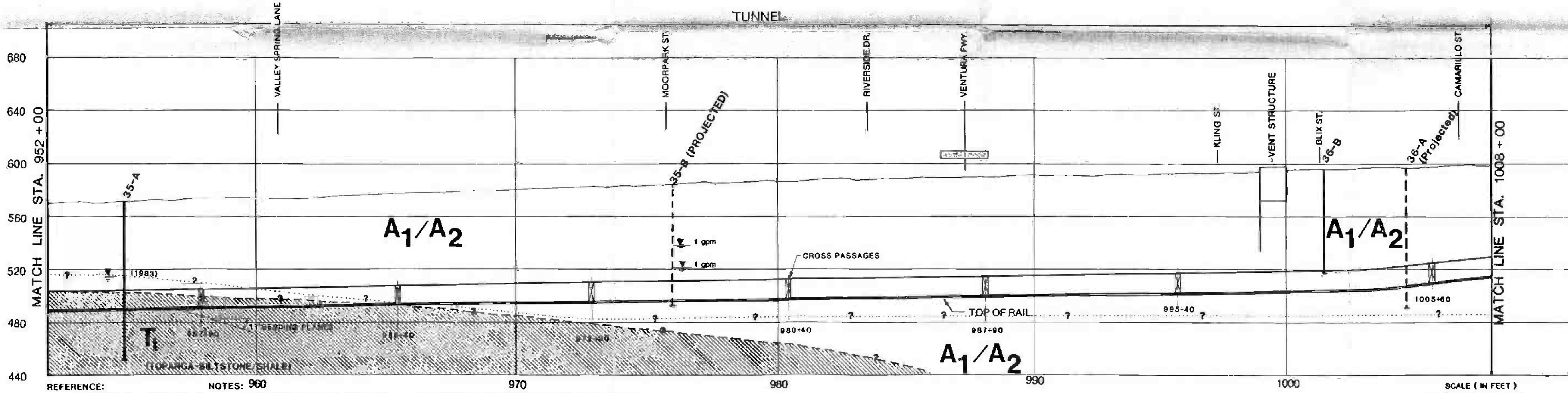
CCI/ESA/GRC
General Geotechnical Consultants
Submitted R.M. Pride Date 5-14-84

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

DESIGN UNIT A430
LOCATION OF BORINGS AND GEOLOGIC SECTION

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





REFERENCE:

MILESTONE 10, SHEET 20 OF 21 ALIGNMENT PLAN AND PROFILE STATION 952+00 TO STATION 1008+00, DATED MARCH, 1983

NOTES:

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

2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO.7

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted T.H. Spellman Date 5-25-84

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DESIGN UNIT A430
LOCATION OF BORINGS AND GEOLOGIC SECTION

83-1140
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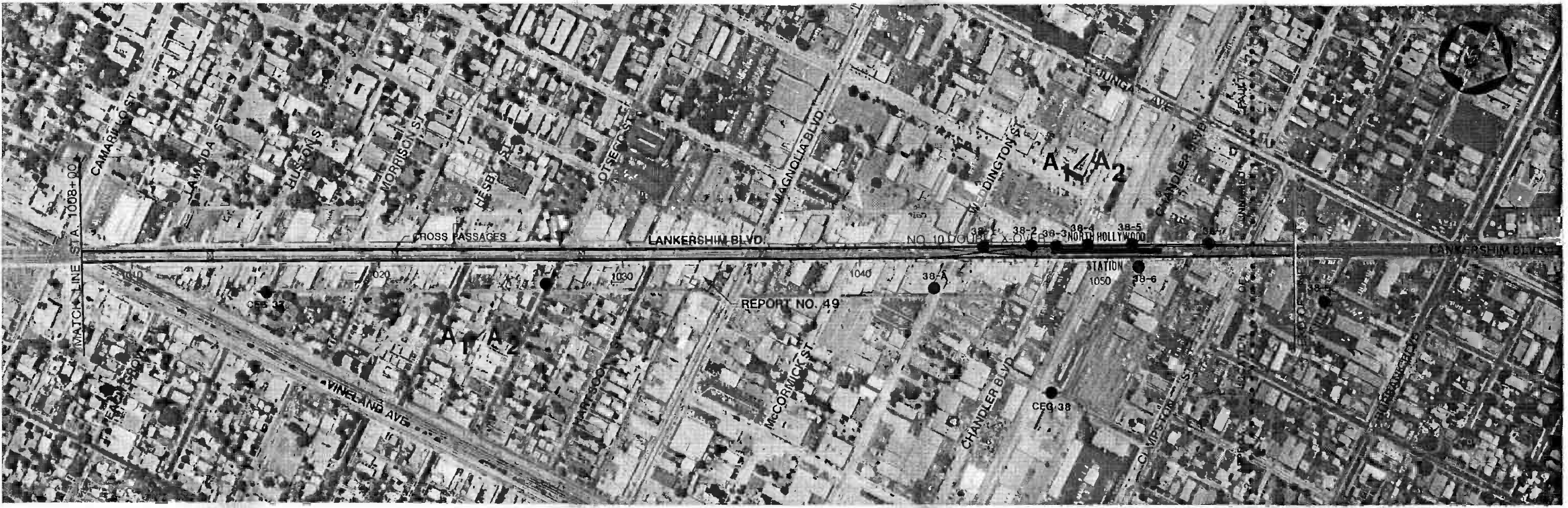
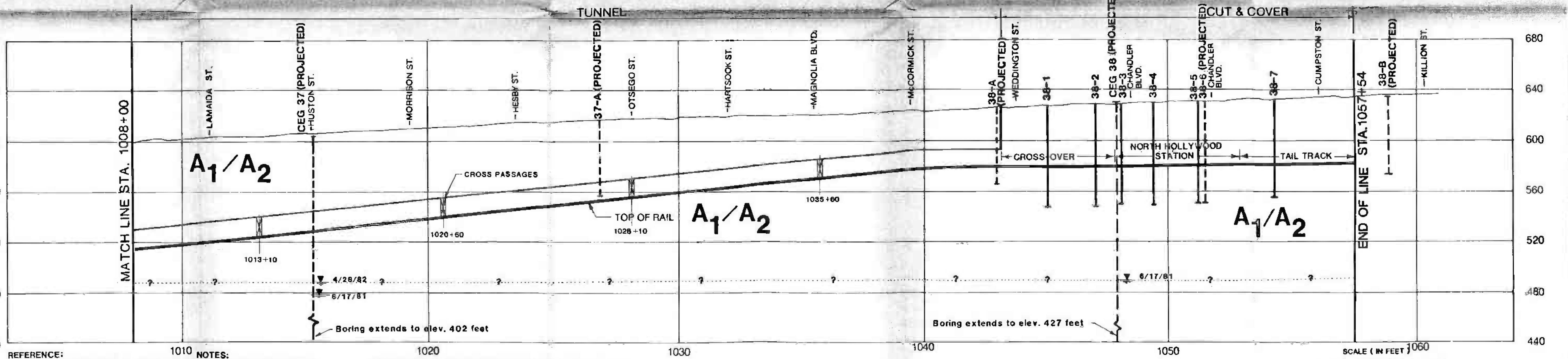
SHEET NO.





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 10 CHANGE.

2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 7

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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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 H.A. Spellman Date 5-25-84

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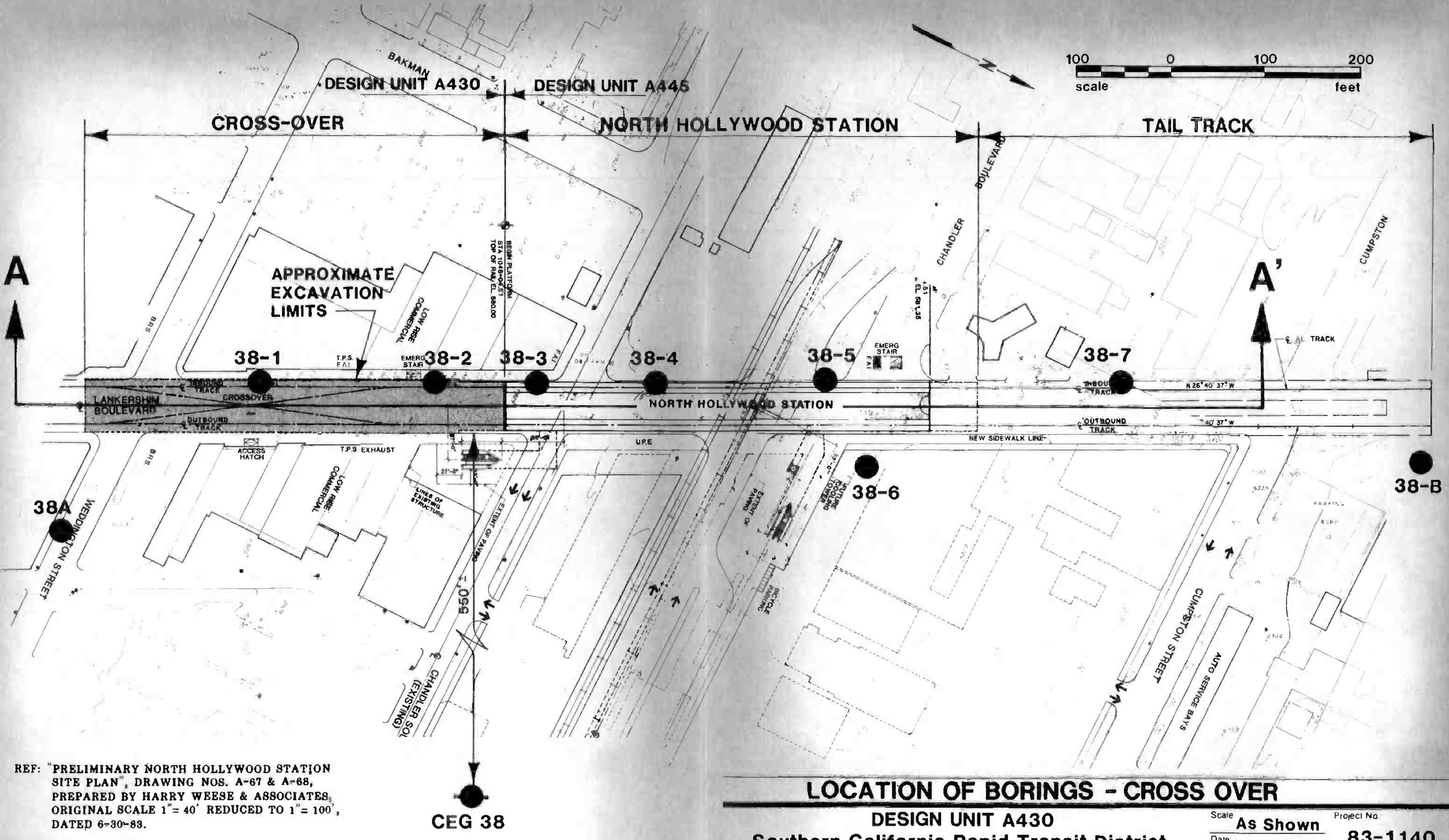
DESIGN UNIT A430
LOCATION OF BORINGS AND GEOLOGIC SECTION

CONTRACT NO. 83-1140
DRAWING NO. 4
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.

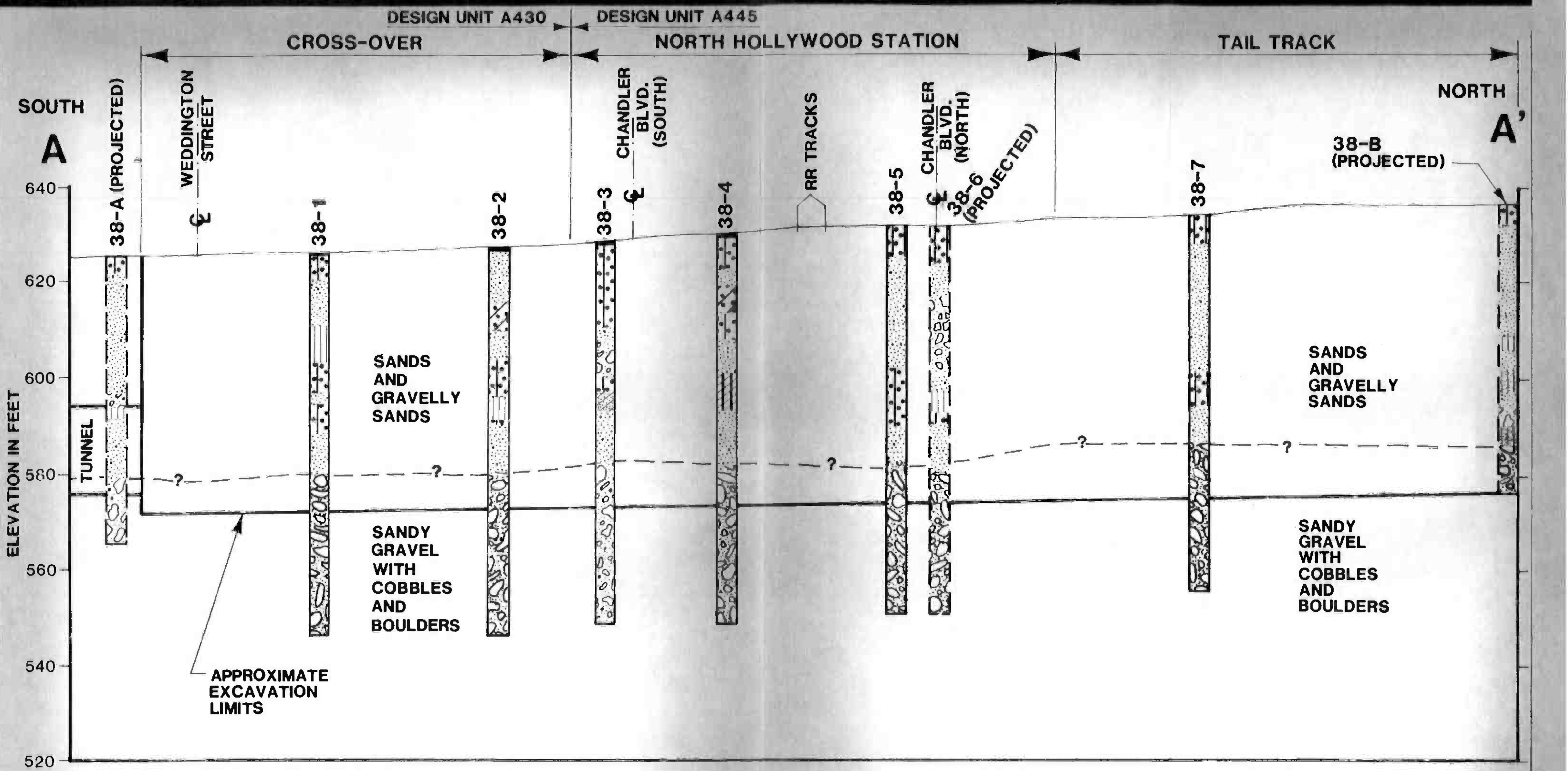
NOTES: 1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 6
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 7

LOCATION OF BORINGS - CROSS OVER

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

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



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. 5

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 7

SUBSURFACE SECTION A-A'

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale	As Shown	Project No.	83-1140
Date	May, 1984		
Prepared by	MMT	Drawing No.	6
Checked by	RG		
Approved by	JAD		

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

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

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
▭	CEMENTED ZONE
▓	META—SANDSTONE
▒	BASALT
▒	BRECCIA
▤	SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A430
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 7

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A430 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, and the 1983 borings drilled for this A430 investigation. The specific boring logs included are summarized below:

- 1980 and Earlier

  37, 38, WC-11

- 1983

  34-C, 34-D, 35-A, 35-B, 36-A, 36-B, 37-A, 38-A, 38-1 through 38-3

Locations of the borings are shown on Drawings 2, 3 and 4. Ground water observation wells (piezometers) were installed in the borings listed in Section 5.3 (Table 5-1). Geophysical downhole surveys were made for the 1981 investigation at Boring CEG-38 within the A430 investigation site.

The borings were drilled to depths generally ranging from 26 to 200+ feet. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was generally good.

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

A.2 FIELD STAFF AND EQUIPMENT

A.2.1 Technical Staff

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

A.2.2 Drilling Contractor and Equipment

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

A.3 SAMPLING AND LOGGING PROCEDURES

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

A.3.1 Sampling

In the overburden at about 10-foot intervals, the Converse ring sampler was driven using a down-hole 320-pound to 340-pound slip-jar hammer with an 18-inch drop. The Converse sampler was followed with a standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the alluvium and Fernando Formation were encountered, the borings were generally continuously sampled using a Pitcher Barrel sampler and Converse ring sampler.

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

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

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

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

A.3.2 Field Classification of Soils

All soil types were classified in the field by the field geologist using the "Unified Soil Classification System". Based on the characteristics of the soil, this system indicates the behavior of the soil as an engineering construction material. (For a more complete discussion of the Unified Soil Classification System, refer to Corps of Engineers, Technical Memorandum No. 3-357, March 1953, or Department of the Interior, Bureau of Reclamation, Earth Manual, 1963.) Although particle size distribution estimates were based on volume rather than weight, the field estimates should fall within an acceptable range of accuracy. A description of the Unified Soil Classification Symbols used on the borings logs is presented in Table A-1 below.

TABLE A-1
UNIFIED SOIL CLASSIFICATION SYMBOLS

GRANULAR SOILS

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

FINE-GRAINED SOILS

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

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

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

A.3.3 Field Description of the Formations

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

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

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

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

Bedrock description terms used on the boring logs are given on Table A-3. In addition, the rock quality designation (RQD) based on core recovery is shown on the boring logs in the "Remarks" column. The RQD percentage represents the approximate percentage of intact pieces of core that are more than 10 cm (4 inches) long from a particular core run.

A.4 PIEZOMETER INSTALLATION

Standpipe piezometers were installed in borings 34-C, 34-D, 35-A, 37 and 38. Procedures for piezometer installation were as follows:

A 2-inch diameter plastic ABS pipe was installed in the boring. At least the lower 20 feet of the ABS pipe was perforated, and the annulus of the boring around the perforated portion of the pipe was backfilled with a coarse sand/pea gravel aggregate. Concrete/bentonite slurry was used to backfill around the non-perforated portion of the pipe to prevent surface water from artificially recharging the gravel-packed hole or contaminating local ground water. After the piezometer was installed, the boring was flushed using air lift provided by a trailer-mounted air compressor. The piezometer was covered with a standard 7-inch diameter steel water meter cap held at surface grade by a grouted in-place 3- to 4-foot long, 5-inch diameter plastic sleeve. Ground water data obtained from the standpipe piezometers are presented in Section 5.3 of the text.

TABLE A-3 Bedrock Description Terms

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

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

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

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

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

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

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

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BORING LOG 34C

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

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

Sheet 1 of 2

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

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

Sheet 2 **of** 2

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BORING LOG 34D

Proj: DESIGN UNIT A425 **Date Drilled** 1/2-3/83 **Ground Elev.** 565'

Drill Rig MAYHEW 1000 **Logged By** G. Halbert **Total Depth** 101'

Hole Diameter 6" **Hammer Weight & Fall** SPT 140lb. 30"

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

Sheet 1 of 5

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

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

Sheet 2 of 5

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

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

Sheet 3 of 5

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

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

Sheet 4 of 5

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

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

Sheet 5 of 5

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BORING LOG 35A

Proj: DESIGN UNIT A430 **Date Drilled** 1-31-83 **Ground Elev.** 572

Drill Rig Mayhew 1000 **Logged By** G. Halbert **Total Depth** 121'

Hole Diameter 6" **Hammer Weight & Fall** 320 lb @ 24"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.3 ASPHALT			RD	
	ML	0.3 16.0 SANDY SILT: dark yellow brown; stiff; moist				
2						
4						
6						
		occasional sandier layers				
8						
10			C-1		DR	recovery 1.1/1.5
			J-1	4	SS	pen: 3.0 tsf
12				5		
				7		
					RD	
14						
16	SP	16.0-25.0 SAND: mottled dark yellow brown and medium yellow brown; moist; very dense				
18						
			C-2		DR	
20						

Sheet 1 **of** 6

Project DESIGN UNIT A430 Date Drilled 1-31-83 Hole No. 35A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	16.0-25.0 SAND: (continued) dark yellow brown; moist; very dense	J-2	17 28 42	SS	recovery 1.5/1.5
					RD	
22						
		with silty zones				
24						
	ML	25.0 29.0 SANDY SILT: moderate yellow brown				
26						
28		grading finer				
	CL	29.0-34.0 SILTY CLAY: medium yellow brown; with sand; very moist; stiff				
30			J-3	2 5 8	SS	recovery 1.5/1.5 pen: 1.5 tsf
					RD	
32						
34	GW	34.0-51.0 SANDY GRAVEL: very dense; white, light brown & black				
36						
38						
40						
42						
			C-3		DR	
44						

Sheet 2 of 6

Project DESIGN UNIT A430 Date Drilled 1/31/83 Hole No. 35A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	GW	34.0-51.0 SANDY GRAVEL: (continued) with cobbles	J-4	100	SS	recovery 0.0/0.5
46	(SM)	45.6-47.0-silty sand lens				
48		interbedded silt lenses				
50		gravel to 1"	C-4		DR	recovery 0.9 /1.0
	CL (GW)	51.0-55.5 SILTY CLAY: with interbedded sand and gravel	J-5	27 18 26	SS	recovery 1.5/1.5
52						
					RD	
54		54.2-54.8-gravelly lens				
	CL	55.5-68.0 SILTY CLAY: with fine sand; mottled olive gray and orange; moist; hard				
56						
58						
60		light olive gray	J-6	26 26 48	SS	recovery 1.1/1.5 pen: 4.0 tsf
62						
64						
66						
68						

Sheet 3 of 6

Project DESIGN UNIT A430 Date Drilled 1/31/83 Hole No. 35A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
		BEDROCK				
68		68.0-120.5 SILTSTONE: olive black moist; laminated with fine sand; wavy bedding; with interbedded shale and sandstone; greenish black and olive gray; Physical Condition low hardness friable to weak strength; dominantly shale			RD	
70			J-7	45 60	SS	recovery 0.7/0.7 refusal at 8" pen: 4.0 tsf
					RD	
72						
74						
76						
78						
	17°					
80		olive gray	J-8	80/5"	SS	recovery 0.4/0.4
					RD	
82						
84						
86						
88						
		olive black; low hardness; fissile, wavy bedding				
90			C-5		DR	
					RD	
92						

Sheet 4 of 6

Project DESIGN UNIT A430 **Date Drilled** 1/31/83-2/1/83 **Hole No.** 35A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		68.0-120.5 <u>INTERBEDDED SILTSTONE AND SANDSTONE</u>: (continued) olive black, laminated, low hardness			RD	
94						
96						
98		occasional laminations of slightly harder brownish black shale				
100		dominantly shale; with fine to medium sand				
102						
104						
106						
108						
110		becoming sandier; Physical Condition: low hardness; friable to weak strength, fissile				
112						
114						
116						

Sheet 5 of 6

Project DESIGN UNIT A430 Date Drilled 1/31/83-2/1/83 Hole No. 35A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		68.0-120.5 SHALE WITH INTERBEDDED SUBORDINATE SANDSTONE: (continued)			RD	
118						
120	23°		C-6		DR	
		End of boring 120.5'				installed 120' pvc tubing, perforated from 80' to 120'
122						
124						
126						
128						
130						
132						
134						
136						
138						
140						

Sheet 6 of 6

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BORING LOG 35B

Proj: DESIGN UNIT A430 **Date Drilled** 10-4-83 **Ground Elev.** 584

Drill Rig Bucket **Logged By** J. Stellar **Total Depth** 92'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 A. C. PAVEMENT				
		0.5-5.0 FILL SILTY SAND: with broken brick; dark brown; moist; loose to medium dense				hole stands well minor caving @ 37.5' and 61.0'
2						
4						
	SP	5.0-13.0 ALLUVIUM SAND: with layers of silty sand; medium dense				
6						
8						
10						
12						
	ML	13.0-18.0 SILT: dark brown, moist, firm; with layers of sandy silt				
14						
16						
18						
	SP	18.0-28.0 SAND: light brown; moist; medium dense				
20						

Sheet 1 of 4

Project DESIGN UNIT A430 **Date Drilled** 10-4-83 **Hole No.** 35 B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	18.0-28.0 SAND: (continued)				
22						
		with layers of silt and sandy silt				
24						
26						
28	ML	28.0-35.0 SANDY SILT: with layers of silt; dark brown; moist to very moist; firm				
30						
32						
34						
	SP	35.0-39.0 SAND: light yellow brown; moist dense; with layers of gravelly sand				
36						
						37.5 minor caving
38						
	SW	39.5-41.4 GRAVELLY SAND: orange brown; moist dense				
40						
42	ML	41.4-46.0 SANDY SILT: with layers of silty sand				
44						

Sheet 2 of 4

Project DESIGN UNIT A430 **Date Drilled** 10-4-83 **Hole No.** 35 B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	4.4-46.0 SANDY SILT: (continued)				
46	SW	46.0-47.0 GRAVELLY SAND: with cobbles to 6" wet				perched water @ 46' 1 gpm
	ML	47.0-54.0 SILT: dark grey; moist to very moist; with layers of clayey silt; stiff				
48		color change to blue grey				
50						
52						
54	SP	54.0-56.0 SAND: orange brown; moist; layers of silty sand; with gravel				bag sample at 55'
56	SW	56.0-66.0 GRAVELLY SAND: dense; moist; gravel to 2"				
58						
60		becomes wet				61.0' minor caving perched water @ 62' 1 gpm
62						
64						
66	ML	66.0-69.0 CLAYEY SILT: blue grey; moist to very moist; silty; with layers of silty sand				
68						

Sheet 3 of 4

Project DESIGN UNIT A430 **Date Drilled** 10-4-83 **Hole No.** 35 B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	ML	66.0-69.0 CLAYEY SILT: (continued)				
70	ML	69.0-72.0 SILT: grey; moist; very stiff; with layers of sandy silt				
72 74 76 78 80 82 84	SP	72.0-85.0 SAND: grey; very moist, dense; with layers of silty sand				
86 88 90	ML	85.0-91.0 SILT: dark brown to grey; moist to very moist; stiff to very stiff; with layers of silty sand				minor caving @ 37.5' & 61.0' - due to perched water; water sample taken
92	SP	91.0-92.0 SAND: grey; moist; dense; with layers of silty sand				

B.H. 92.0 Terminate Hole

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BORING LOG 36A

Proj: DESIGN UNIT A 430
Date Drilled 1-31-83
Ground Elev. 598'
Drill Rig B. AUGER
Logged By D. Gillette
Total Depth 105'
Hole Diameter 36"
Hammer Weight & Fall

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 CONCRETE				
	ML	0.5-3.0 ALLUVIUM SANDY SILT: greyish brown; moist				
2						
						hole stands well
	SM/ SP	3.0-17.0 SILTY SAND/SAND: pale brown; moist				
4						
6						
8						
10						
12						
14						
16	SW	17.0-34.0 SAND: moderate yellowish brown; and moderate brown; moist; with fine to coarse gravel				
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A 430 Date Drilled 1-31-83 Hole No. 36A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	17.0-34.0 SAND: (continued) contains lenses of cobbles to 5"; occasional boulders; loose to medium dense 21.0 sandy silt streaks (1"-4")				hole stands well
22						
24						
26						26.0-54.0' hole subject to caving
28						
30						hole diameter increase 6" to 2' due to caving
32		32.0-33.5 sandy silt streaks, reddish brown				
34	SP/GP	34.0-53.0 SAND AND GRAVEL: dark reddish brown and moderate yellowish brown; moist trace silt;				intervals hole stands well - 34.0-35.5; 37.0-40.0'
36		37.0-40.0 silt streaks				
38						
40		40.5-41.5 clay streaks; medium grey 41.5 sand lens				
42		43.0-45.0 sand layer; ferrous oxide cement				
44						

Sheet 2 of 5

Project DESIGN UNIT A430 Date Drilled 1-31-83 Hole No. 36A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP/GP	34.0-53.0 **SAND AND GRAVEL**: (continued) 45.0 gravel lens (6") 34.0-53.0 sand occurs in clean lenses & mixed with gravel & silt gravel and cobble lenses (4-6" thick)				
46						
48	(ML)	48.0-49.5 silt streaks; reddish brown				
50						
52						
	SW	53.0-105.0 **SAND:** moderate yellowish brown and moderate brown; contains medium grey silty inclusions;				
54		55.0 clayey sand streak				54-105' hole stands well
56						
58		moist; occasional cobbles and boulders 59.0 silty sand streak				
60						
62		63.0-64.0 silty sand				
64		64.0-66.5 cobbles 65.0-66.5 clean sand lens				
66						
68						

Sheet 3 of 5

Project DESIGN UNIT A 430 Date Drilled 1-31-83 Hole No. 36A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	53.0-105.0 SAND: (continued) 68.0-72.0 clay streaks & lenses; medium grey				hole stands well
70						
72						
74						
76		76.5-78.0 silt steaks (6-10")				
78	(ML)	79.0 stratified sand 78.0-83.0 sandy silt				
80						
82						
84						
86						
88						
90		90.0-91.5 silt streaks (6-12")				
92						

Sheet 4 of 5

Project DESIGN UNIT A430 **Date Drilled** 1-31-83 **Hole No.** 36A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SW	53.0-105.0 <u>SAND</u>: (Continued)				
						hole stands well
94						
96						
98						
		99.5-100.0 silt streaks (6-10")				
100						
102						
104						
		B.H. 105.0 Terminate hole				from 1'-105' hole backfilled with slurry and capped with concrete to street grade
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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

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

BORING LOG 36B

Proj: DESIGN UNIT A430
Date Drilled 21-22 Nov. '83
Ground Elev.
Drill Rig FAILING 750
Logged By STEVE SLAFF
Total Depth 80.4
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lbs., 30" SS
320 lbs., 18" DR

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5	CONCRETE:			GB	
			ALLUVIUM:				
	ML	0.5-13.5	SANDY SILT: dark yellowish brown; moist; very soft;			AD	
2					1	DR	0.8/1.0 recovery
			3.0 becoming soft	C-1	1		
						AD	
4					2	SS	1.1/1.5 recovery
			5.0 becoming firm	J-1	2		
					4		
6						AD	
						RD	
8			8.0 becoming stiff				
			increase in sand content; trace fine gravel	J-2	4	SS	1.0/1.5 recovery
					5		
					7		
10						RD	
			11.0 becoming very stiff				
12			12.0 sand content increasing		8	DR	0.9/1.0 recovery
				C-2	14		
		13.5-18.0	SILTY SAND: grayish orange; moist; medium dense			RD	
14					11	SS	0.9/1.5 recovery
				J-3	16		
					12		
16						RD	
18							
	ML	18.0-26.2	SANDY SILT: moderate yellowish brown; moist; stiff to very stiff	J-4	6	SS	1.1/1.5 recovery
20					8		

Sheet 1 **of** 4

Project DESIGN UNIT A430 **Date Drilled** 21-22 NOV '83 **Hole No.** 36-B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	ML	18.0-26.2 SANDY SILT: (continues)	J-4	7	SS	1.1/1.5 recovery
					RD	
22		sand content decreasing		11	DR	0.9/1.0 recovery
			C-3	18		
					RD	
24		sand content increasing				
			J-5	8 9 9	SS	1.2/1.5 recovery
26					RD	rig chatter
	SM	26.2-53.5 SILTY SAND: Dark yellowish brown; moist; dense: with interbedded sandy silt				
	GP					
28	SM GP SM	27.0-27.4 27.9-28.3 31.2-31.7 32.0 } lenses of sandy gravel fine gravel; decreasing fines				
			J-6	16 20 19	SS	0.9/1.5 recovery
30	ML SM GP SM				RD	switched to 4 7/8" tricone rock bit
32				23	DR	rig chattering 1.0/1.0 recovery
			C-4	33		
					RD	
34	(SP)	34.8 sand fine to medium/ becoming very dense	J-7	19 27 35	SS	0.9/1.5 recovery
36					RD	slight rig chatter
38		some zones light brown				
40		some interbedded layers of clean sand, becoming medium dense	J-8	11 12 11	SS	0.7/1.5 recovery rig chatter
					RD	
42				45	DR	0.6/0.1 recovery
			C-5	50		
44	GP SM	43.2-43.5 lens of sandy gravel			RD	

Sheet 2 **of** 4

Project DESIGN UNIT A430 Date Drilled 21-22 NOV '83 Hole No. 36-B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM ML	26.2-53.5 SILTY SAND: (continues)	J-9	7	SS	0.5/1.5 recovery
	SM	44.2-44.8 lens of sandy silt, becoming very dense with gravel		22		
				60		
46					RD	
48	(SP)	some interbedded layers of clean sand				
			J-10	6	SS	0.3/1.5 recovery
50	ML	50.0-50.8 lens of sandy silt		24		
	SM			32		
					RD	rig chatter
52				25	DR	0.8/1.0 recovery
			C-6	52		21 Nov. '83 22 Nov. '83
	ML	53.5-57.5 SANDY SILT: Pale yellowish brown; wet; stiff;			RD	
54			J-11	5	SS	1.5/1.5 recovery
				5		
				8		
56					RD	
58	SM	57.5-64.4 SILTY SAND: Moderate yellowish brown; moist to wet; very dense				
			J-12	26	SS	0.7/1.4 recovery refusal at 16 1/2"
60				41		
				50/4.5"		
		color change to pale yellowish brown			RD	
62	GP	color mottled with dark yellowish orange, and grayish orange		48	DR	0.9/0.9 recovery refusal at 9 1/2"
	SM		C-7	50/4.5"		
		63.0-63.3 sandy gravel			RD	
64	ML	64.4-69.0 SANDY SILT: Mottled-light olive gray, dusky yellow, light brown; moist; very stiff; iron oxide stains	J-13	7	SS	1.3/1.5 recovery
				9		
				16		
66					RD	slight rig chatter
68						

Sheet 3 of 4

Project DESIGN UNIT A430 **Date Drilled** 11/21-22/83 **Hole No.** 36B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	ML	64.4-69.0 SANDY SILT: (continued)			RD	
	SM	69.0-71.5 SILTY SAND: mottled-pale yellowish brown and moderate brown; moist; very dense;	J-14	30 50-5.5"	SS	0.6/1.0 recovery refusal @ 11½"
70					RD	
				19	DR	1.0/1.0 recovery
	ML	71.5-77.8 SANDY SILT: mottled moderate yellowish brown and light brown; moist; hard	C-8	22		
72					RD	
		71.5-71.8 light olive grey 73.7 color change to dusky yellow				
74			J-15	7 12 38	SS	1.5/1.5 recovery
76					RD	
78	SM	77.8-80.4 SILTY SAND: dark yellowish orange; wet; very dense				
			J-16	12 43 50-5.5"	SS	1.4/1.4 recovery refusal @ 17" 11-22-83
80		B.H. 80.4'Terminate Hole				Installed 2' diameter ABS piezometer from 0.0-80.4', perforated from 60.4' to 80.4'. Backfilled annulus with pea gravel
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

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

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

BORING LOG CEG 37

Proj: DESIGN UNIT A430
Date Drilled 12/8-10/80
Ground Elev. 604
Drill Rig Failing 1500
Logged By J.D. Gallinatti
Total Depth 202½
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	SM	0.0-20.5 <u>SILTY SAND</u>: dark yellowish brown; moist; loose to medium dense			AD	started drilling 12/8/80
2						
4						
6						
8						
10			J-1	3	SS	
				5		recovery 1.0/1.5
				6		pen: 1.5 tsf
12					RD	(breakapart) 2/4/81
14						
16						
18						
20						

Sheet 1 of 9

Project DESIGN UNIT A430 **Date Drilled** 12/8/83 **Hole No.** CEG 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	0.0-20.5 SILTY SAND: (continued)	C-1		DR	recovery 0.8/1.0
	SP	20.5-23.5 SAND: very pale orange, very dense		12 22 30	SS	no recovery
22						
					RD	
24	SM ML	23.5-29.0 SILTY SAND/SANDY SILT: moderate brown, scattered gravel; very stiff/medium dense				
26						
28						
	SP	29.0-30.5 GRAVELLY SAND:				
30			J-2	5 8 11	SS	recovery 0.4/1.5 pen: 4.5 tsf
	ML	30.5-33.5 SANDY SILT: dark moderate brown; very stiff				
					RD	
32						
34	SP SW	33.5-54.0 GRAVELLY SAND: light yellowish brown; very dense				
36						
38						
40			C-2		DR	recovery 0.5/1.0
			J-3	15 50	SS	recovery 0.5/0.75
42		42.0-45.0-increased gravel content			RD	refusal at 9" chatter from 42' to 45'
44						

Sheet 2 **of** 9

CEG

Project DESIGN UNIT A430 **Date Drilled** 12/9/80 **Hole No.** 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP/SW	33.5-54.0 GRAVELLY SAND: (continued)			RD	
46						
48						
50			J-4	41 50	SS	refusal at 11" recovery 0.4/0.9
					RD	50.9' stop drilling for 12/8/90 (4:30) start drilling 7:00 on 12/9/80
52						
54	SP	54.0-74.0 SAND: very light brown; fine sand; moist to dry; very dense				
56						
58						
60		60.0 some light brown staining	C-3		DR	recovery 1.0/1.0
			J-5	18 32 36	SS	recovery 1.0/1.5
62					RD	
64						
66						
68						

Sheet 3 of 9

Project DESIGN UNIT A430 **Date Drilled** 12/9/80 **Hole No.** CEG 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	54.0-74.0 SAND: (continued)			RD	
70		70.0-fine sand	J-6	50	SS	refusal at 5"
					RD	recovery 0.4/0.4
72						
74	CL CH	74.0-78.0 SANDY CLAY: light brown; firm to stiff				
76						
78	SP	78.0-83.0 SAND: pale yellowish brown; very dense, slightly moist				
80			C-4		DR	recovery 0.6/1.0
			J-7	50	SS	refusal at 6"
					RD	recovery 0.5/0.5
82						
	ML	83.0-85.0 SANDY SILT: moderate brown; firm				
84						
	SP	85.0-108.0 SAND: moderate yellowish brown; very dense; slightly moist				
86						
88		88.0-94.0-occasional thin sandy silt layers 0.5"-1.0" thick				
90			J-8	50	SS	refusal at 4"
					RD	recovery 0.3/0.3 pen: 0.75 tsf
92						

Sheet 4 **of** 9

CEG

Project DESIGN UNIT A430 Date Drilled 12-9-80 Hole No. 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SP	85.0-108.0 SAND: continued			RD	
94						
96						
98						
100			C-5		DR	0.6/0.6 recovery
			J-9	50	SS	refusal at 6" 0.4/0.5 recovery
					RD	
102						
		103.0-108.0 organic fragments				
104						
106						
108	CL	108.0-115.0 SANDY CLAY: moderate brown; hard; slightly moist				
110			J-10	24 32 40	SS	1.5/1.5 recovery
					RD	
112						
114						
	SP	115.0-131.0 SAND: yellowish brown, dense to very dense, slightly damp				
116						

Sheet 5 of 9

CEG

Project DESIGN UNIT A430 Date Drilled 12-9-80 Hole No. 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	SP	115.0-131.0 SAND: continued			RD	
118						
120			C-6		DR	0.4/0.4 recovery
			J-11	50	SS	refusal at 6"
					RD	0.4/0.5 recovery
122						
124						
126						
128						
	SP	129.0-131.0 GRAVELLY SAND: fine to coarse gravel				refusal at 4"
130			J-12	50	SS	0.1/0.3 recovery
					RD	
	ML	131.0-137.0 SANDY SILT: moderate brown; occasional gravelly lenses; stiff				
132						
134						
136						
	SP	137.0-159.0 SAND: light yellowish brown; very dense				
138						
140						

Sheet 6 of 9

Project DESIGN UNIT A430 **Date Drilled** 12-9-80 **Hole No.** CEG 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	SP	137.0-159.0 SAND: continued	C-7		DR	0.5/0.5 recovery
			J-13	48 50	SS	refusal at 12" 0.8/1.0 recovery
142					RD	
144						
146						
148						
150			J-14	50	SS	0.1/0.5 recovery
					RD	refusal at 6"
152						
154						
156						
158						
	ML	159.0-160.5 SANDY SILT: dusky yellowish brown, very stiff				
160			C-8		DR	0.9/0.9 recovery
	SP	160.5-172.0 SAND: light yellowish brown, very dense	J-15	24 50	SS	refusal at 11" 0.2/0.9 recovery
162					RD	
164						

Sheet 7 of 9

Project DESIGN UNIT , A430 **Date Drilled** 12-9-80 **Hole No.** CEG 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	SP	160.5-172.0 SAND: continued 164.0 occasional silt lenses			RD	
166						
168						
170			J-16	28 38 50	SS	1.2/1.3 recovery refusal at 16"
172	CL	172.0-177.0 SANDY CLAY: light brown			RD	
174						
176						
	SP	177.0-188.1 SAND: light yellowish brown, dense to very dense				
178						
180			C-9		DR	12/10/80 0.4/0.5 recovery refusal at 10"
			J-17	18 50	SS	0.6/0.8 recovery
					RD	180' stop drilling 12/9/20 start drilling on 12/10/80
182						
184						
186						
188		188.0 thin claey gravel lens				

Sheet 8 of 9

Project DESIGN UNIT A430 **Date Drilled** 12-10-80 **Hole No.** CEG 37

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188	SM	188.1-196.0 SILTY SAND: moderate olive brown, very dense, moist				
190			J-18	50	SS	refusal at 6" 0.5/0.5 recovery pocket penetrometer 4.5 tsf 2/9/81
192						
194						
196	SP	196.0-201.5 SAND: yellowish gray, fine grained, very dense, wet				
198						
200					DR	1.1/1.5 recovery
				15	SS	
			J-19	38 47		12-10-80
202		END OF BORING 202.0'				water sampled 2/10/81
204						
206						
208						
210						
212						

Sheet 9 **of** 9

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

BORING LOG 37A

Proj: DESIGN UNIT A430
Date Drilled 10-3-83
Ground Elev. 618'
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 A. C. PAVEMENT				
	SM	0.5-9.0 ALLUVIUM SILTY SAND: dark brown; slightly moist; medium dense				hole stands well
2						
		with layers & streaks of clean sand				no caving 0.0-38.0' very minor caving 38.0-60.0'
4						
6						
8						
	SP	9.0-35.5 SAND: medium brown; moist; medium dense				
10						
		layers of silty sand with trace gravel				
12						
14						
16						
18						
20						

Sheet 1 of 3

Project DESIGN UNIT A430 Date Drilled 10-3-83 Hole No. 37-A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	9.0-35.5 SAND: (continued)				
22						
24						
26						
28						
30		layers and streaks of silty sand				
32						
34						
36	SM	35.5-38.0 SILTY SAND: dark brown; moist; medium dense				
38	SP	38.0-44.6 GRAVELLY SAND: very light brown; moist; dense; gravel to 3"				
40						
42						
44						

Sheet 2 of 3

Project DESIGN UNIT A430 Date Drilled 10-3-84 Hole No. 37-A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP	38.0-44.6 GRAVELLY SAND: (continued)				
	ML	44.6-46.4 GRAVELLY SILT: with lenses of sand; silty sand; & gravelly sand; dark brown; stiff; very moist; occasional cobbles to 8"				
46						
	SP	46.4-60.0 GRAVELLY SAND: very light brown; moist; dense				
48		gravel to 5"				
		cobbles to 8"				
50		lenses of gravelly silt & silt; very moist				
52						
54		gravelly sand; gravel to 4"				
						bag sample @ 55'
56						
58						
60		B.H. 60.0 Terminate Hole				
62						
64						
66						
68						

Sheet 3 of 3

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		11.0 silty sand, occasional 1" gravel; one 0.1' layer of very fine sand; moist	J-1	7 13 12	SS	1.0/1.5 recovery
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 24 38	SS	
22						
					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 22 50	SS	
32						
					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 29 50	SS	
42		42.0 silty sand with trace gravel				0.6/1.0 recovery
					RD	1.2/1.5 recovery
44						

Sheet 2 **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
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						

Sheet 4 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
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						

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; fine grained pyrite common				
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
					RD	
164						

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	
174	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				
176						
178						
180		180.0 clayey fine sand	S-5		PB	1.3/1.5 recovery
182		increase in grain size with depth	J-15	50	SS RD	0.1/0.3 recovery
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
		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 perferated casing water sampled 2-25-81 peizometer: from 200' to surface, perforated from 180' to 195', from 120' to 140', and 60' to 100'
202						
204						
206						
208						
210						
212						

Sheet 9 of 9

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	SP (SM) (ML)	34.0-47.0 SAND: (continued) trace gravel to 2"				
46						
48	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		50.0-60.0 cobbles, small boulders to 12"; Fe Oxide staining on cobbles				caving below 50" slowing drill progress
52						
54						
56						
58						
60		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
62						
64						
66						
68						

Sheet 3 of 3

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 2.4/2.5
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	

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						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	PB 2		PB	
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 to very dense; 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						recovery 1.5/1.5 recovery 0.4/0.5
		43.5 gravelly sand	PB 4		PB	
44			J-5	62	SS	

Sheet 2 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
44	SP	37.5-45.5 <u>SAND</u>: (continued)			RD	
46	SP/GW	45.5-79.8 <u>SANDY GRAVEL/GRAVELLY SAND INTERBEDDED</u>: salt and pepper; variable percentages of sands and gravels; very dense; moist; granitic origin; rounded; occasional sand lenses				intense chatter smoother mixed mud recovery 0.4/0.4
48			C-5	150-5"	DR	disturbed chatter
					RD	
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 3.5"	DR RD	all from shoe
70						
72		cobbles				intense chatter
74	(SM) (ML)	thin silty sand, sandy silt lenses				
			C-10	200 5.5"	DR RD	recovery 0.5/0.5 rock in top, 2 rings good; rest from shoe
76						moderate chatter
78						1 good ring, rest from shoe
			C-11	200 4"	DR	recovery 0.3/0.3
80		B.H. 79.8" Terminate Hole				Complete hole 11-10-83
						Tremied grout to surface
82						
84						
86						
88						
90						
92						

Sheet 4 **of** 4

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

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	J-2	11 19 22	SS	recovery 1.5/1.5
16		contains wood fragments			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-	DR 3"	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	S	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			C-3	44 56	DR	recovery 0.8/1.0
	(ML)	29.0-sandy silt lens			RD	
30						
	ML	31.0-35.8 SILT: dark yellowish brown low plastic fines; trace fine sand; hard; moist				
32			PB-3		PB	recovery 2.3/2.5
34		34.0-clayey sand lens				
		35.0 sandy silt	J-4	11 25 44	SS	recovery 0.8/1.5
36	SP	35.8-37.0 SILTY SAND: salt and pepper; fine to medium sand; trace non-plastic fines; very dense; moist			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	79 100-5"	DR	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			C-5	110 100-2"	DR RD	recovery 0.5/0.7 disturbed but representative
54						
56						falling in on bit from above
58						mixed mud
		sand lens	C-6	200	DR RD	recovery 0.2/0.3
60						
62						
64				64	CR	recovery 0.4/0.5
		65.0-sand lens	C-7	195	RD	1/2 disturbed 1/2 in rings ok
66						
68						

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	
70					RD	
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	recovery 0.3/0.3 disturbed but representative
78					RD	
						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.
Earth Sciences Associates
Geo/Resource Consultants

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	SS	recovery 1.0/1.5
				6		
				3		
6					RD	set tub and cased to 5', mixed mud
						down 25 min., joint on kelly hose breaks
8				4	DR	recovery 0.4/1.0
			C-1	6		
					RD	
10						
12					PB	recovery 0/2.5
14						
			PB-1		PB	recovery 1.3/2.5
16						
			J-2	14		recovery 1.2/1.5
				25		
18	SP	17.5-21.0 SAND: salt and pepper and yellowish brown; fine sand; trace silt; occasional gravel; dense; moist		32		
					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' thich 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	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	46 53	DR	recovery 0.5/1.0
					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 23 43	SS	
34						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			C-3	66 100-5.5	DR	recovery 0.8/1.0
					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	
					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						
			C-4	74	DR	recovery 0.3/0.5
50		sand lens		70-5"	RD	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	SS	
				16		recovery 0/1.0 sluff rock stuck in shoe, blows not valid
				34		
					RD	
60						
62						intense chatter
						mixed mud
			C-6	175-5"	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 <u>GRAVELLY SAND/SANDY GRAVEL</u>: (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
		B.H. 79.2' Terminated hole; tremied grout to surface				completed drilling 11/4/83
80						
82						
84						
86						
88						
90						
92						

INFORMATION FOR BORING 11

Project Southern Calif Papid Transit Dist.' Starter Line Project No. 41021J

Boring No. 11 Date Started 12/21/77 Completed 12/21/77

Location Los Angeles County Flood Control District Yard, south of Los Angeles River Channel, west of Lankershim Blvd.

Location Reference: Thomas Brothers, Page 23, F-4

Elevation (ft) 540 Ref. Drainage Map 443, L.A. Dept. Public Works

Drilling Co. Pioneer Driller Elliot Vanderpoppe

Type Drill Rig B-50

Type of Drilling 8" dia. continous-flight, hollow-stem auger to 26 ft, rotary wash, 3¼" dia. tri-cone bit to 32½ ft.

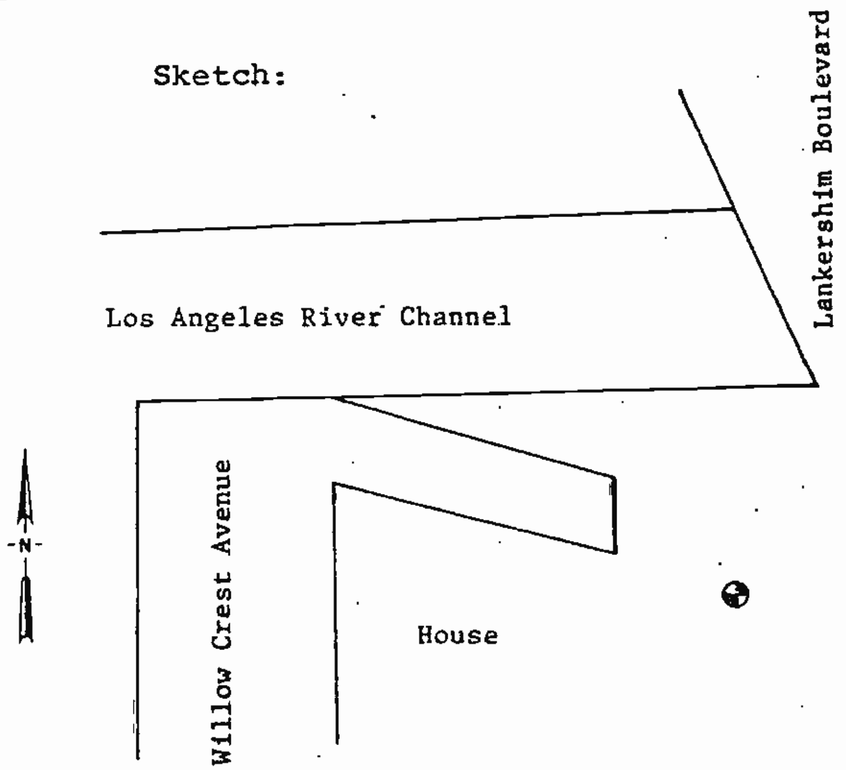
Sampling Technique Modified California and NX cores

Groundwater Depth (ft) 21 ft on 12/21/77

Logged by Steve Young

Problems: None

Sketch:



DATE OF BORING 21 Dec 77 **WATER DEPTH** 21 ft. **DATE MEASURED** 21 Dec 77 **SAMPLES** Modified Calif. & NX Core
TYPE OF DRILL RIG Mobil B-50 **HOLE DIAMETER** 8" / 3¼" **WEIGHT OF HAMMER** 140 lbs. **FALLING** 30 in.

SURFACE ELEVATION: 500

DEPTH, FT.	SAMPLES	BLOWS / 6 IN.	GROUND WATER	DESCRIPTION	ROTARY DRILLING RATE, sec./ft. & PRESSURE, psi	PERCENT SAMPLE RECOVERY	PIEZOMETER SCREENS AND PLUGS
	1	5 5 5		Medium dense, damp, light brown to tan SILTY SAND (SM) More SILT Dark brown with rust or oxide color	*	100	
10	2	3 2 2		Medium stiff, moist, brown with black streaks SILTY CLAY to CLAYEY SILT (CL-ML)		90	PLUG
				Medium dense to dense, moist, black SANDY SILT (ML)			
20	3	31/ 4½"	▽	Very dense, moist-to-saturated, light tan to brown SAND to SILTY SAND (SP-SM) with GRAVEL Driving on cobble or boulder, hard drilling 21 Dec 77		100	
	4	45		Very dense, tan to brown SILTY SAND (SM) Hard drilling, switched to rotary at 26'		100	
30	5			SILTSTONE - sandy; dark gray to black; horizontally bedded with laminations of dark gray clay and medium-to fine-grained light gray sand; abundant micaceous sediments; moist; soft to friable, slightly brittle in some places, few natural fractures, most occured along sand lenses when transfered to core box, most pieces are 3" to 4" long with some as much as 6" long, possibly the Modello or Topanga Formation Core from 26½' to 32½' 26½' to 27' - sandstone, fine-grained; calcareous cement; gray; horizontally bedded, dry with water along fractures; well fractured with one large vertical fracture running into the siltstone; hard, most pieces 2" long or less 27' to 32½' - SILTSTONE as described above	140 400*		

Bottom of Boring at 32½ ft.

* Pressure gauge ..ot working, some pressures have been estimated

Project : SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT, STARTER LINE
Job Number : 41021J

LOG OF BORING 11

Appendix B

Geophysical Exploration

APPENDIX B GEOPHYSICAL EXPLORATION

B.1 DOWNHOLE SURVEY

B.1.1 Summary

Downhole shear wave velocity surveys were performed in Boring CEG-38 for Design Unit A430. Measurements were made at 5-foot intervals from the ground surface to depths of 200 feet. A description of the technique and a summary of the results are attached.

B.1.2 Field Procedure

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

B.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure B-1. The timing line shows a 20 millisecond (MS) break at the end of the record, indicating that each vertical line is 10 MS. The time of the first arrivals of compressional shear energy is indicated by P and S, respectively. Wave arrival records similar to Figure B-1 were analyzed to estimate wave travel times and velocities for Borings CEG-38.

B.1.4 Discussion of Results

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

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

TABLE B-1
DOWNHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
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.

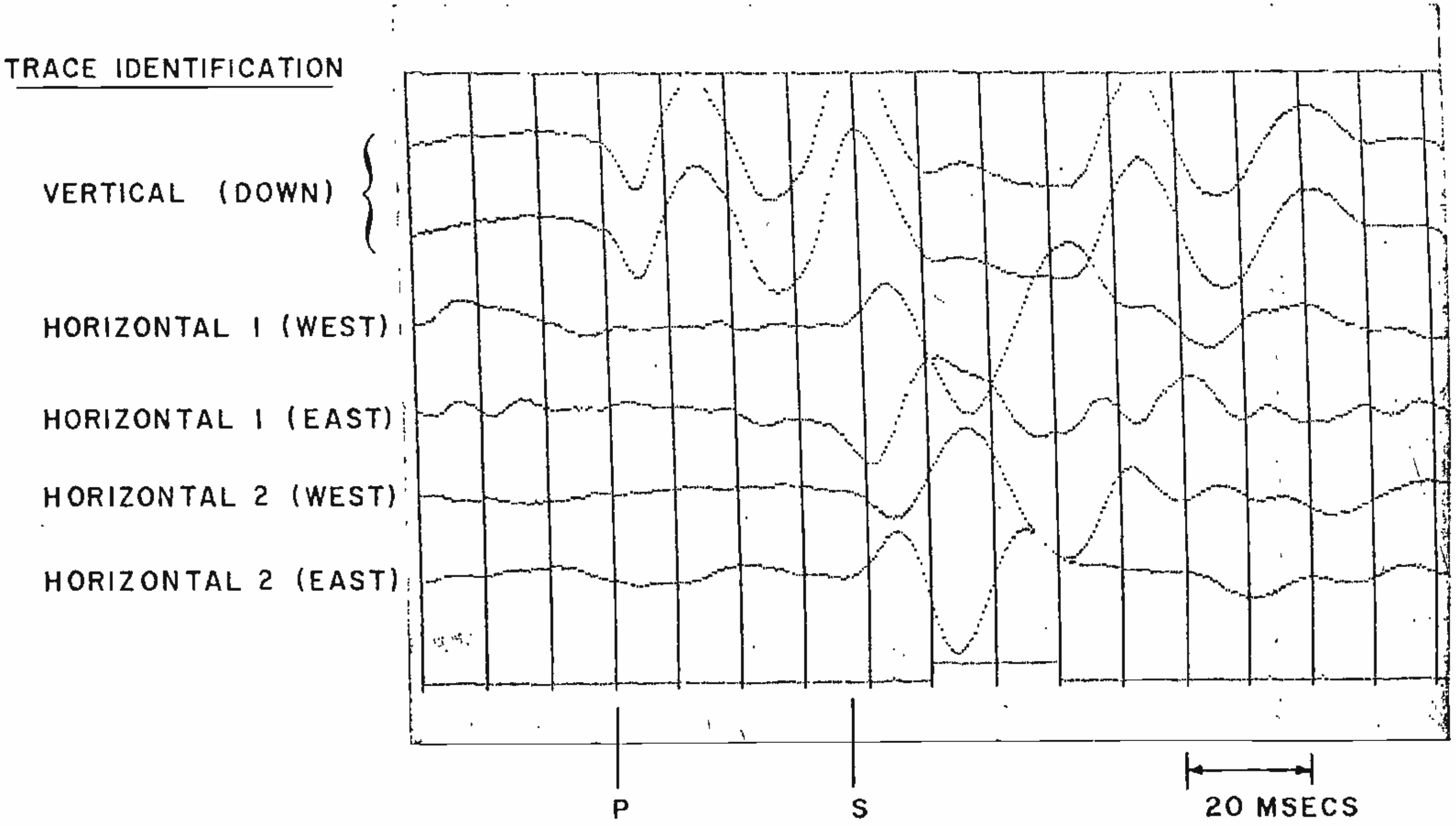
Np = number of points used for straight line fit of compressional wave.

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.

Approved for publication 5/84 by



BOREHOLE: 13

DEPTH: 70 FT

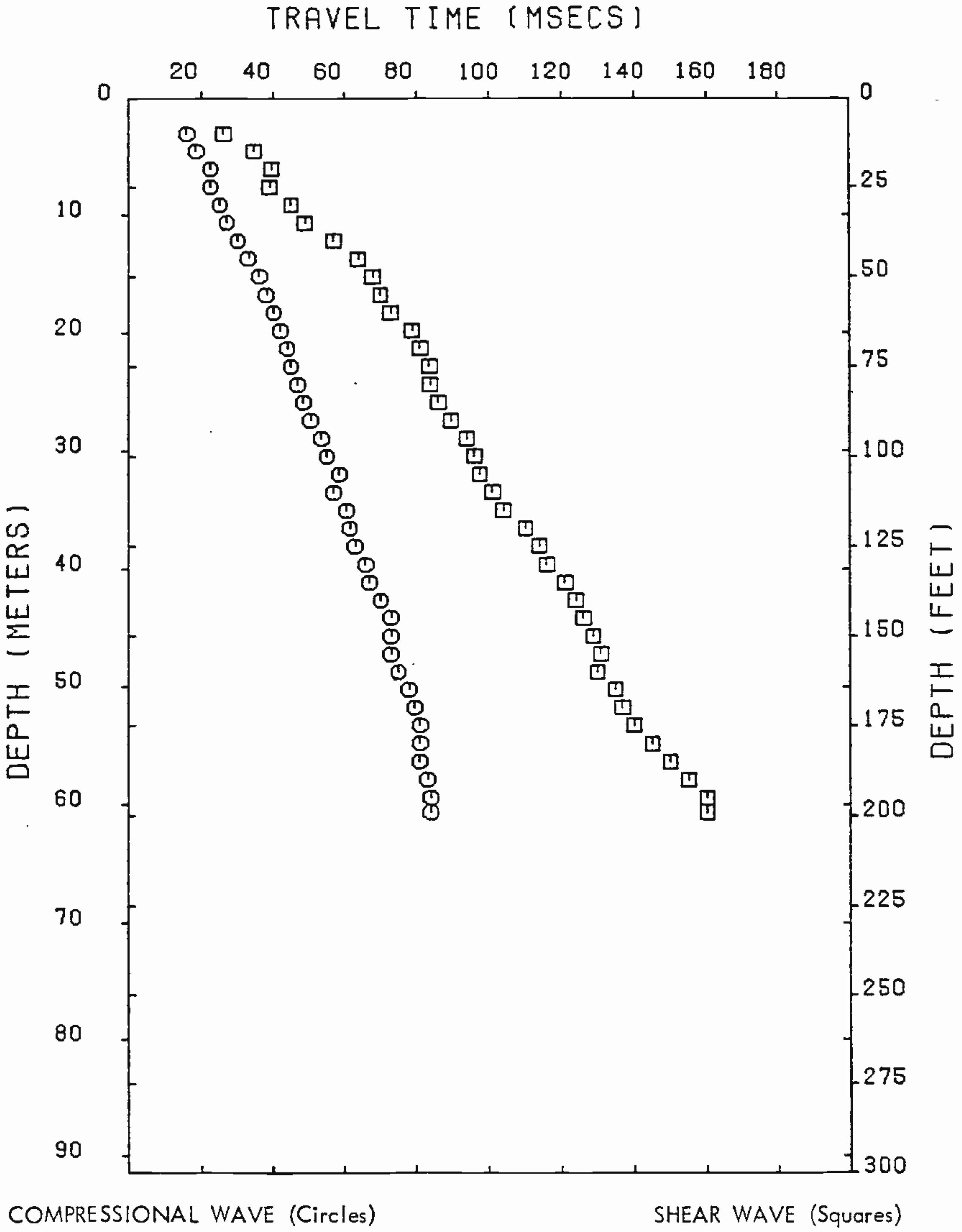
DOWNHOLE SAMPLE RECORD

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Project No.
83-1140

Figure No.
B-1



Approved for publication 5/84 by JAO

DOWNHOLE TRAVEL TIME PROFILE - BORING 38

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Figure No.
B-2

Converse Consultants Geotechnical Engineering and Applied Sciences

Appendix C

Geotechnical Laboratory Testing

APPENDIX C GEOTECHNICAL LABORATORY TESTING

C.1 INTRODUCTION

This appendix presents laboratory geotechnical tests performed on selected soil samples obtained from the borings drilled from Univeral to North Hollywood Station sites Design Unit A430.

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

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

The laboratory test data from the present investigation are presented in Table C-1, while data from the 1981 geotechnical investigation are presented in Table C-2. The geologic units listed in these tables are described in Section 5.0 of the report. Figures C-1 through C-6 summarize strength and modulus data for fine-grained alluvium, granular alluvium, San Pedro sand, and bedrock at this site and other nearby station sites.

C.2 INDEX AND IDENTIFICATION

C.2.1 Visual Classification

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

C.2.2 Grain-Size Distribution

Grain-size distribution tests were performed on representative samples of the geologic units to assist in the soils classification and to correlate test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D-422-63 test method. Combined sieve and hydrometer analyses were performed on selected samples which had a significant percentage of soil particles passing the No. 200 sieve. Results of these analyses are presented in the form of grain-size distribution or gradation curves on Figures C-7 through C-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.

C.2.3 Atterberg Limits

Atterberg Limit Tests were performed on selected soil samples to evaluate their plasticity and to aid in their classification. The testing procedure was in accordance with ASTM D-423-66 and D-424-59 test methods. Test results are presented on Figure C-13 and Tables C-1 and C-2.

C.2.4 Moisture Content

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

C.2.5 Unit Weight

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

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

C.3 ENGINEERING PROPERTIES: STATIC

C.3.1 Unconfined Compression

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

C.3.2 Triaxial Compression

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

C.3.2.1 Consolidated Undrained (CU) Tests

- The undisturbed test specimen was trimmed to a length to diameter ratio of approximately 2.0.
- The specimen was then covered with a rubber membrane and placed in the triaxial cell.

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

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

Results of the triaxial compression tests are presented on Figures C-14 through C-27.

C.3.3 Direct Shear

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

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

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

C.3.4 Swell

No swell tests were perfomred in this design unit.

C.3.5 Consolidation

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

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

Results of consolidation tests on the undisturbed samples are presented on Figure C-28.

C.3.6 Permeability

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

C.4 ENGINEERING PROPERTIES: DYNAMIC

C.4.1 Resonant Column

The resonant column test provides data by which the shear modulus and damping of soil specimens can be determined for 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.

C.4.1.1 Sample Preparation and Handling

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

C.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures, ($\sigma 3c$), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressures and progressing to the higher confining pressures. At each confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures C-29 through C-34.

C.4.1.3 Apparatus

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

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

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

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

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

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

C.4.1.4 Data Reduction

Data obtained from the resonant column tests were reduced in accordance with the ASTM "suggested Methods of Test for Shear Modulus and Damping of Soils by the Resonant Column" using a proprietary computer program developed by Converse Consultants.

C.4.2 Cyclic Triaxial Compression

Evolved from the static triaxial procedure, 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.

C.4.2.1 Sample Preparation and Handling

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

C.4.2.2 Test Conditions and Parameters

Test conditions and parameters may vary in the 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 did not occur. A consolidation ratio ($K_c = \sigma_{1c}/\sigma_{3c}$) of 1.0 was used for this program.

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

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

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

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

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

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

C.4.2.4 Data Reduction

The following methods and definitions were 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 strain: 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 C-35.

C.4.3 Dynamic Triaxial Compression

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

C.4.3.1 Sample Preparation and Handling

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

C.4.3.2 Test Conditions and Parameters

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

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

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

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

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

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

C.4.3.3 Apparatus

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

C.4.3.4 Data Reduction

The following methods and definitions were employed in the reduction of test data from the dynamic triaxial tests.

- Axial stress: Given in terms of axial load and the unconsolidated specimen crosssectional area.

- Axial strain: Given in terms of the consolidated specimen length.

- Dynamic axial strain: The peak-to-peak axial strain for any given loading cycle.

- Shear modulus and shear strain conversion: Axial stress, axial strain and Young's modulus, E, were converted to equivalent shear stress, shear strain and shear modulus, G, using a Poisson's ratio of 0.5 (undrained, zero volume change condition) for tests on saturated samples, and an assumed Poisson's ratio of 0.40 for tests on saturated specimens tested at their in situ moisture contents. Shear strain values are the strains on a plane located at 45° to the principal stress plane, which has been shown to be the plane of maximum shear strain during triaxial loading.

- Modulus: Shear modulus values are defined as the equivalent linear modulus corresponding to the straight line connecting the end points of the hysteresis loop of each loading cycle.

- Shear strain: Shear strain values given are the maximum shear strains between the end points of the hysteresis loop for a given cycle. The maximum shear strain is calculated according to the equations of solid body mechanics as 1.5 x the maximum axial strain.

Results of the dynamic triaxial tests are presented in Figures C-36 and C-37.

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		SPECIFIC GRAVITY	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_1	104	6						32	0.42						
	PB-1	14.5	Sand/Silty Sand	A_1	99	20										X			X
	C-2	19.5	Sandy Silt/Silty Sand	A_1	103	10						30	0.17			X			
	PB-2	24.5	Silty Sand	A_1	108	16													
	PB-3	34.2	Sandy Silt	A_1	107	19										X			X
	C-4	39.2	Sand	A_1	114	12						30	1.45						
	PB-4	43.5	Gravelly Sand	A_1	117	16										X			X
	C-8	62.9	Gravelly Sand	A_1	130	7										X			
38-2	C-1	9.0	Sand	A_1	97	3						27	0.56						
	PB-1	14.5	Sandy Silt/Silty Sand	A_1	97	25										X			X
	PB-1	14.5	Sandy Silt/Silty Sand	A_1	99	15													
	PB-2	24.5	Sand/Silty Sand	A_1	105	14										X			X
	C-3	29.0	Silty Sand/Sandy Silt	A_1	95	14						27.5	1.10			X			X
	PB-3	34.5	Clayey Sand	A_1	97	27					1.85								
	C-4	38.8	Gravelly Sand	A_1	124	8						34	1.00						
	PB-4	44.5	Sand/Silty Sand	A_1	109	20										X			X
	C-7	65.0	Sand	A_1												X			

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		SPECIFIC GRAVITY	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						
38-3	C-1	9.0	Silty Sand	A_1	92	20						30	0.10			X			
	PB-1	17.0	Silty Sand	A_1	103	15										X			X
	PB-3	33.5	Sandy Clay/Clayey Sand	A_1	106	22					3.03								
	C-3	39.0	Sand	A_1	115	13													
38-4	PB-1	17.0	Silty Sand	A_1	105	17										X			X
	PB-1	17.0	Silty Sand	A_1	100	20													
	C-2	29.0	Sand	A_1	112	15						30	1.10			X			
	PB-3	34.5	Clayey Silt/Clayey Sand	A_1	104	22					2.05								
	PB-4	44.5	Gravelley Sand	A_1	120	10										X			X
38-5	C-1	9.0	Sand/Silty Sand	A_1	105	17										X			
	C-2	19.5	Sand	A_1	117	9													
	C-4	31.0	Silty Sand	A_1	112	12						30	0.55						
	PB-2	35.9	Silty Sand	A_1	110	19										X			X
38-6	C-2	28.2	Sand	A_1	111	12										X			
	PB-3	39.5	Silty Sand	A_1	104	20													
	PB-3	39.5	Silty Sand	A_1	105	21										X			X

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		SPECIFIC GRAVITY	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				ϕ, deg	c, ksf						
38-7	C-1	9.5	Sand	A_1	97	12						35	0.25						
	PB-1	14.5	Silty Sand/Sand	A_1	103	10										X			X
	C-3	29.5	Gravelly Sand	A_1	118	11						30	0.90			X			
	PB-4	43.6	Silty Sand/Clayey Sand	A_1	113	17													
	PB-4	43.6	Silty Sand/Clayey Sand	A_1	111	16										X			X

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

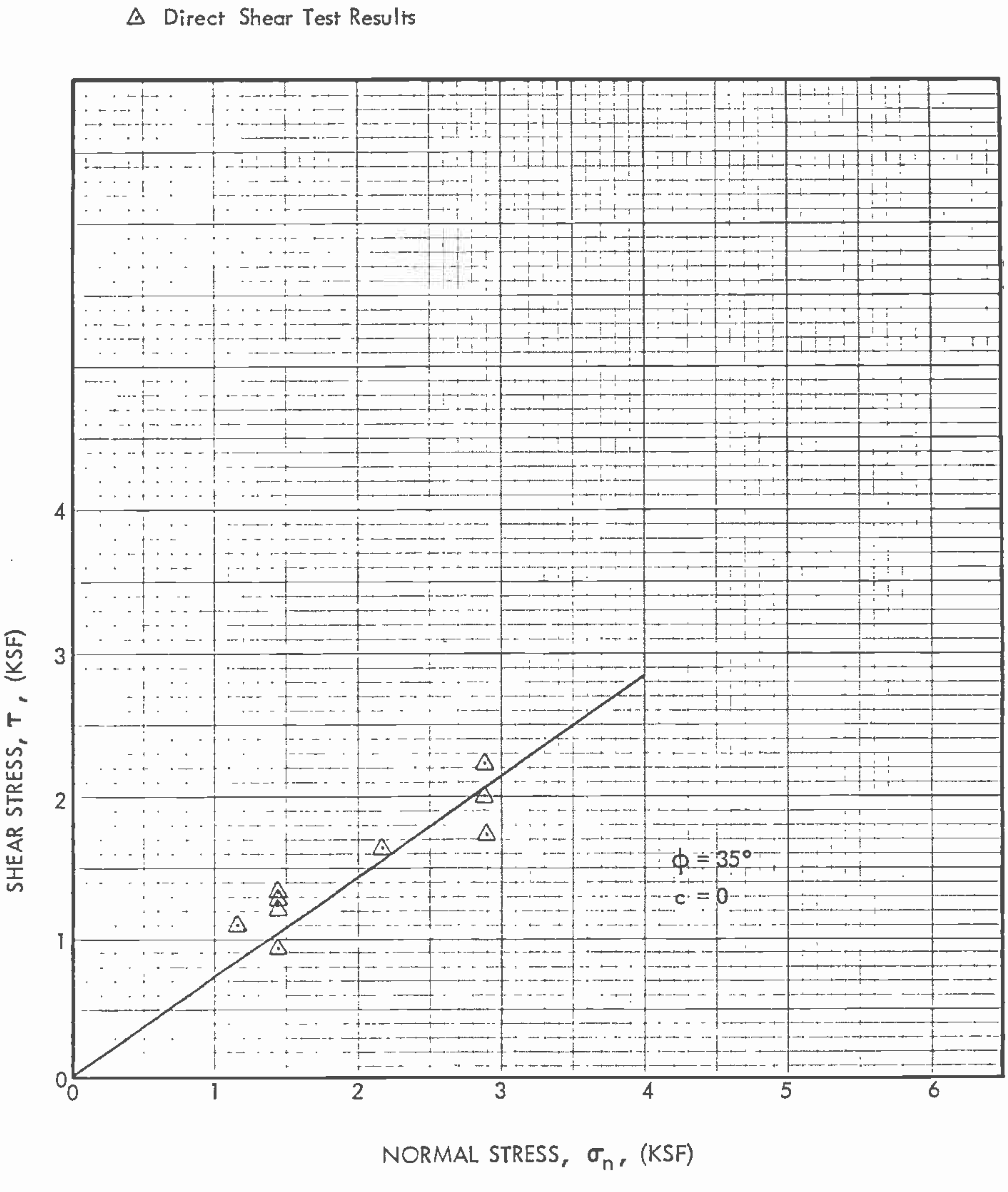
CEG BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		PARTICLE SIZE CUMULATIVE % PASSING SIEVE No.			UNCONFINED COMPRESSION STRENGTH (psi)	Kv, COEFFICIENT OF PERMEABILITY (cm/sec)	COEFFICIENT OF CONSOLIDATION C_c (in/in per Log Cycle)	SPECIFIC GRAVITY	POROSITY (n)	UNDRAINED QUICK DIRECT SHEAR		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	CYCLIC TRIAXIAL (Liquefaction)	DYNAMIC TRIAXIAL (Stress/Strain)	RESONANT COLUMN	TRIAXIAL COMPRESSION
							LL	PI	4	40	200						ø, deg	c, ksf					
37	C1	20	Fine to medium sand	A_1	106	7								.027			35.5	0.19					
	C3	60	Fine to medium sand	A_3	112	19															X		
	C4	80	Fine to medium sand	A_3									3.2E-4			38.2							
	C5	100	Fine to medium sand	A_3	104	11											37.5	0.16					
	C5	100	Fine to medium sand	A_3	108	12											40	0.34					
	J10	110	Fine sandy silt				32	7	100	98	86						37.5	0.22					
	C6	120	Fine to medium sand	A_3	111	14											34	0.28					
	C6	120	Fine to medium sand	A_3	-	-											40.5	0.44					
	J11	120	Silty fine sand						99	63	14												
	C7	140	Silty fine sand	A_3	107	16											39.5	0.25					
	C7	140	Silty fine sand	A_3	-	-											37	0.65					
	C8	160	Silty fine sand	A_3	101	28											26.5	0.65					
	C8	160	Silty fine sand	A_3	101	28											34	0.48					
	C9	180	Silty fine sand	A_3	114	17											33	0.40					

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

CEG BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		PARTICLE SIZE CUMULATIVE % PASSING SIEVE No.			UNCONFINED COMPRESSION STRENGTH (psi)	Kv, COEFFICIENT OF PERMEABILITY (cm/sec)	COEFFICIENT OF CONSOLIDATION C_c (in/in per Log Cycle)	SPECIFIC GRAVITY	POROSITY (n)	UNDRAINED QUICK DIRECT SHEAR		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	CYCLIC TRIAXIAL (Liquefaction)	DYNAMIC TRIAXIAL (Stress/Strain)	RESONANT COLUMN	TRIAXIAL COMPRESSION
							LL	PI	4	40	200						ø, deg	c, ksf					
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	100	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	A_4	109	19						9.3											
	S3	142	Sandy silt	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											

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

CEG BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		PARTICLE SIZE CUMULATIVE % PASSING SIEVE No.			UNCONFINED COMPRESSION STRENGTH (psi)	Kv, COEFFICIENT OF PERMEABILITY (cm/sec)	COEFFICIENT OF CONSOLIDATION C_c (in/in per Log Cycle)	SPECIFIC GRAVITY	POROSITY (n)	UNDRAINED QUICK DIRECT SHEAR		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	CYCLIC TRIAXIAL (Liquefaction)	DYNAMIC TRIAXIAL (Stress/Strain)	RESONANT COLUMN	TRIAXIAL COMPRESSION
							LL	PI	4	40	200						ø, deg	c, ksf					
38	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	



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

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Project No.
83-1140

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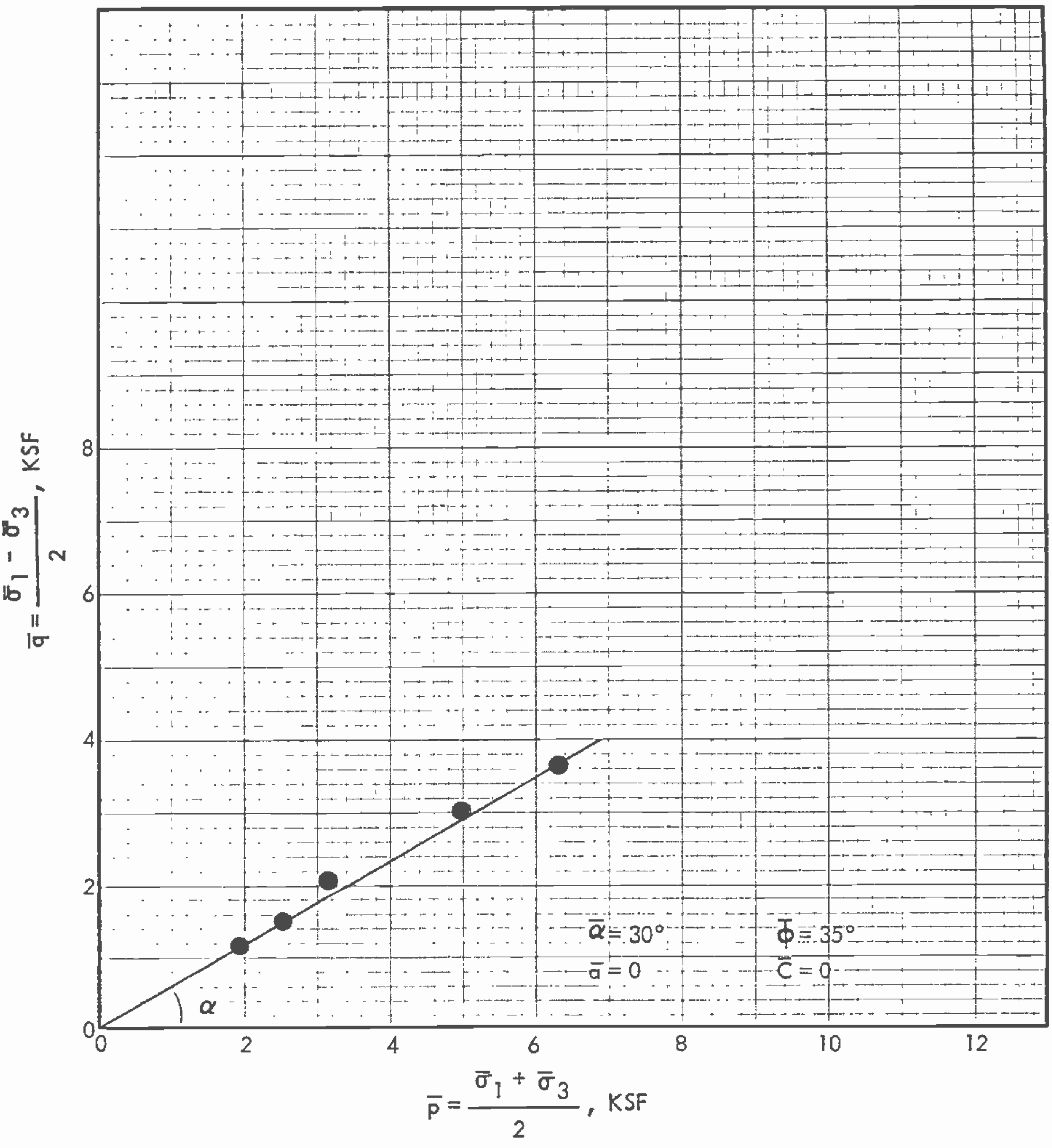
Figure No.
C-1

Approved for publication 5/84 by FAB

● Triaxial Test Results

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



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

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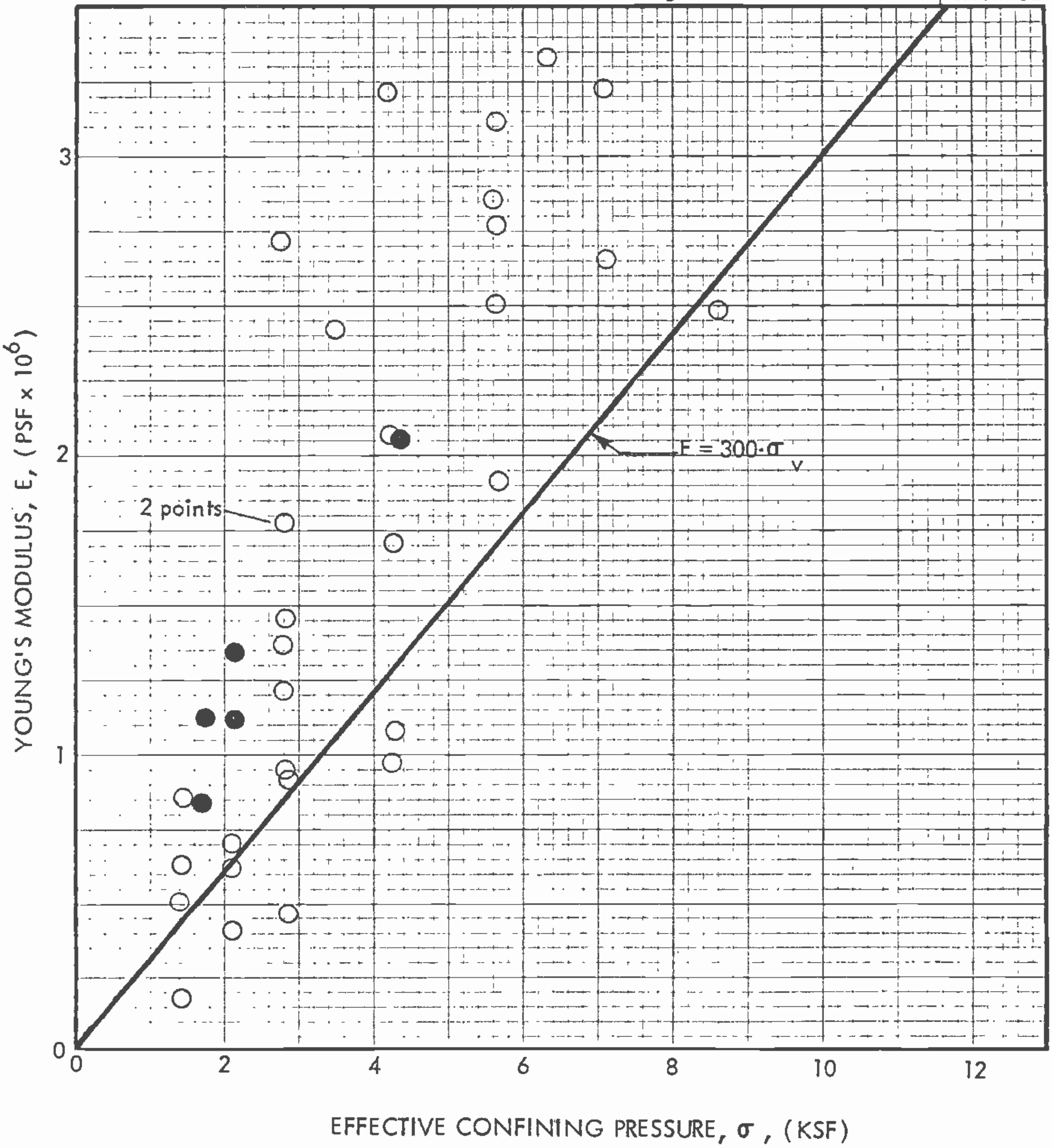
Figure No.
C-2

Approved for publication – 5/i/44 – by JAD

● 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



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SUMMARY OF MODULUS DATA - ALLUVIUM (DEPTH≤15')

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Figure No.
C-3

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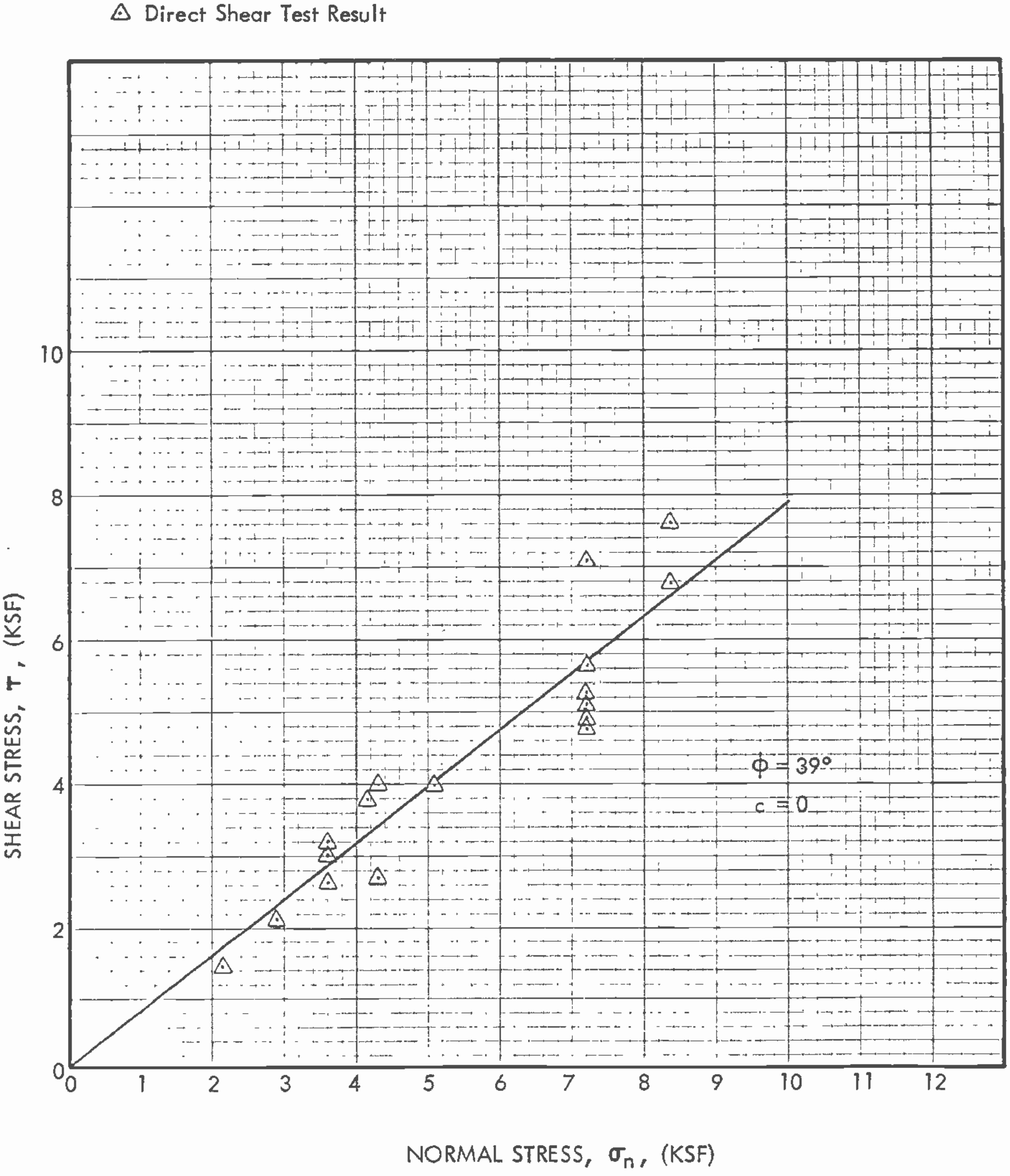

SUMMARY OF DIRECT SHEAR TEST RESULTS - ALLUVIUM (DEPTH >15 FT.)

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83-1140

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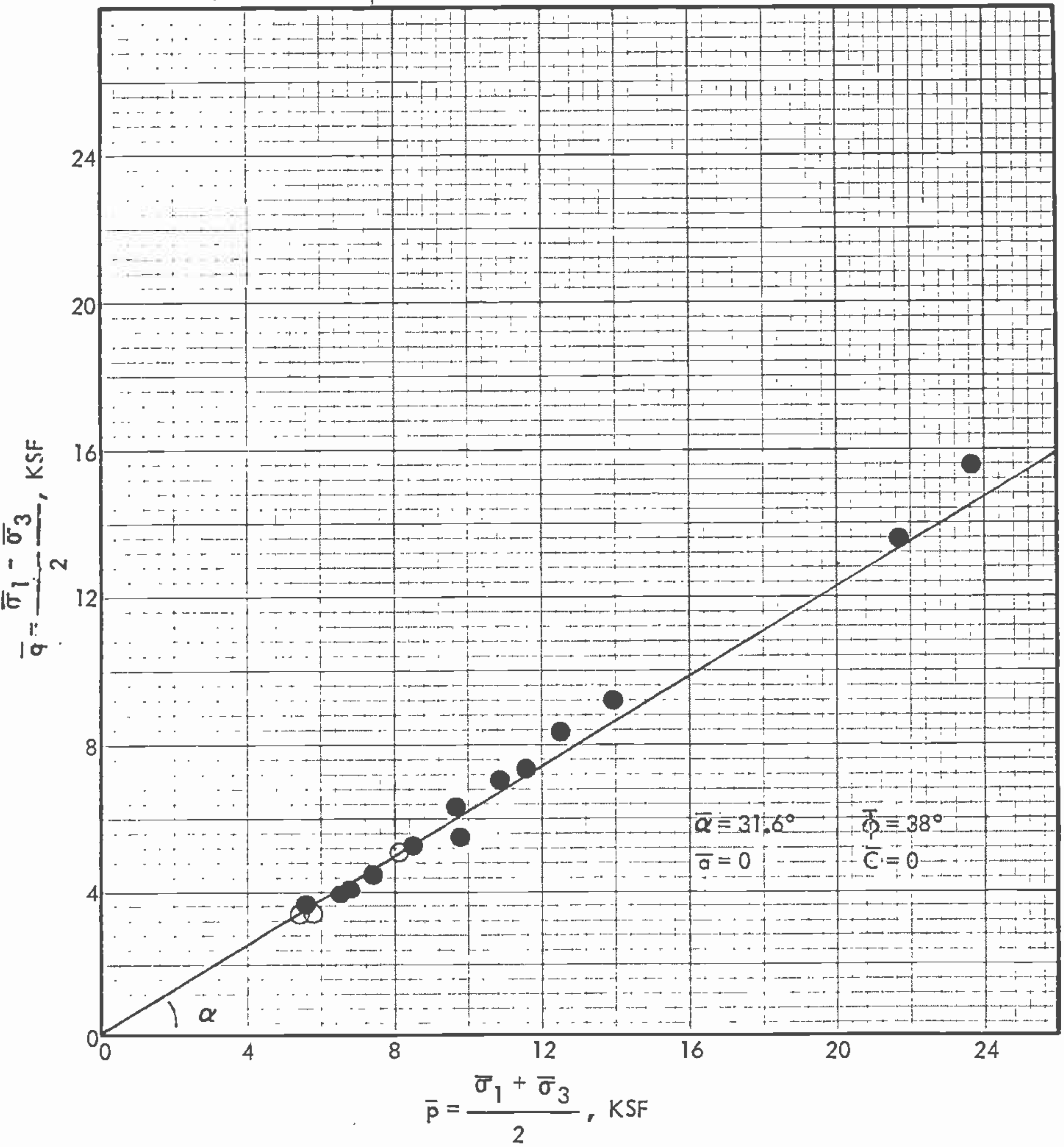
Figure No.
C-4

Approved for publication 5/8/4 by

● Triaxial Test results

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



Approved for publication 5/84 by FAD

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

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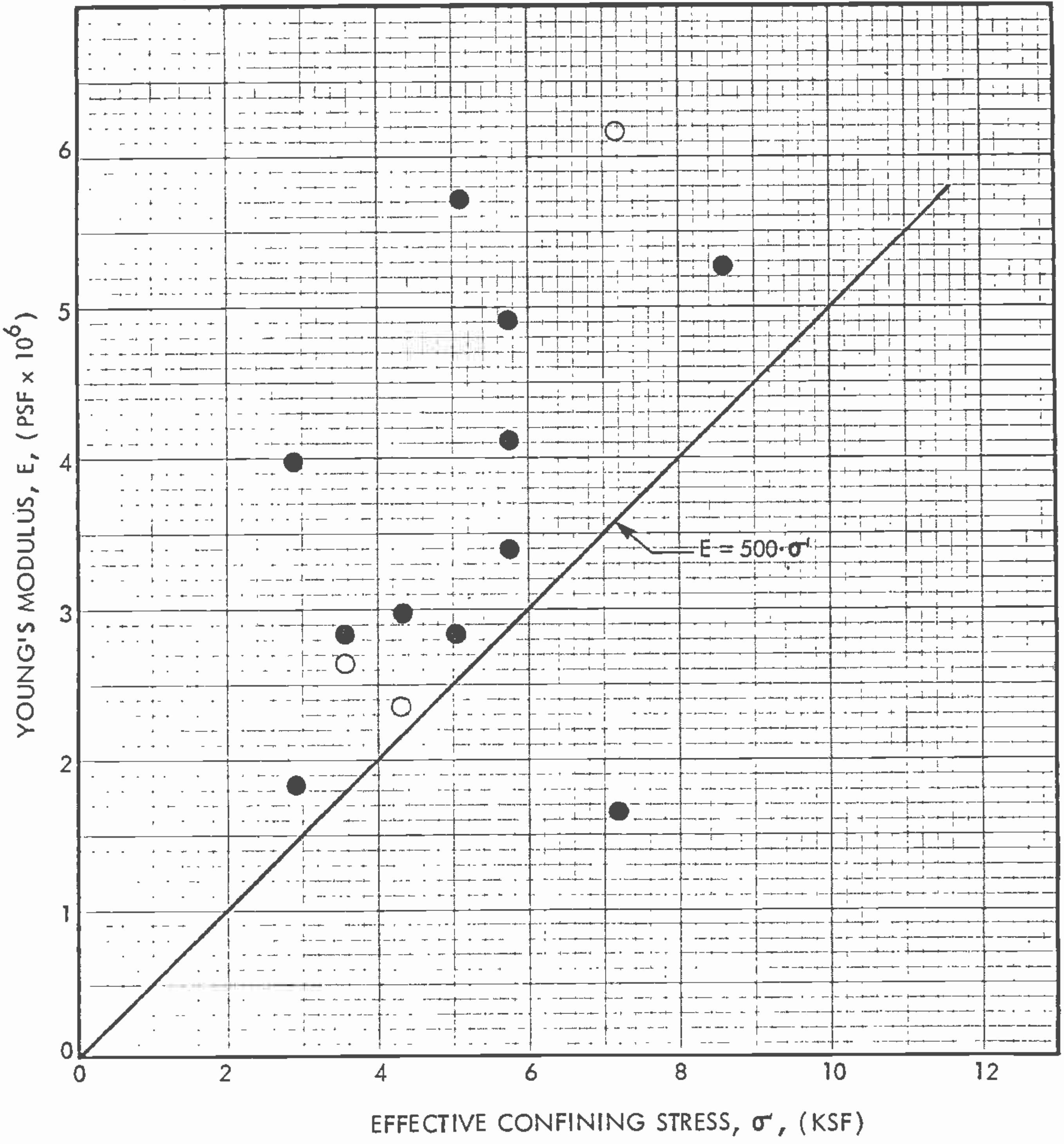
Figure No.
C-5

● Triaxial Test results

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



SUMMARY OF MODULUS DATA - ALLUVIUM (DEPTH >15')

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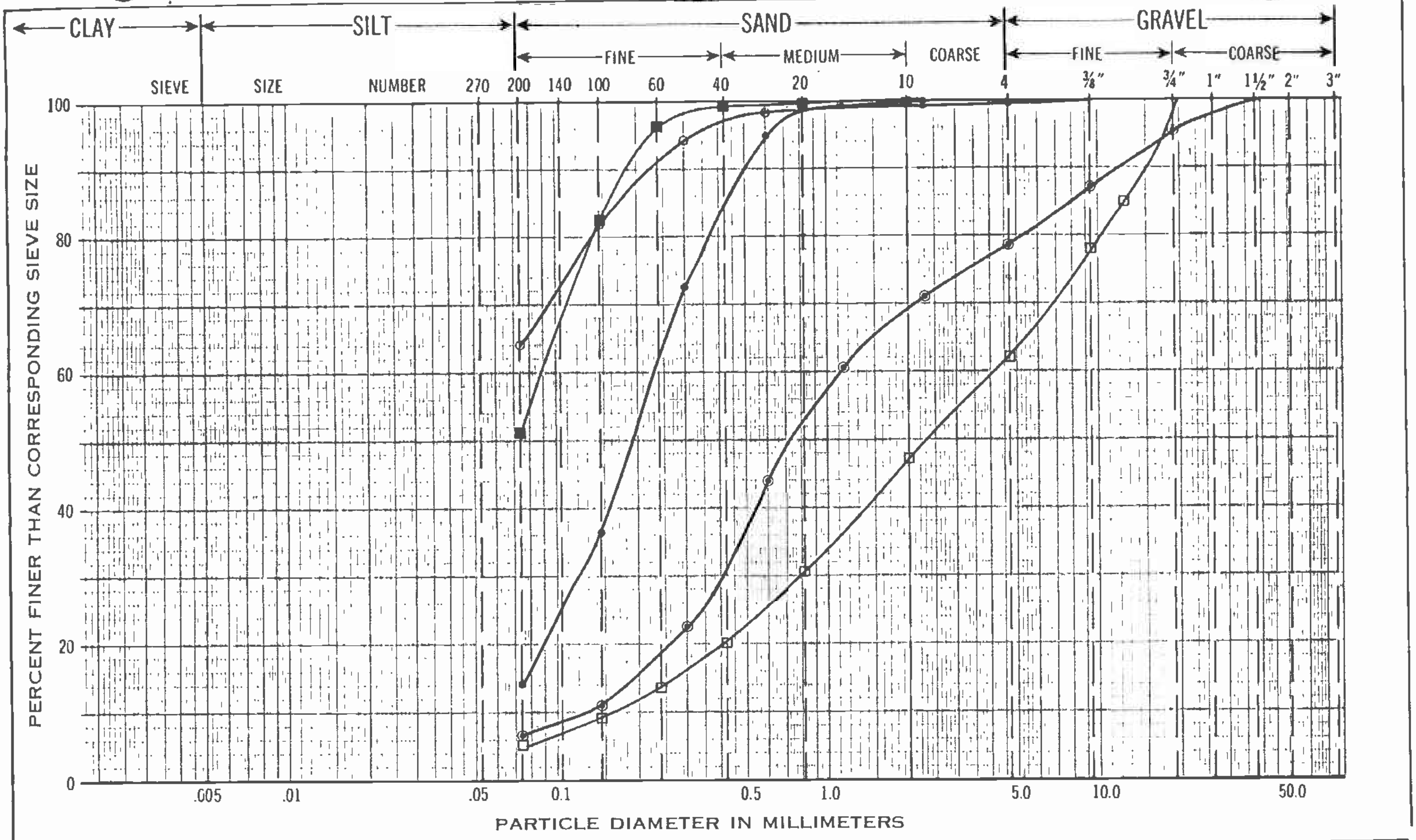
Figure No.
C-6

Approved for publication 5/84 by JAO

FORM NO. D-3-1 Approved for publication 5/84 by



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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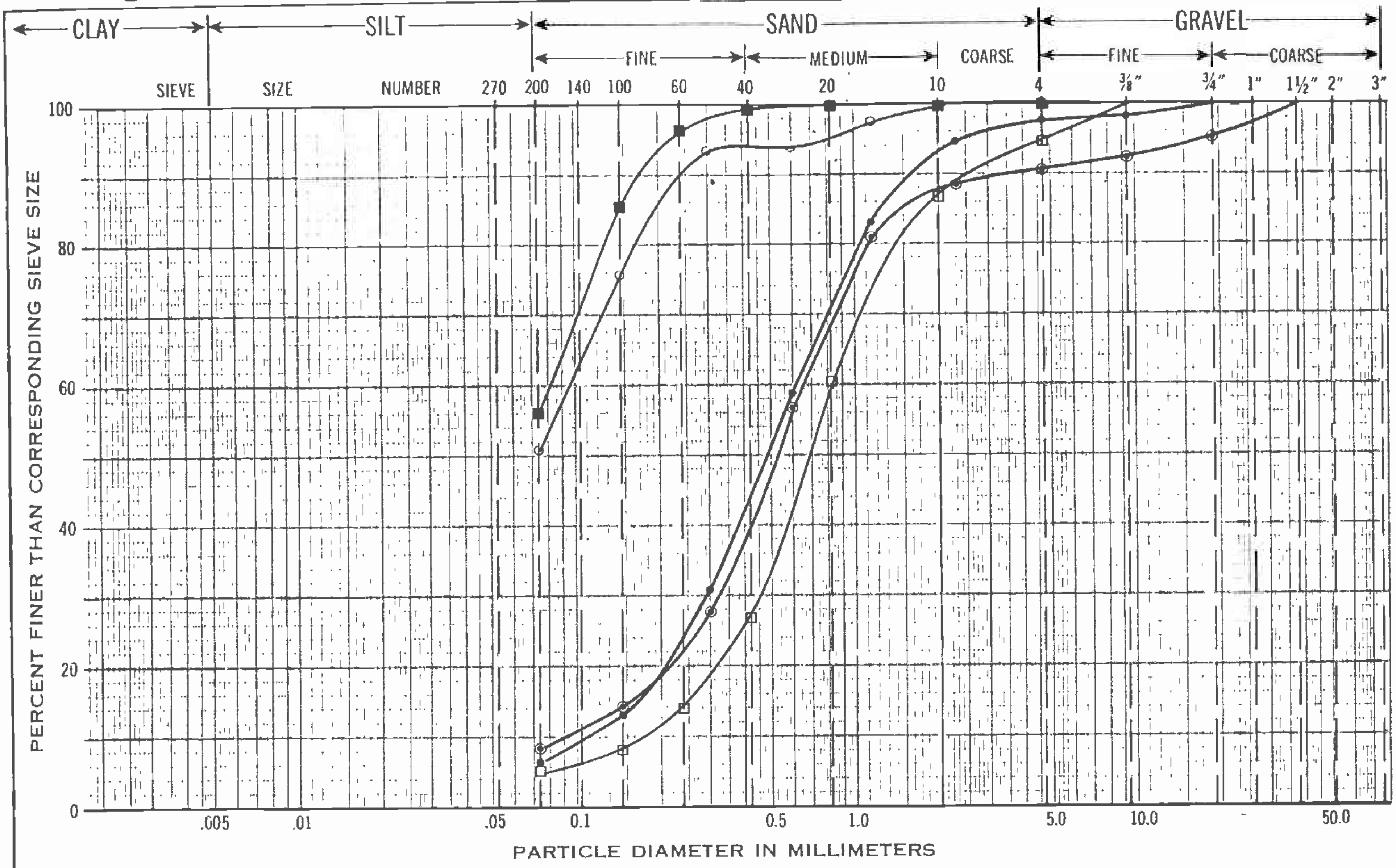
Figure No.
C-7

FORM NO. D-3-1 Approved for publication 5/84 by



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.
C-8

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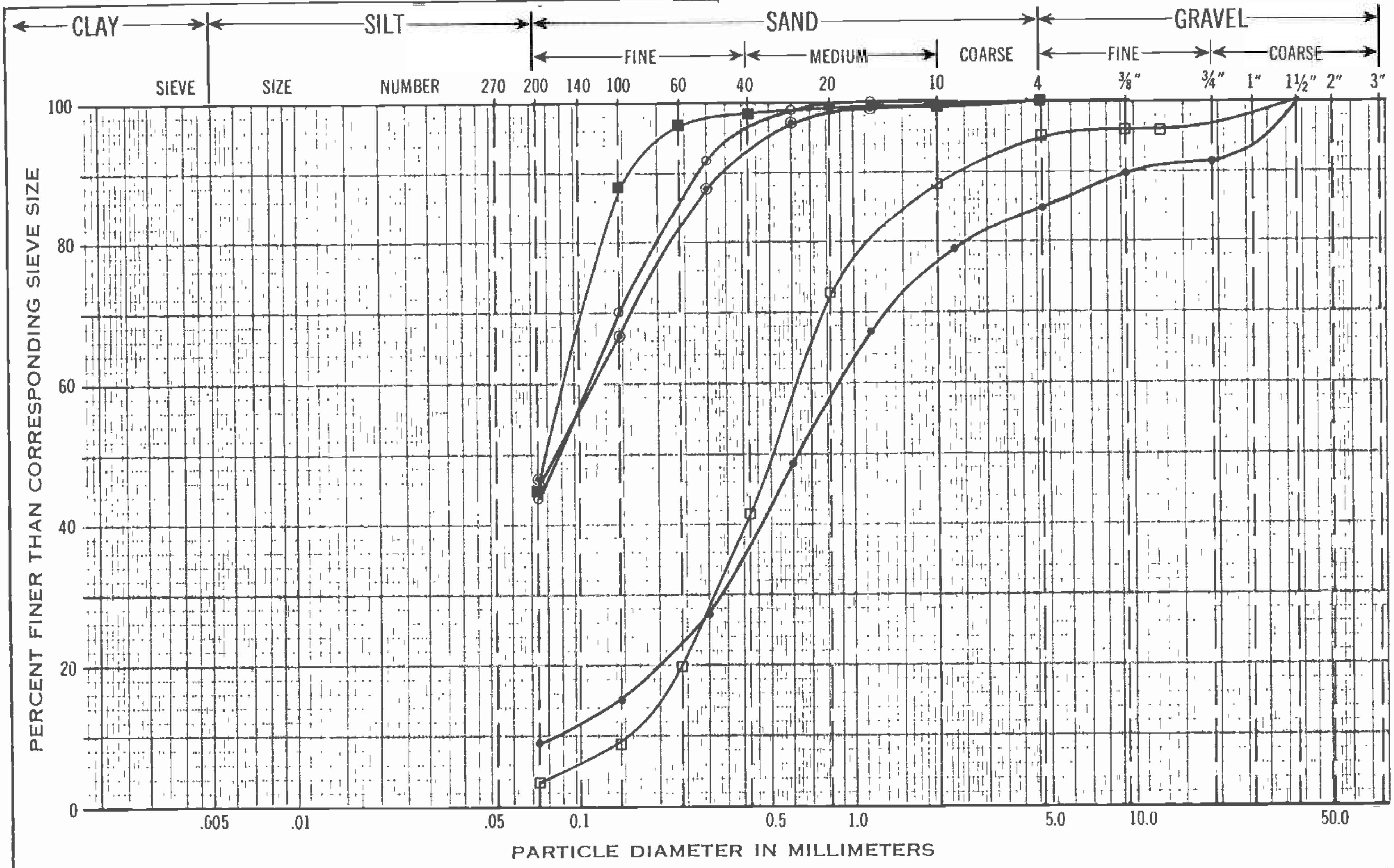
FORM NO. D[illegible]/81 Approved for publication 5/84 by JAD



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

GRAIN-SIZE DISTRIBUTION CHART

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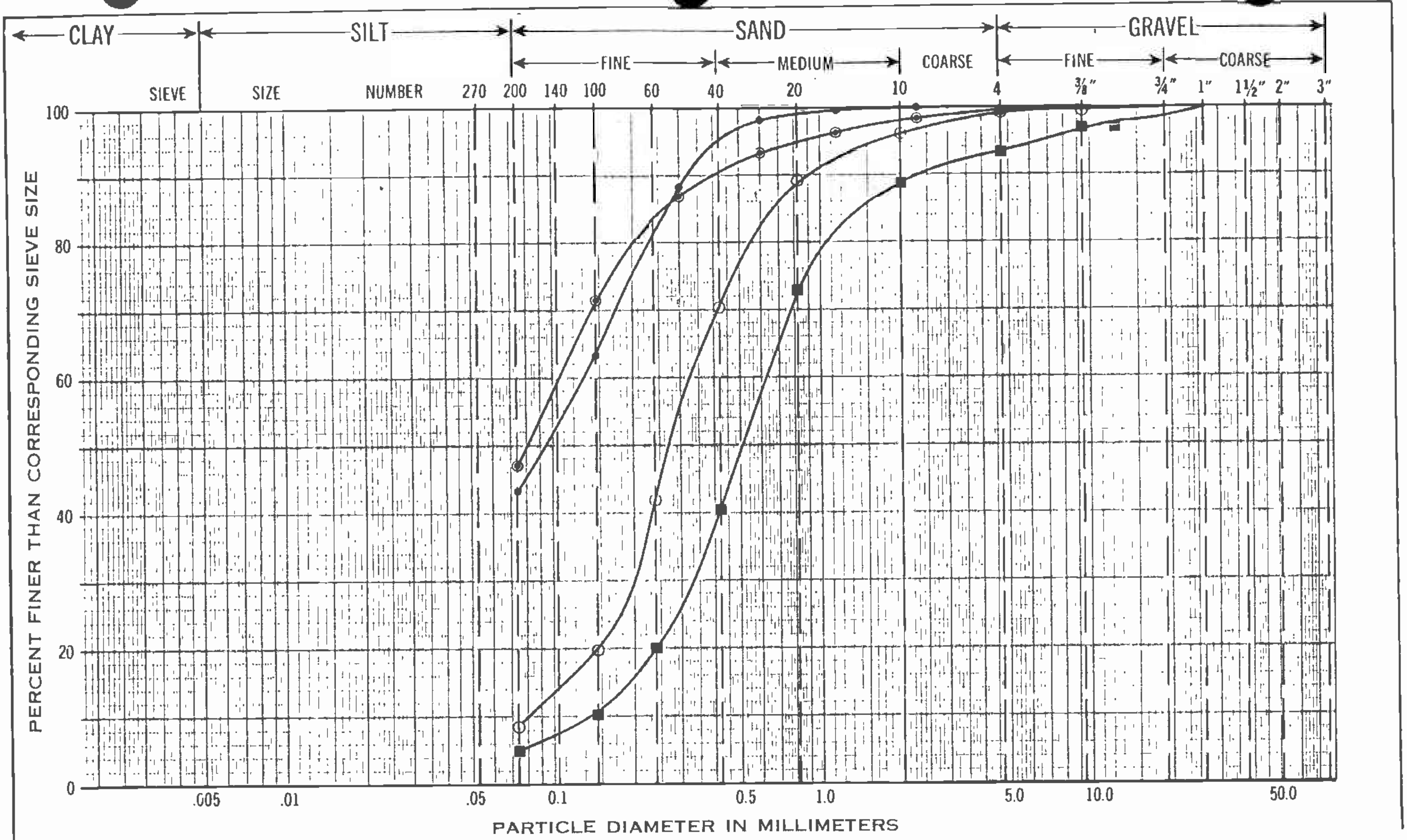
Figure No.
C-9

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



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

GRAIN-SIZE DISTRIBUTION CHART

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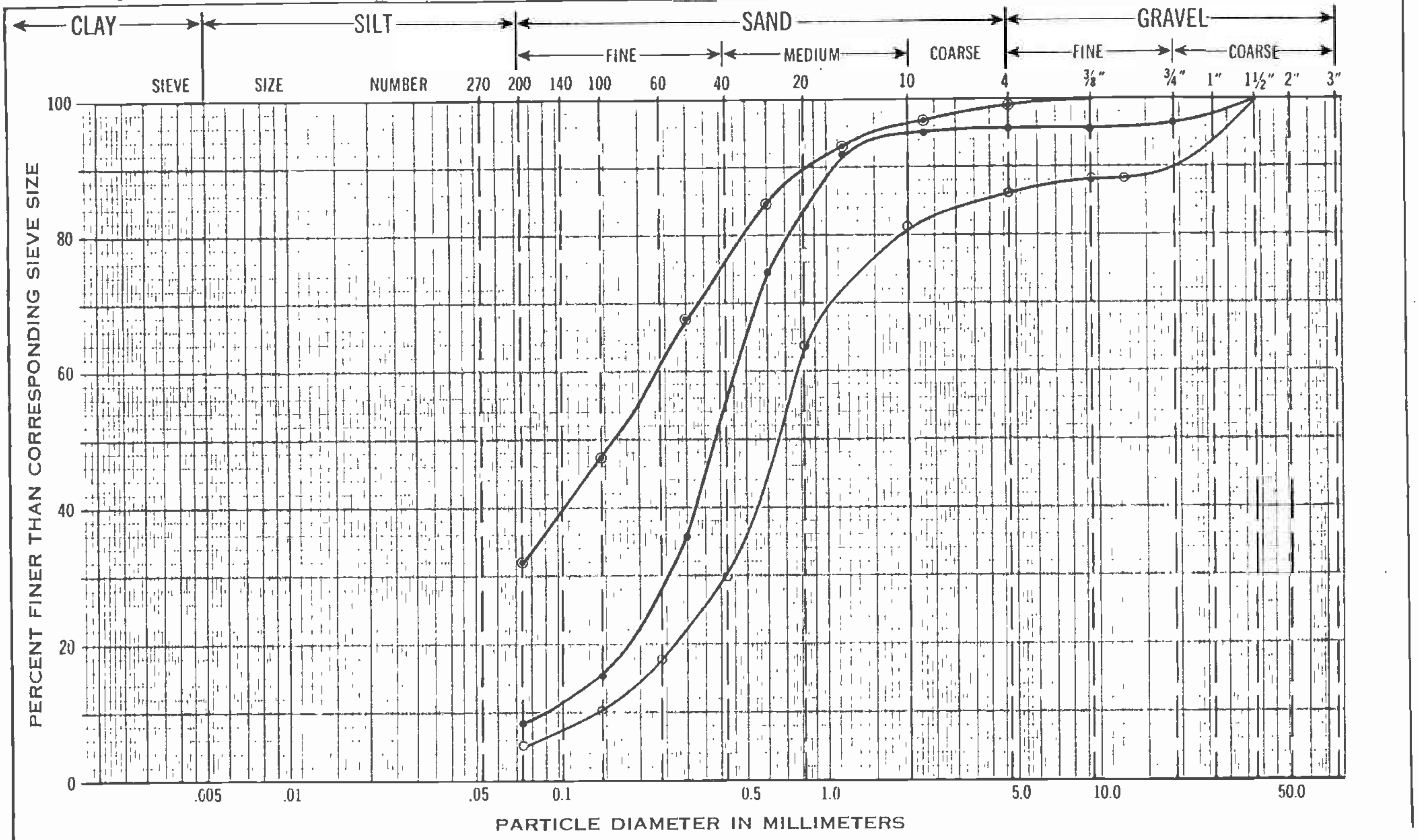
Figure No.
C-10

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



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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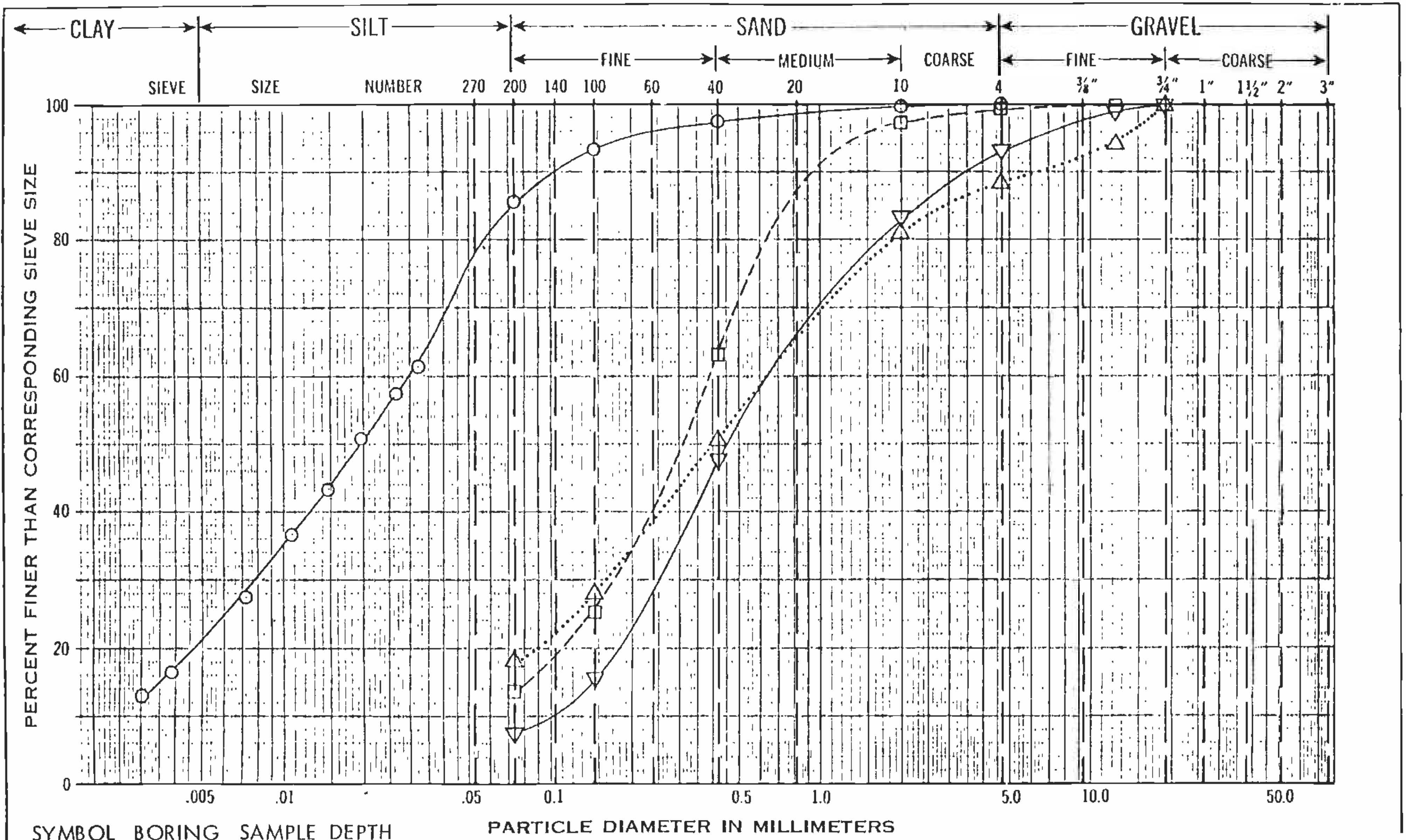
Figure No.
C-11

FORM NO. D22 Approved for publication 5/84 by

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



SYMBOL	BORING	SAMPLE	DEPTH
—○—	37	J-10	110'
---□---	37	J-11	120'
.....△.....	38	J-1	11'
—▽—	38	J-4	42'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-12

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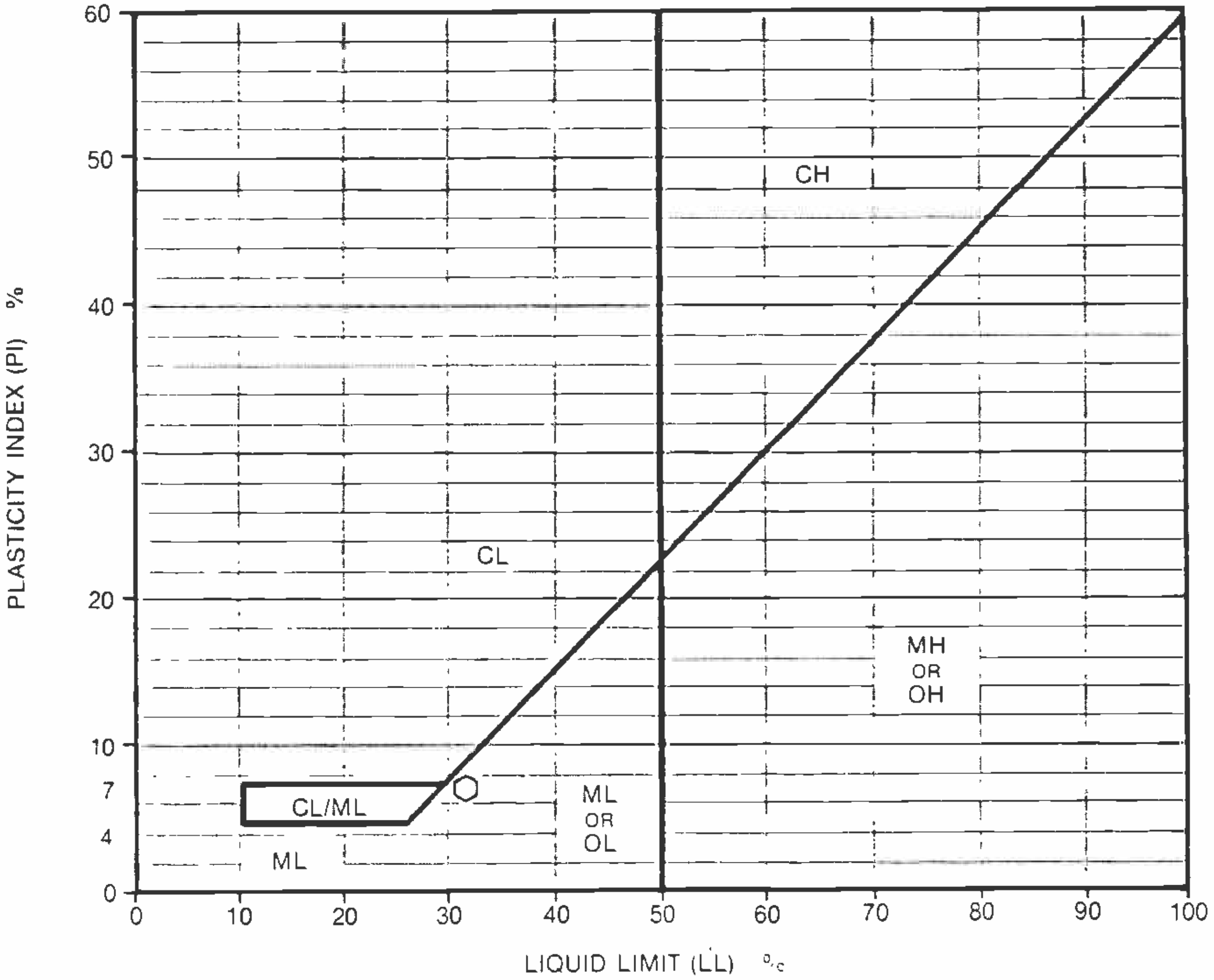

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
⬡	BH 37 110' (ML)	32	25	7	86

PLASTICITY CHART

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Figure No.
C-13

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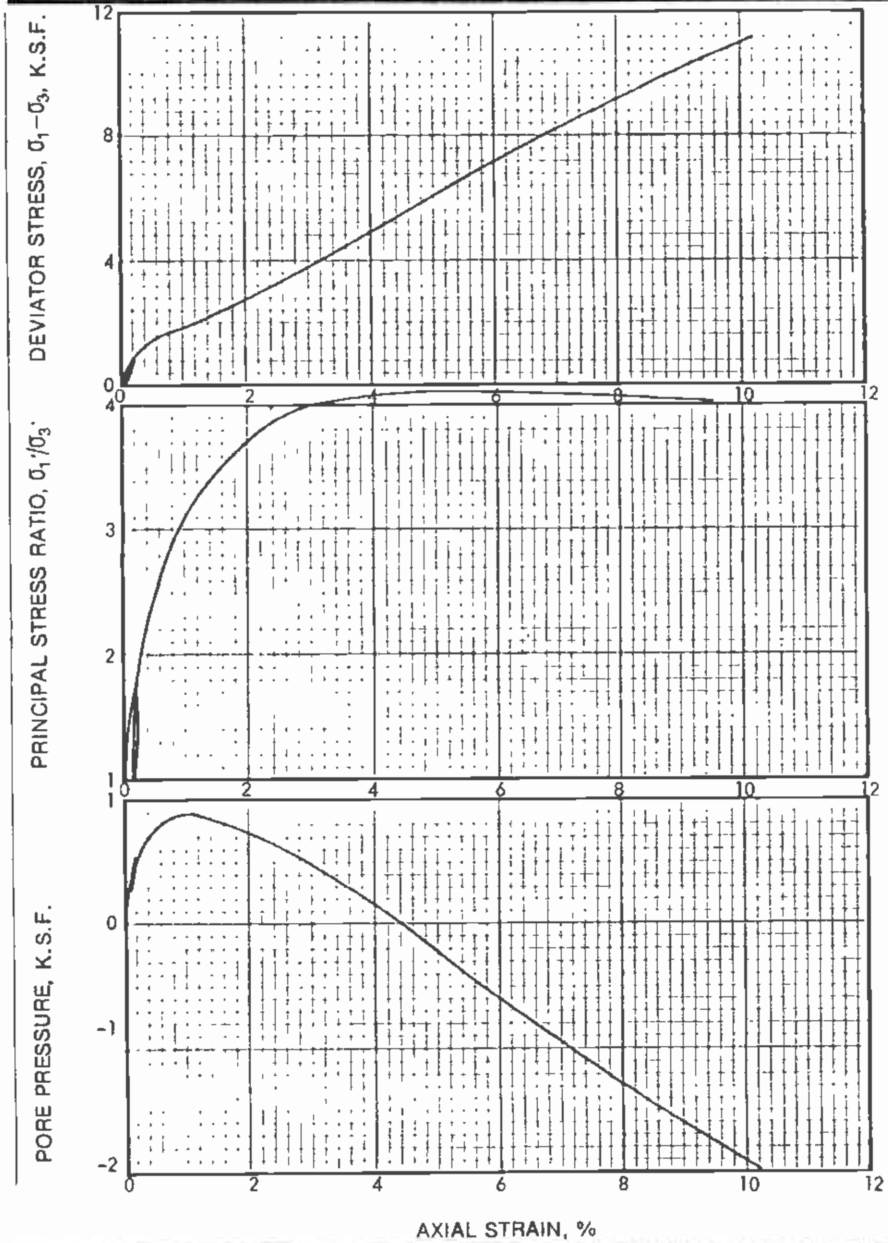

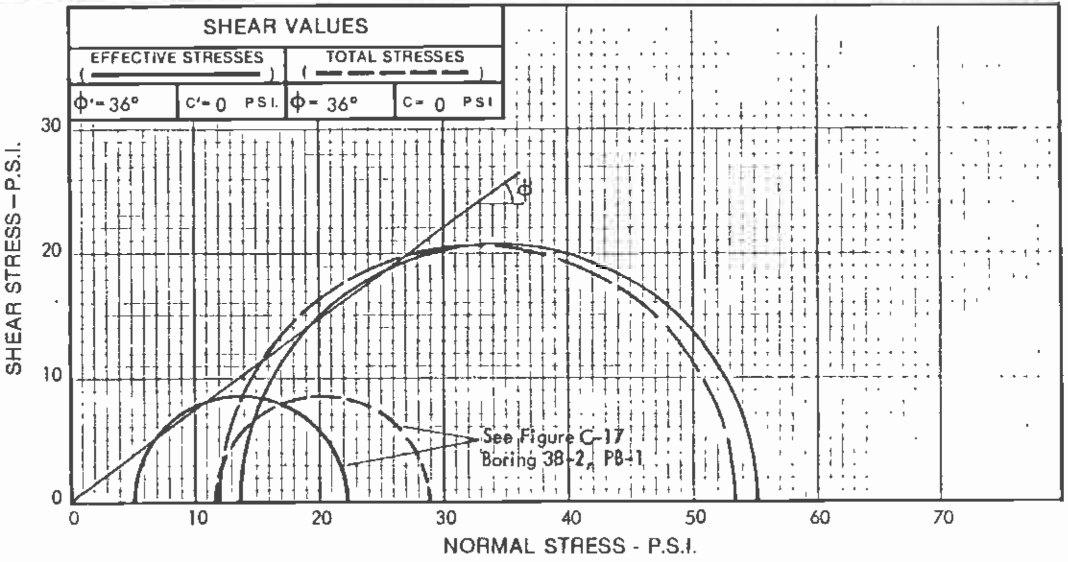

SHEAR VALUES			
EFFECTIVE STRESSES (————)		TOTAL STRESSES (— — — —)	
ϕ'= 36°	C'= 0 PSI.	ϕ= 36°	C= 0 PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-1	38-1	PB-1	12-14.5	SM	6.50	2.84	98.8	24.3	
									Pitcher
				Initial	6.50	2.84	98.8	20.2	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		12	41.8	-1.4	13.4	55.2	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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Project No
83-1140

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Figure No
C-14

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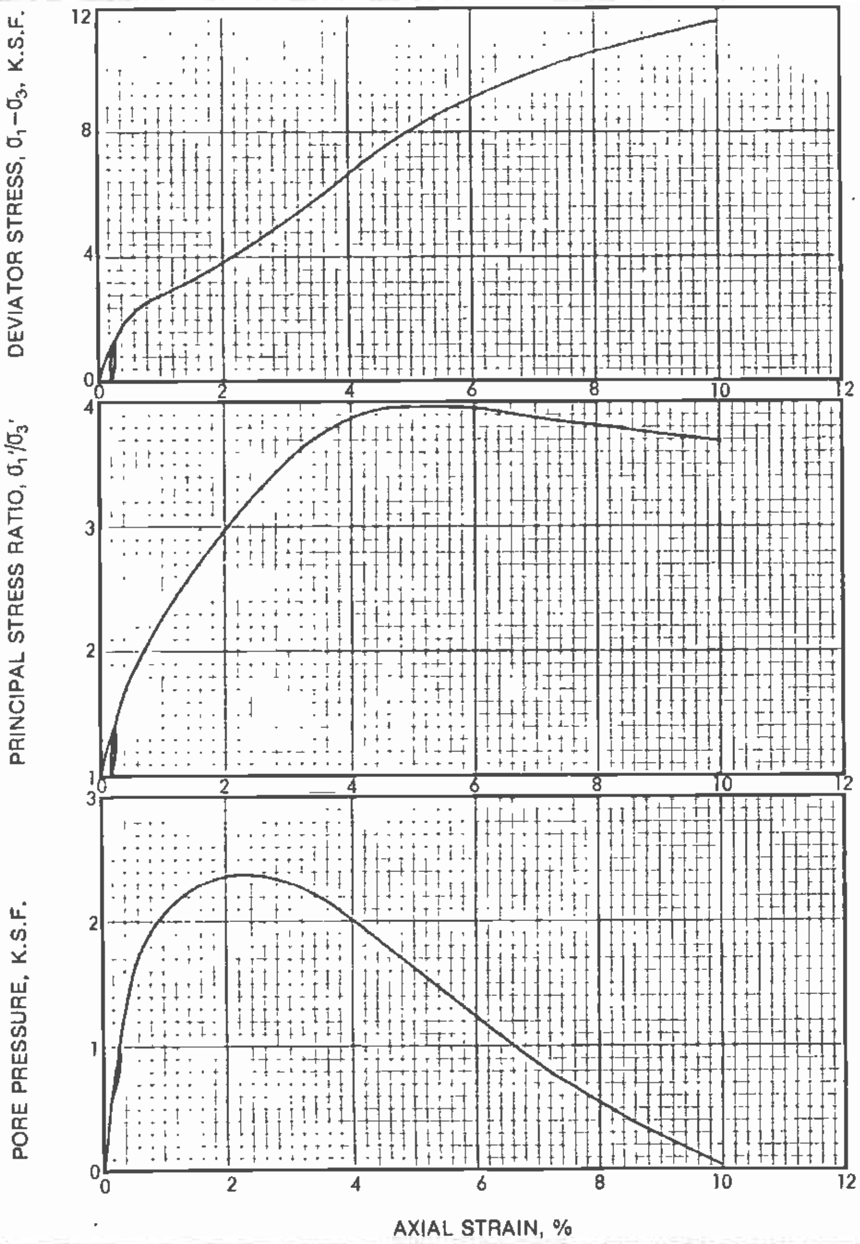

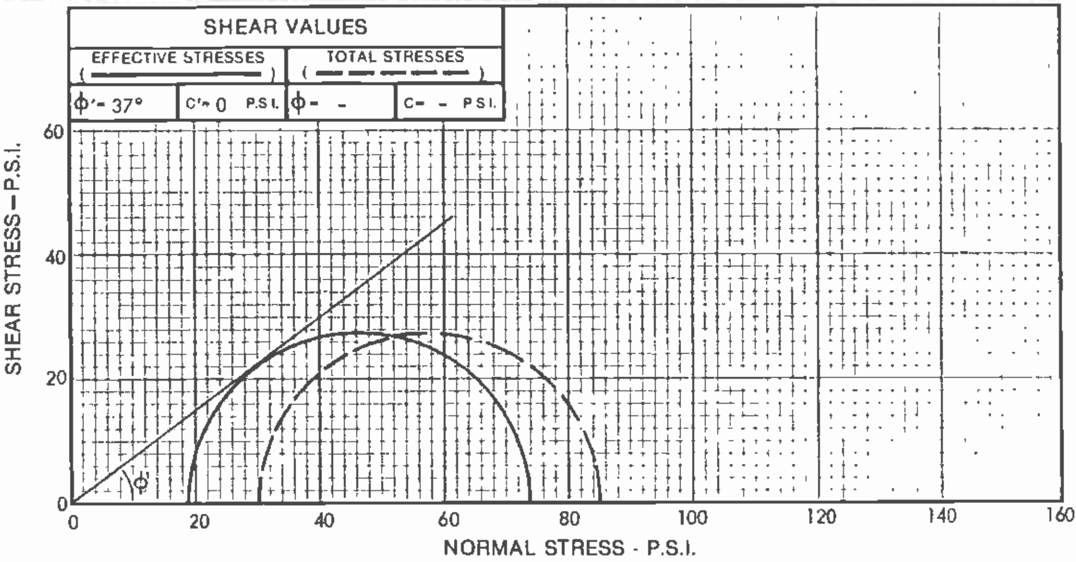

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ'= 37°	c'= 0 P.S.I.	φ= -	c= - P.S.I.

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 $\sigma_1-\sigma_3$ (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		30	55.3	11.3	18.7	74.0	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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Figure No
C-15

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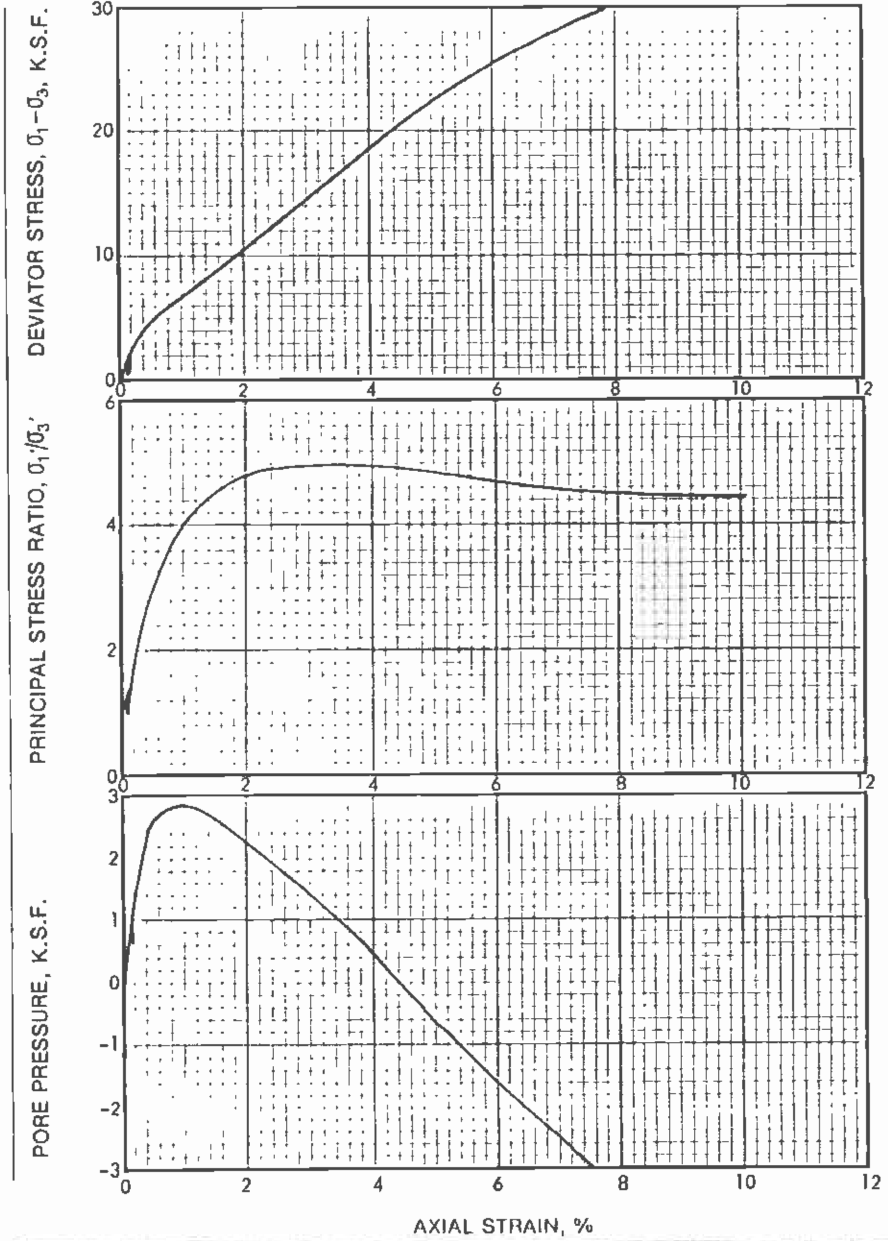

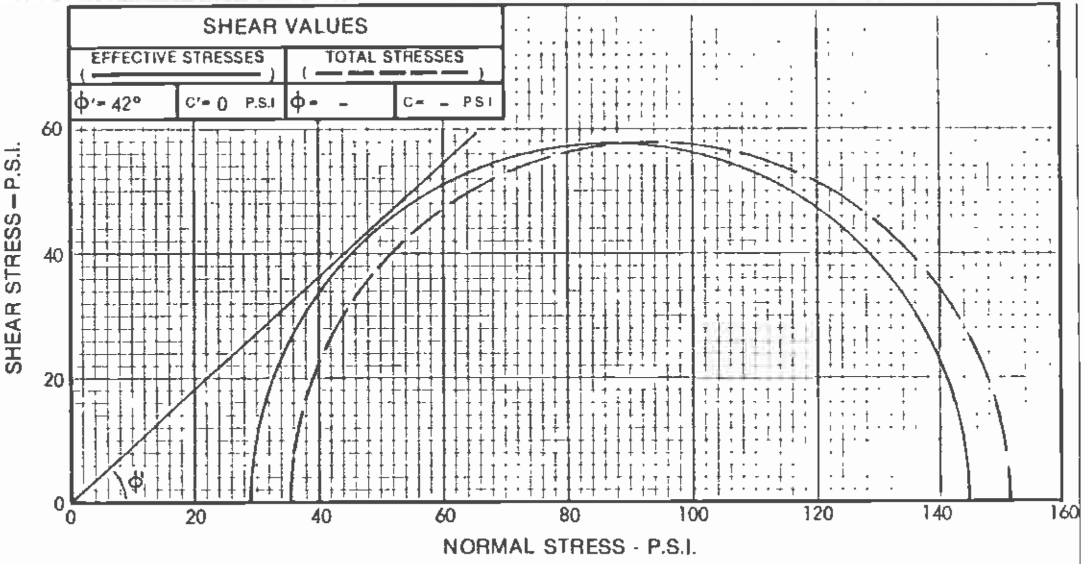

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
φ'= 42°	C'= 0 P.S.I	φ= -	C= - PSI

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-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 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		36	115.4	7.0	29.0	144.4	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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Figure No
C-16

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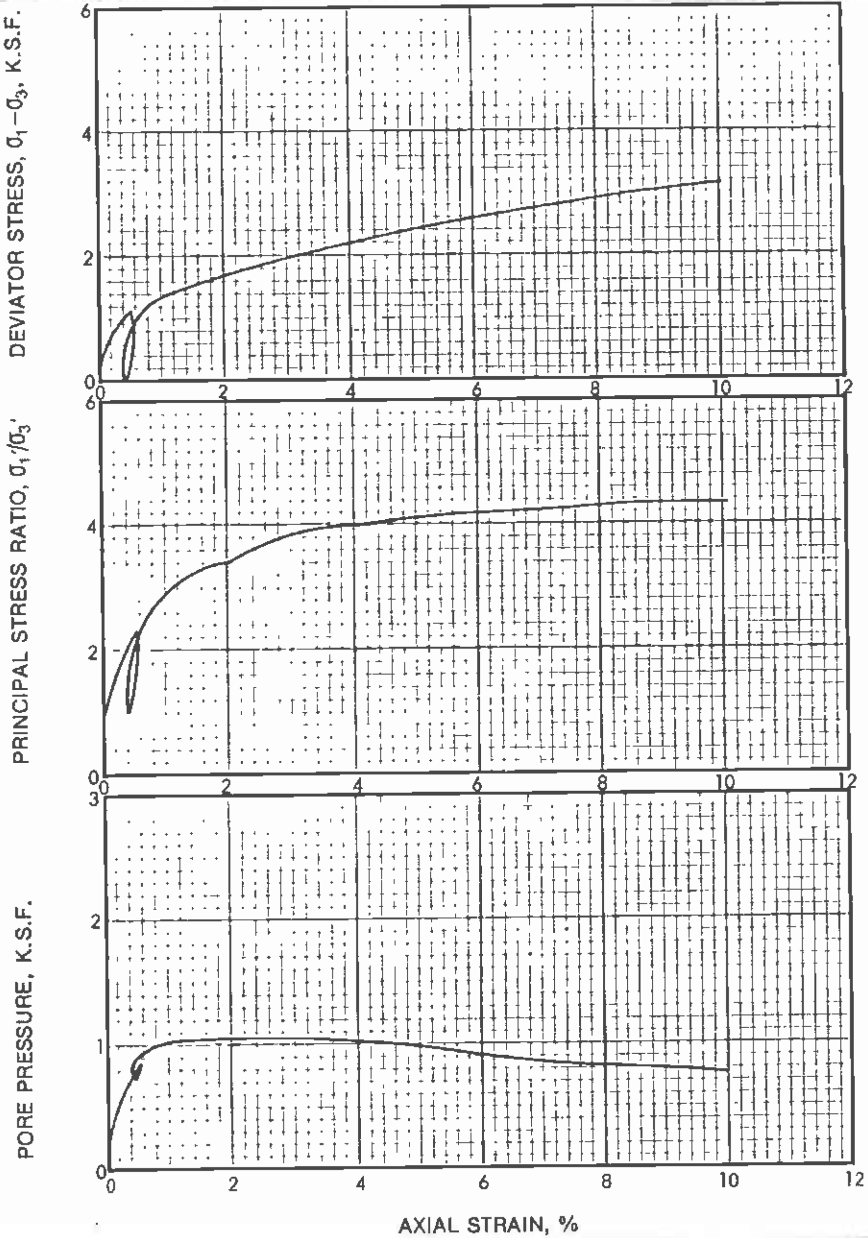

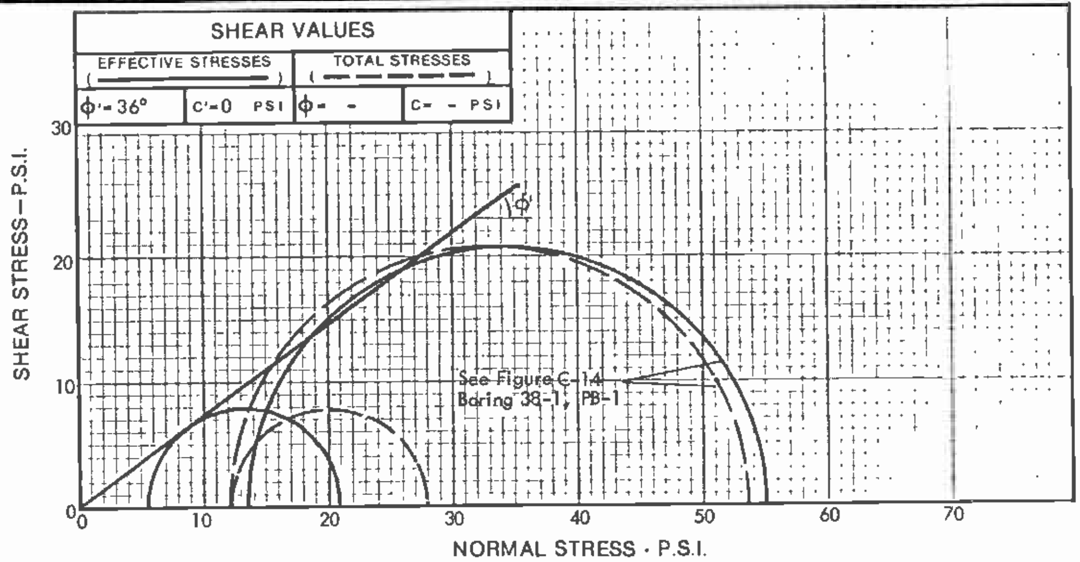

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ'= 36°	c'=0 PSI	φ= -	C= - PSI

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	38-2	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 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	16.5	6.7	5.3	21.8	ICU with Pore Pressure Measure-ments

TRIAXIAL COMPRESSION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

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Figure No
C-17

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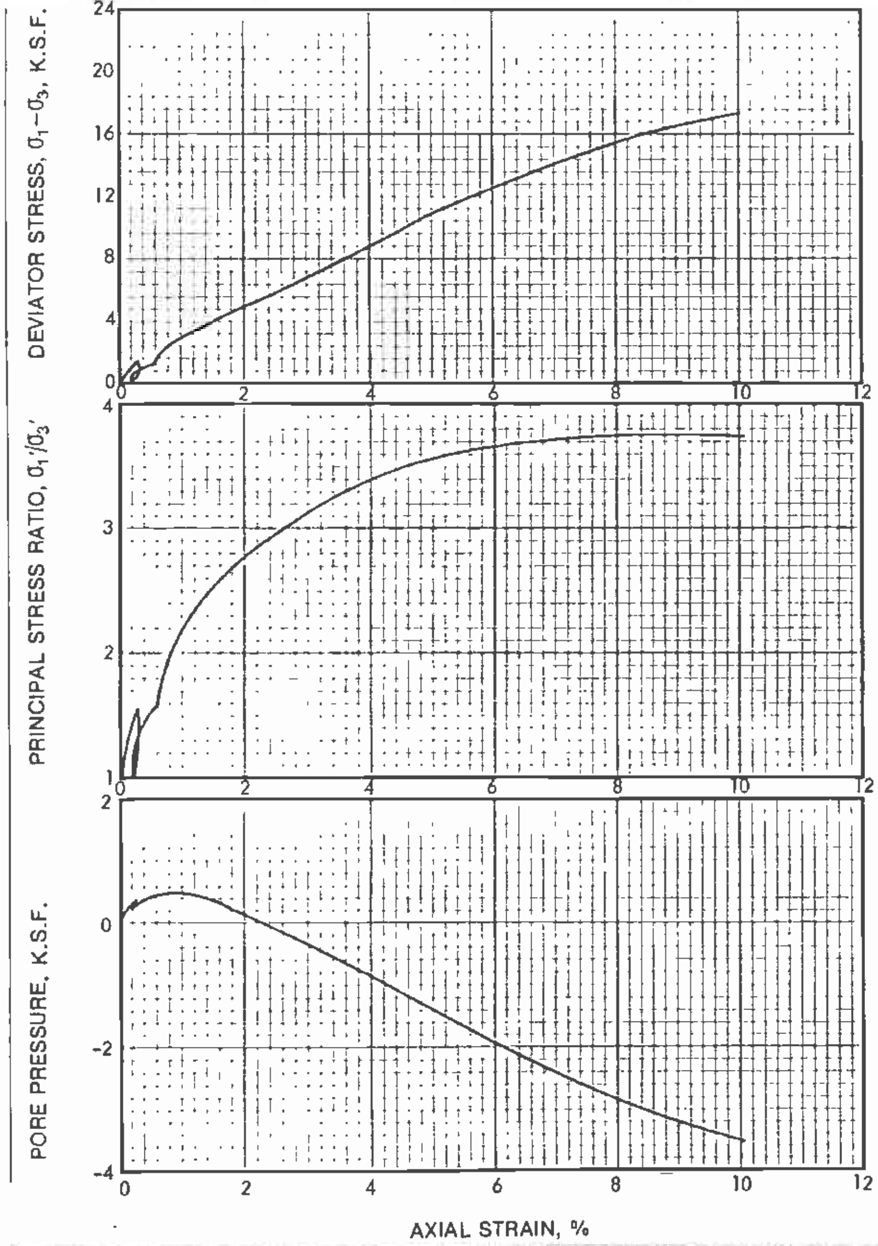

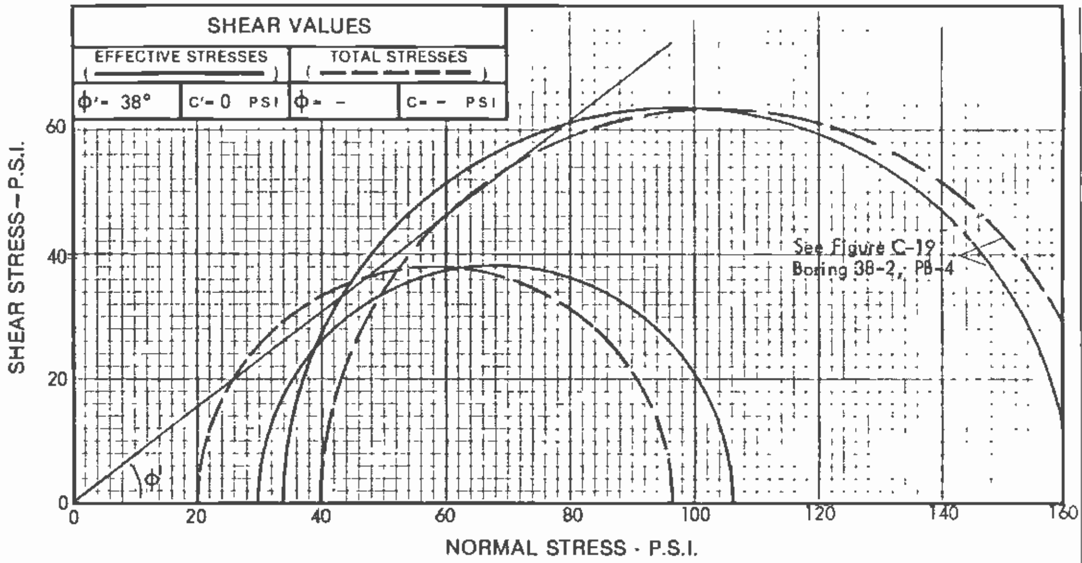

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (– – – –)	
ϕ'= 38°	C'= 0 PSI	ϕ= –	C= – PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-2	38-2	PB-2	22-24.5	SP	6.49	2.83	105.4	21.1	
				Initial	6.50	2.83	105.3	14.4	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-2		20	76.0	-9.8	29.8	105.8	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
C-18

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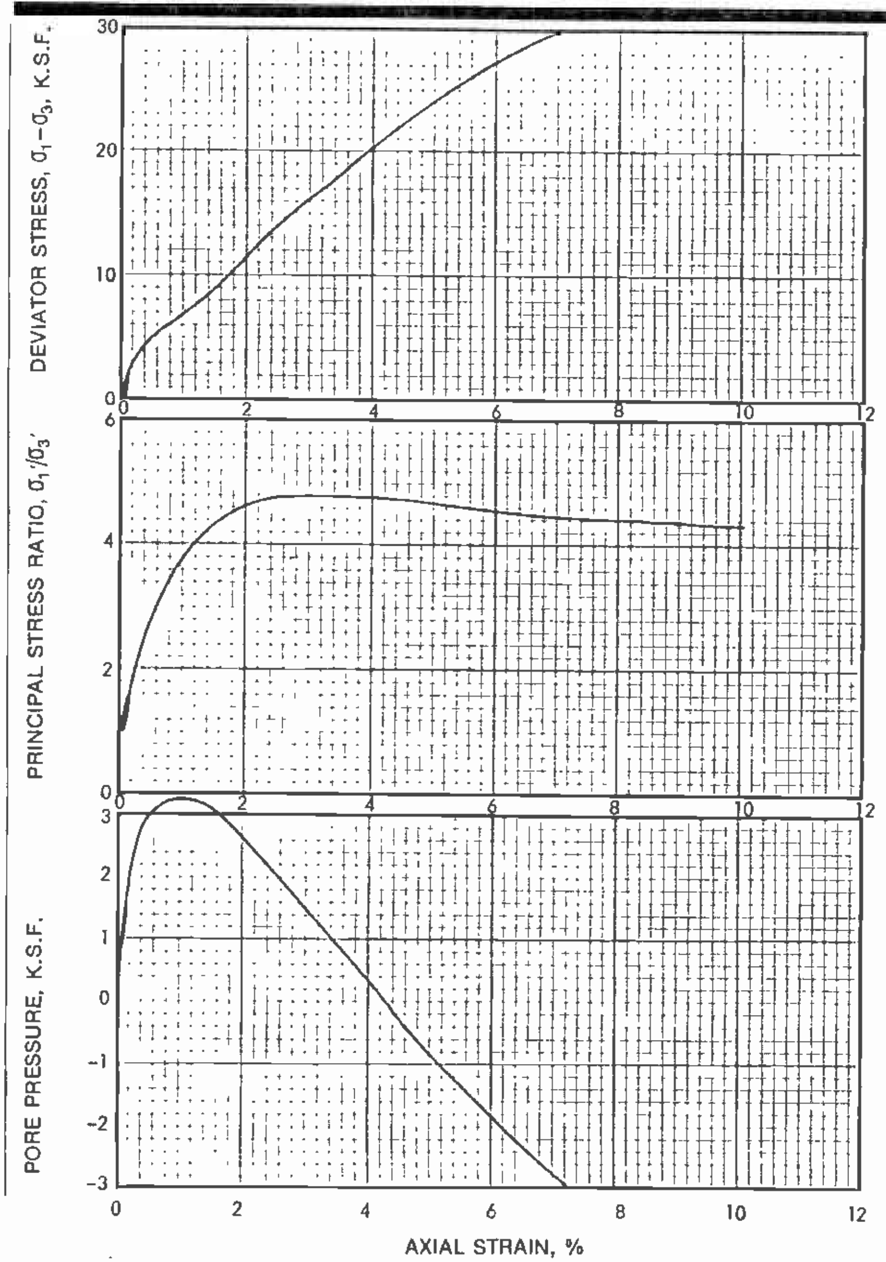

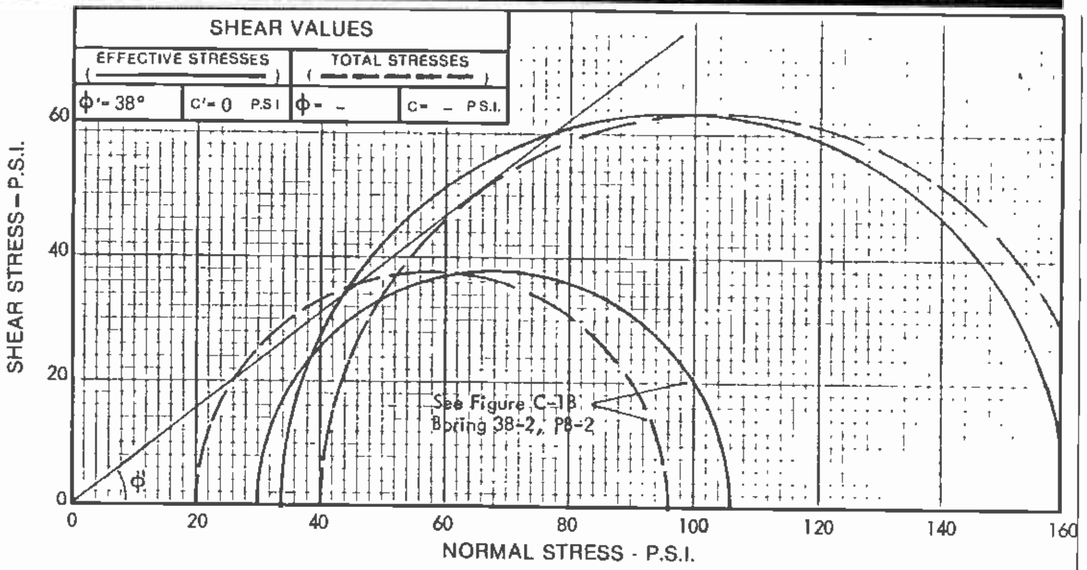

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-4	38-2	PB-4	42-44.5	SP/SM	6.08	2.91	108.9	17.1	Pitcher Undisturbed
				w/gravel					
				Initial	6.13	2.84	108.6	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-4		40	127.2	6.4	33.6	160.8	ICU with Pore Pressure Measurements

TRIAXIAL COMPRESSION TESTS

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METRO RAIL PROJECT

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Figure No
C-19

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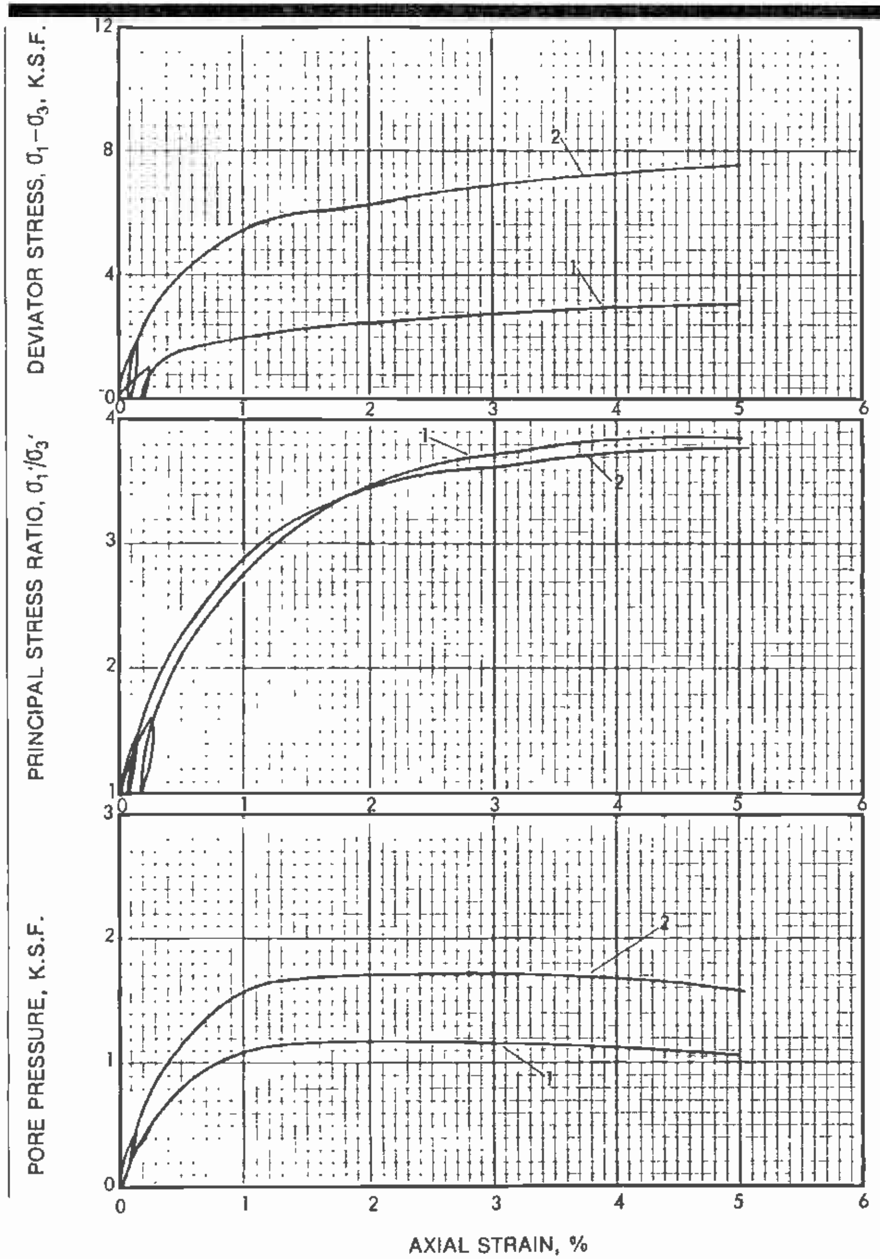

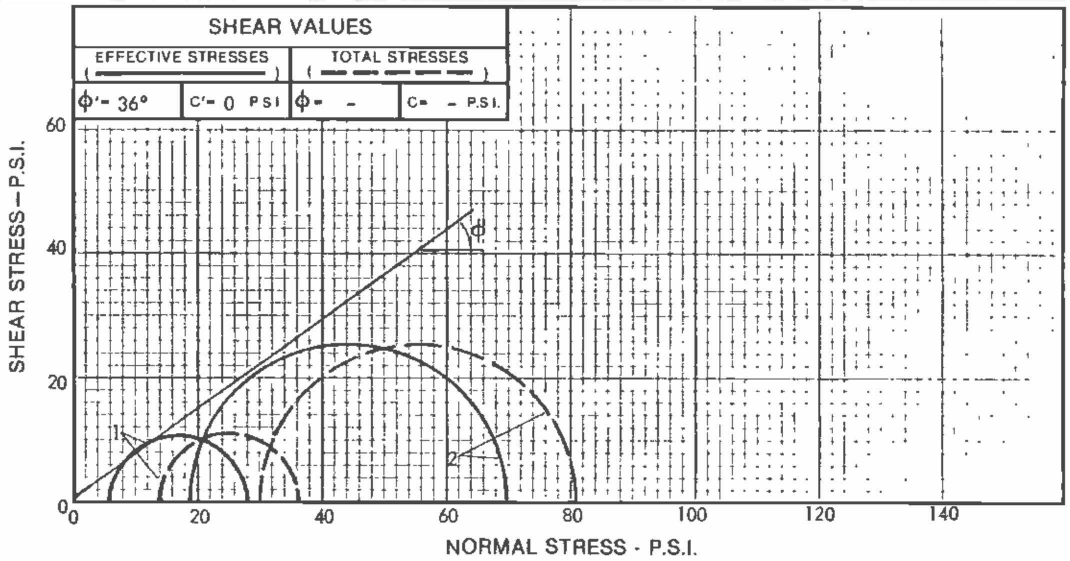

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
φ'= 36°	C'= 0 P.S.I.	φ= -	C= - P.S.I.

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
									Pitcher Undisturbed
				Initial	6.00	2.85	103.1	15.2	

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	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
C-20

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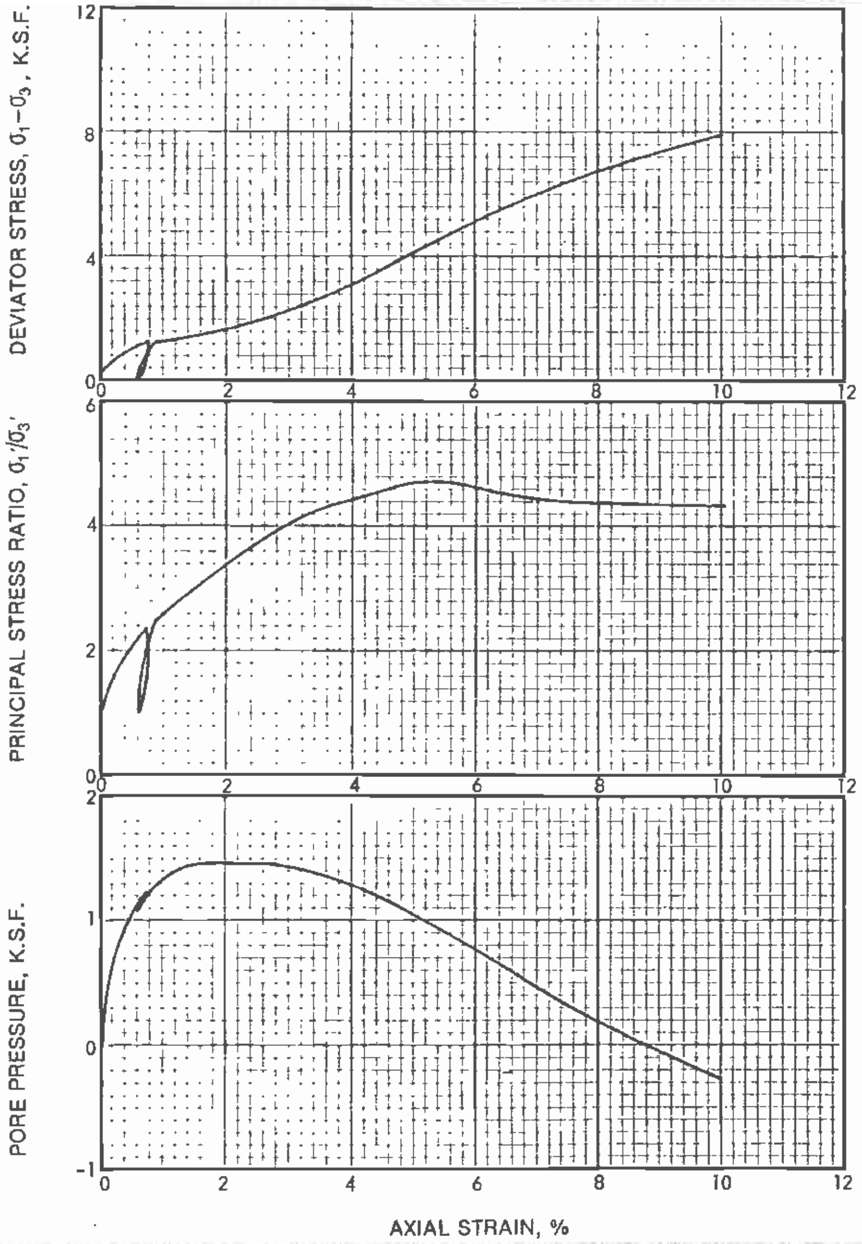

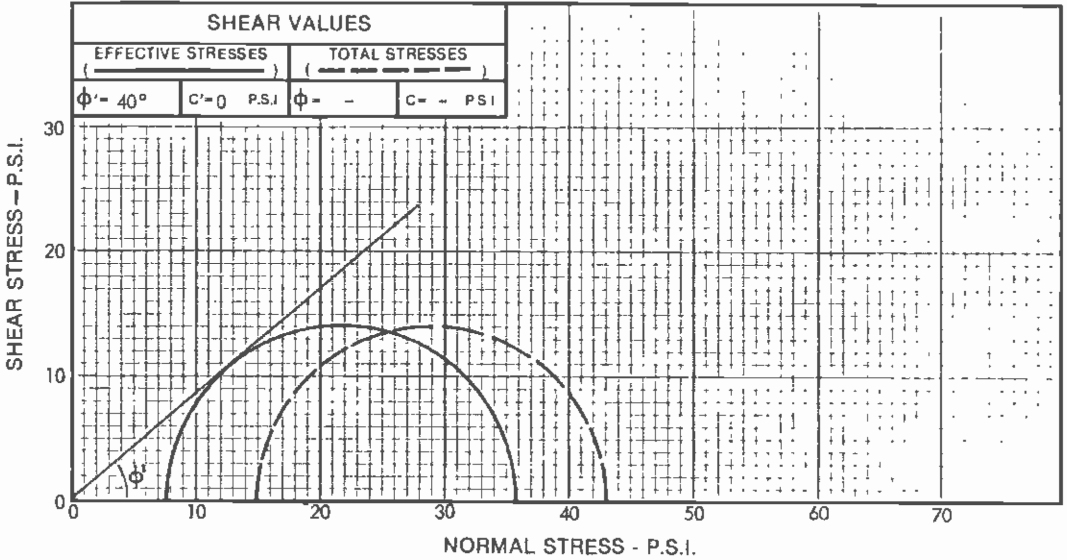

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ'= 40°	C'=0 P.S.I	ϕ= –	C= – PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	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 $\sigma_1-\sigma_3$ (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		15	28.4	7.3	7.7	36.1	ICU with Pore Pressure Measurements

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Figure No
C-21

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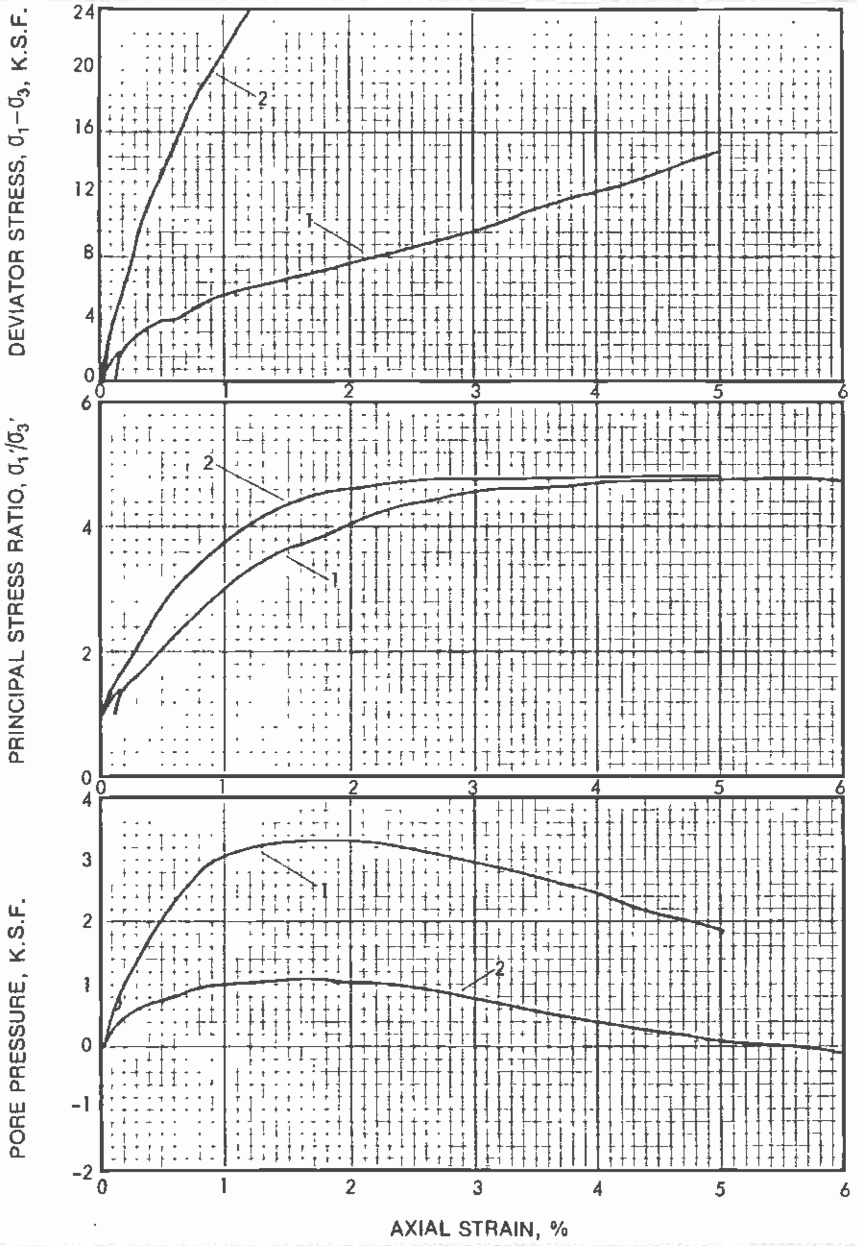

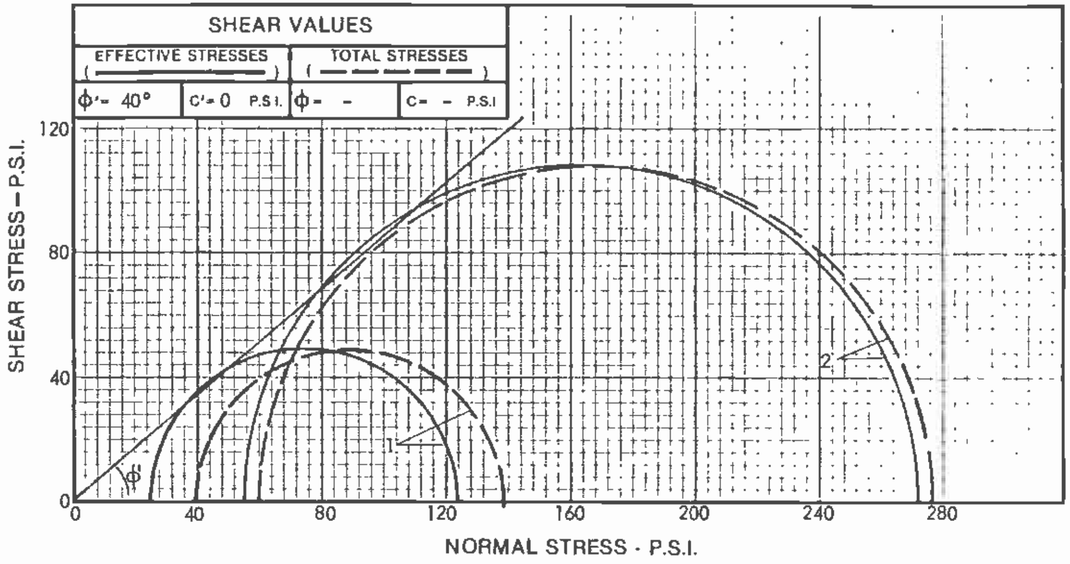

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
φ'= 40°	C'= 0 P.S.I.	φ= -	C= - P.S.I

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
C-22

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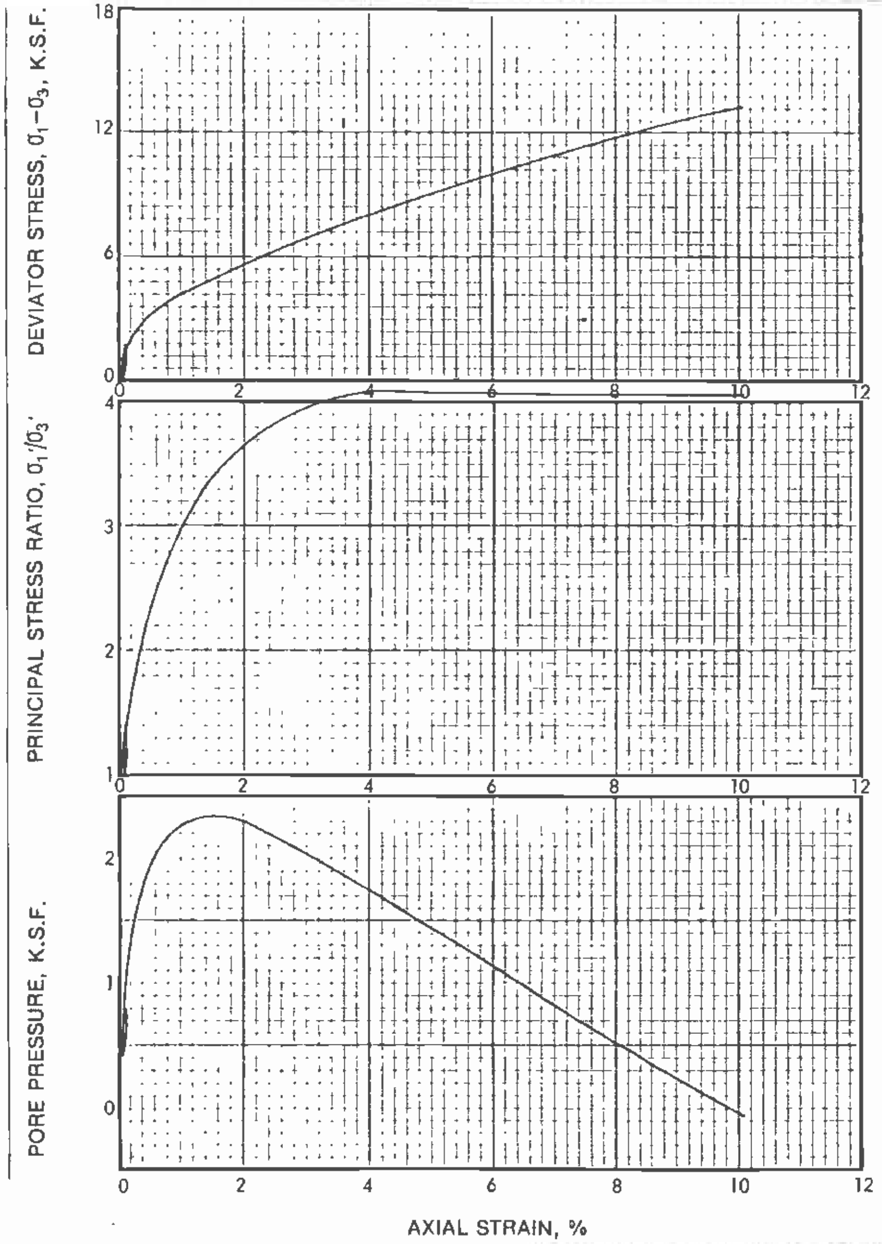

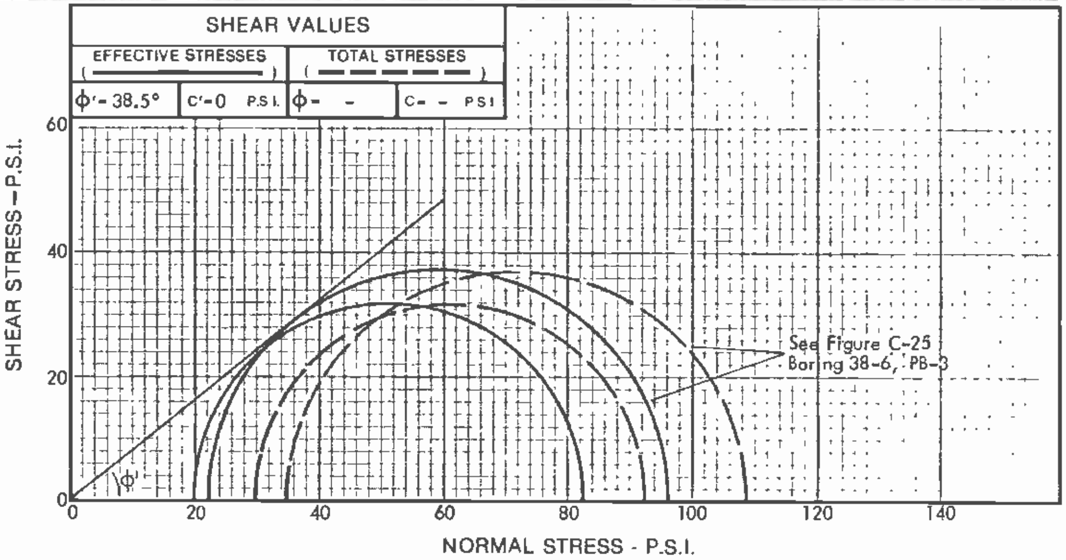

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ' - 38.5°	C'-0 P.S.I.	ϕ - -	C- - P.S.I.

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 σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
PB-2		30	63.0	9.8	20.2	83.2	ICU with Pore Pressure Measurements

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Figure No
C-23

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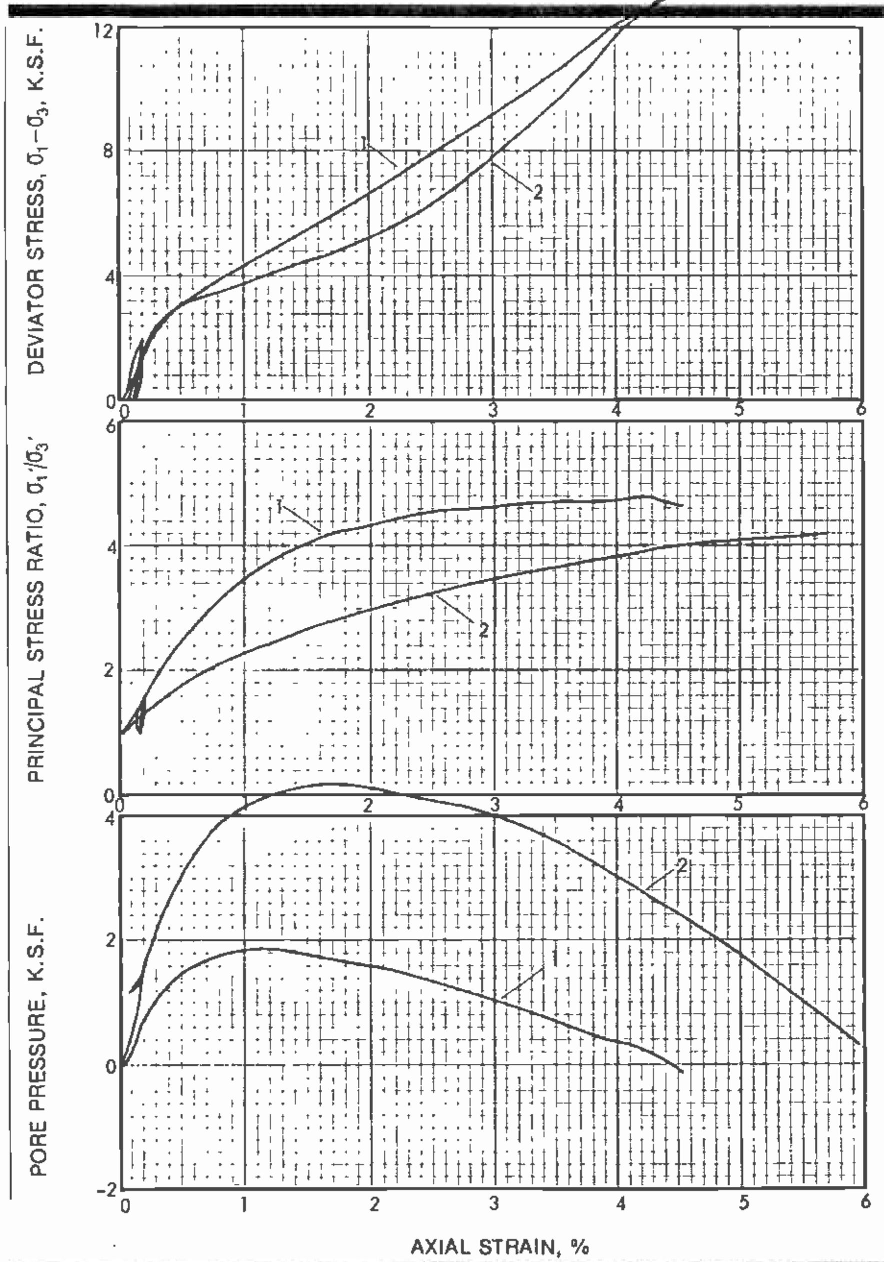

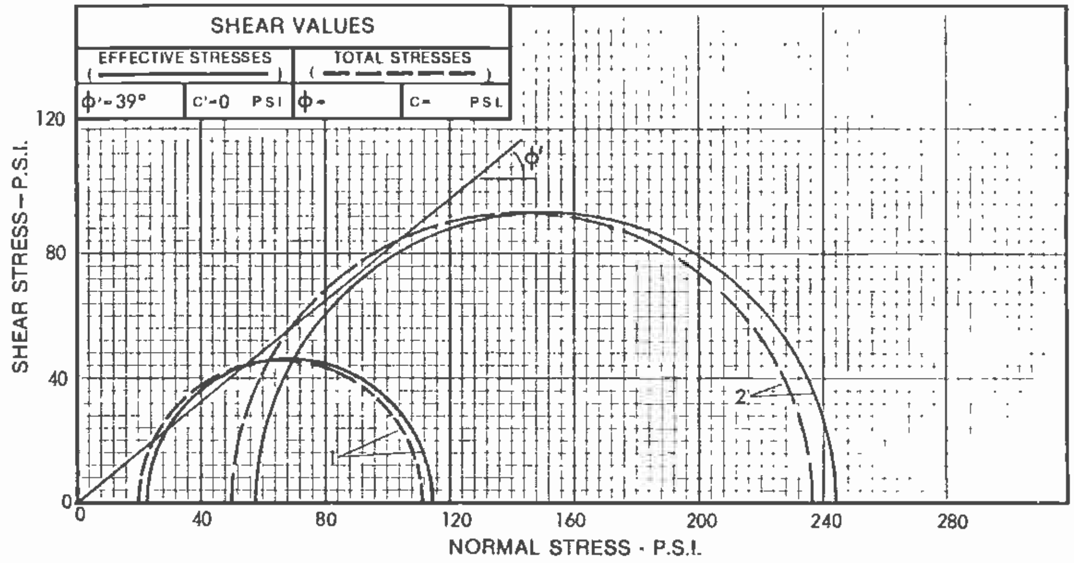

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'=39°	C'=0 PSI	ϕ=	C= PSI

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
PB-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
C-24

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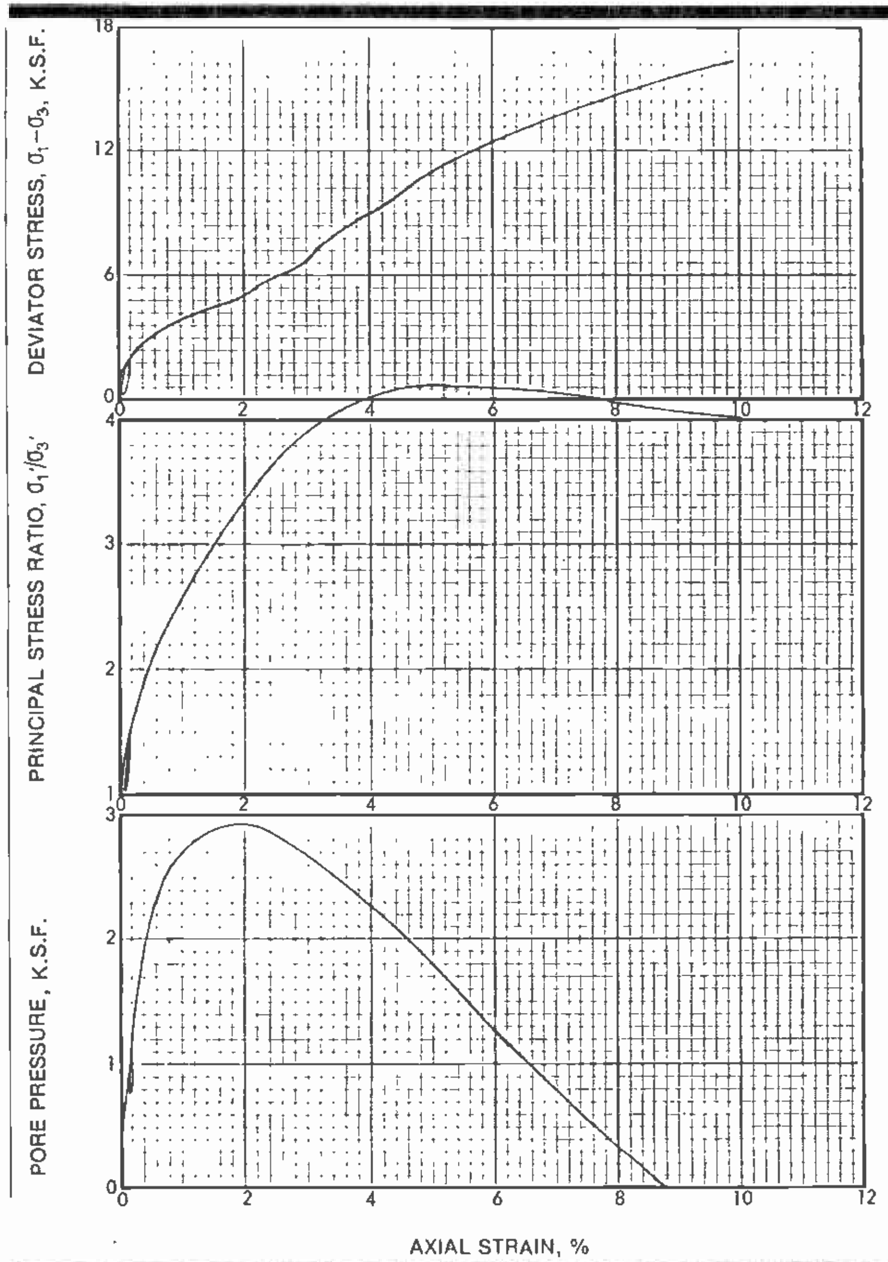

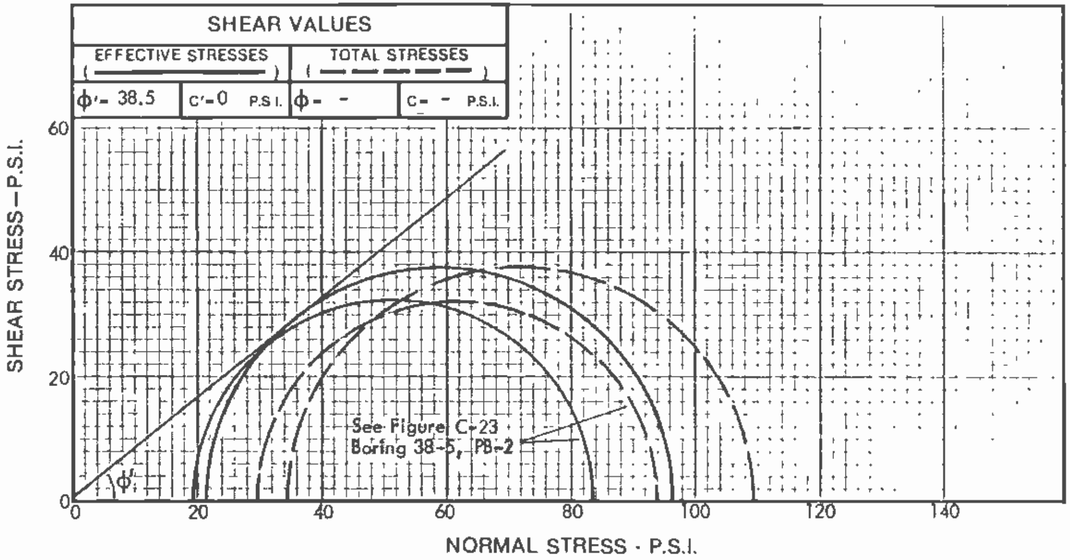

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
φ'= 38.5	c'=0 P.S.I.	φ= -	c= - P.S.I.

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-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
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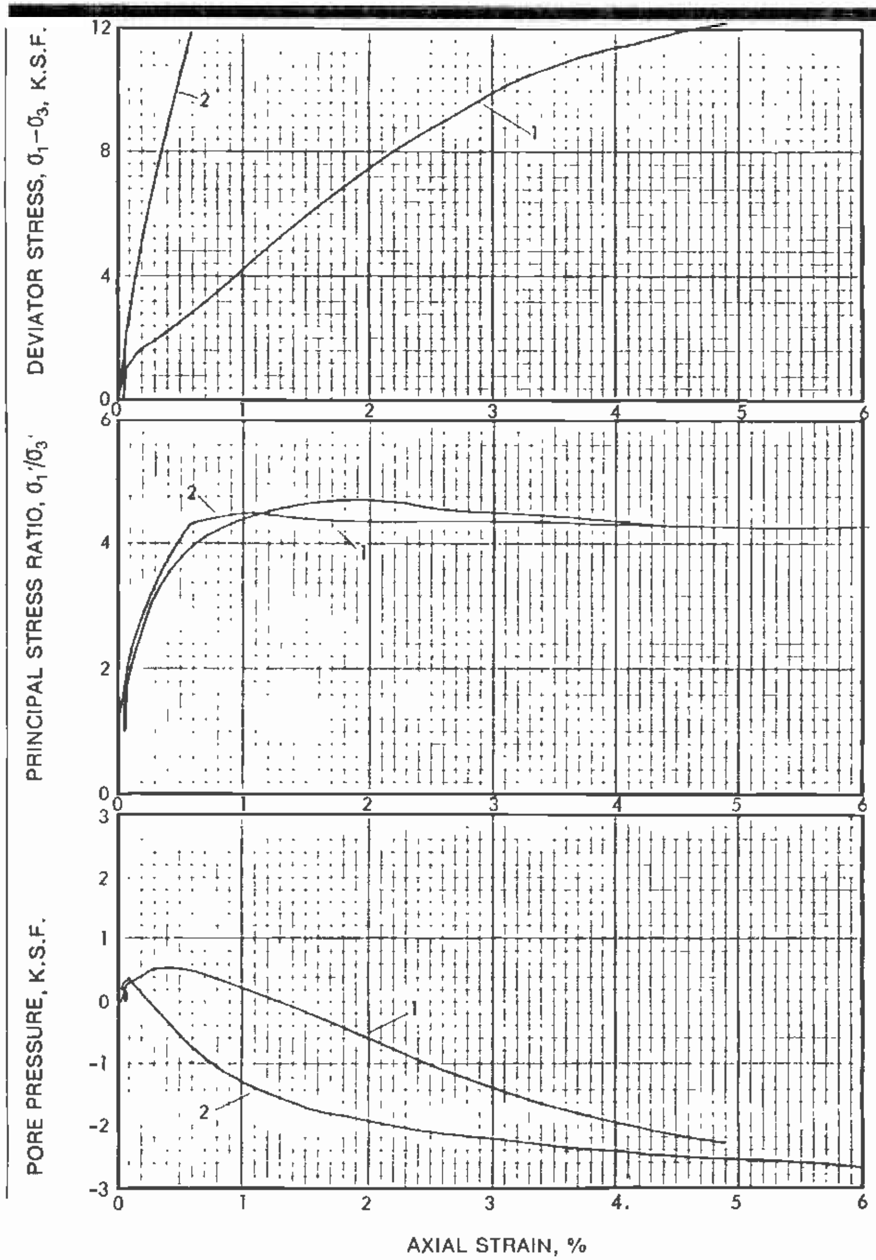

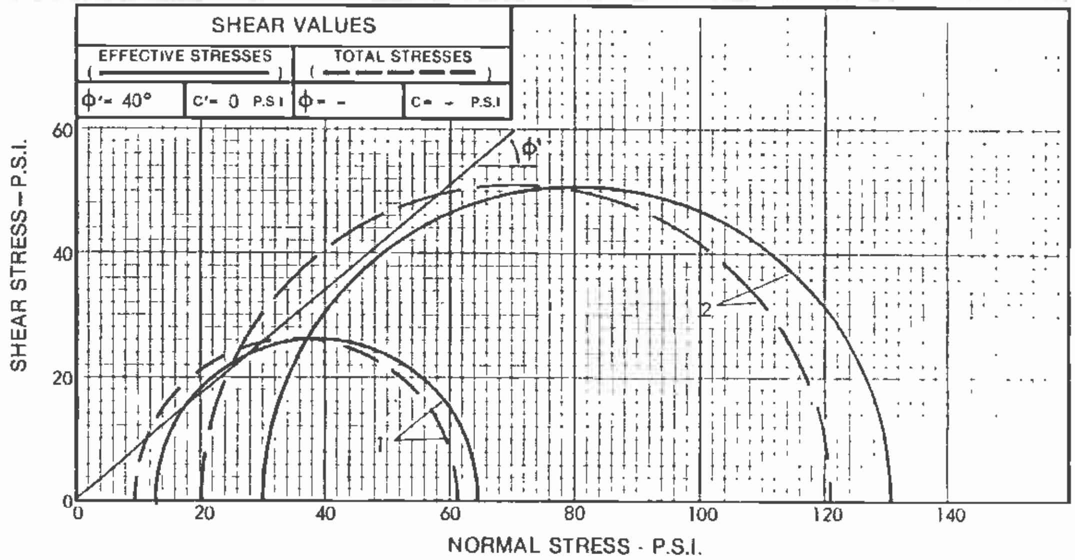

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ'= 40°	C'= 0 P.S.I	ϕ= -	C= - P.S.I

SPECIMEN NUMBER	SPECIMEN LOCATION			SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI-FICATION	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	-	
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	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	10	50.6	-3.7	13.7	64.3	Two Stage ICU with
PB-1	2	20	101.5	-9.6	29.6	131.1	Pore Pressure measure-
							ments

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Figure No
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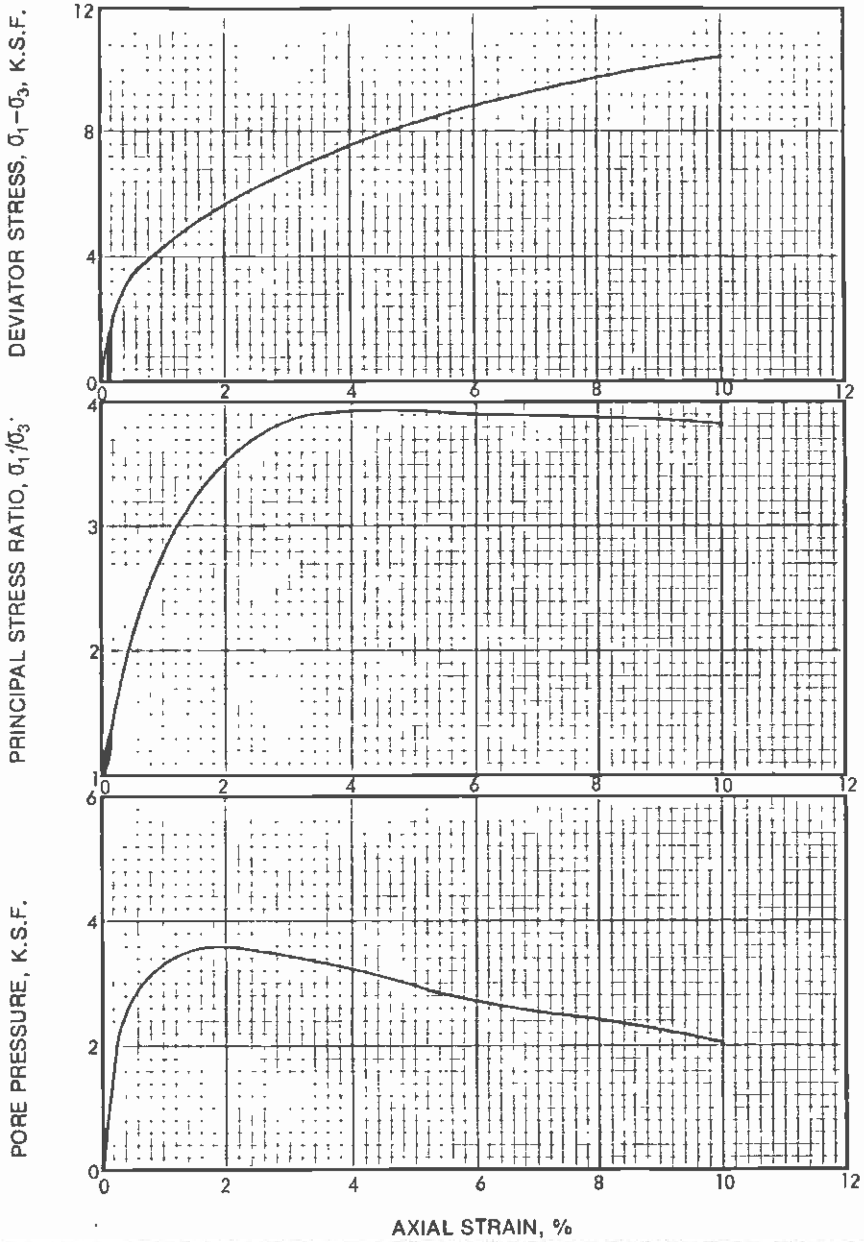
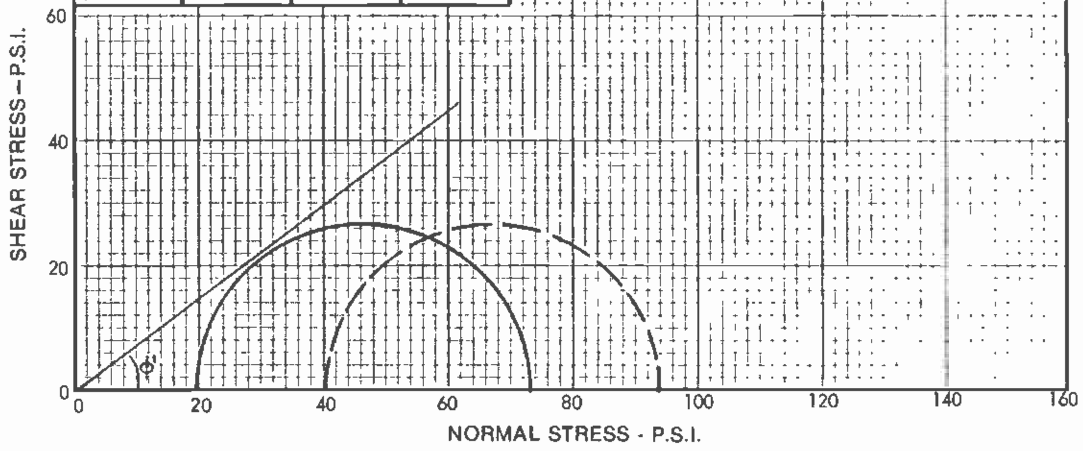


SHEAR VALUES			
EFFECTIVE STRESSES (————)		TOTAL STRESSES (— — — —)	
ϕ'= 37°	C'= 0 P.S.I.	ϕ= -	C= - P.S.I.



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-4	38-7	PB-4	41.5-43.6	SM	6.29	2.84	111.4	16.5	
				Initial	6.34	2.84	110.8	16.4	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-4		40	55.0	21.3	18.7	73.7	ICU with Pore Pressure Measurements

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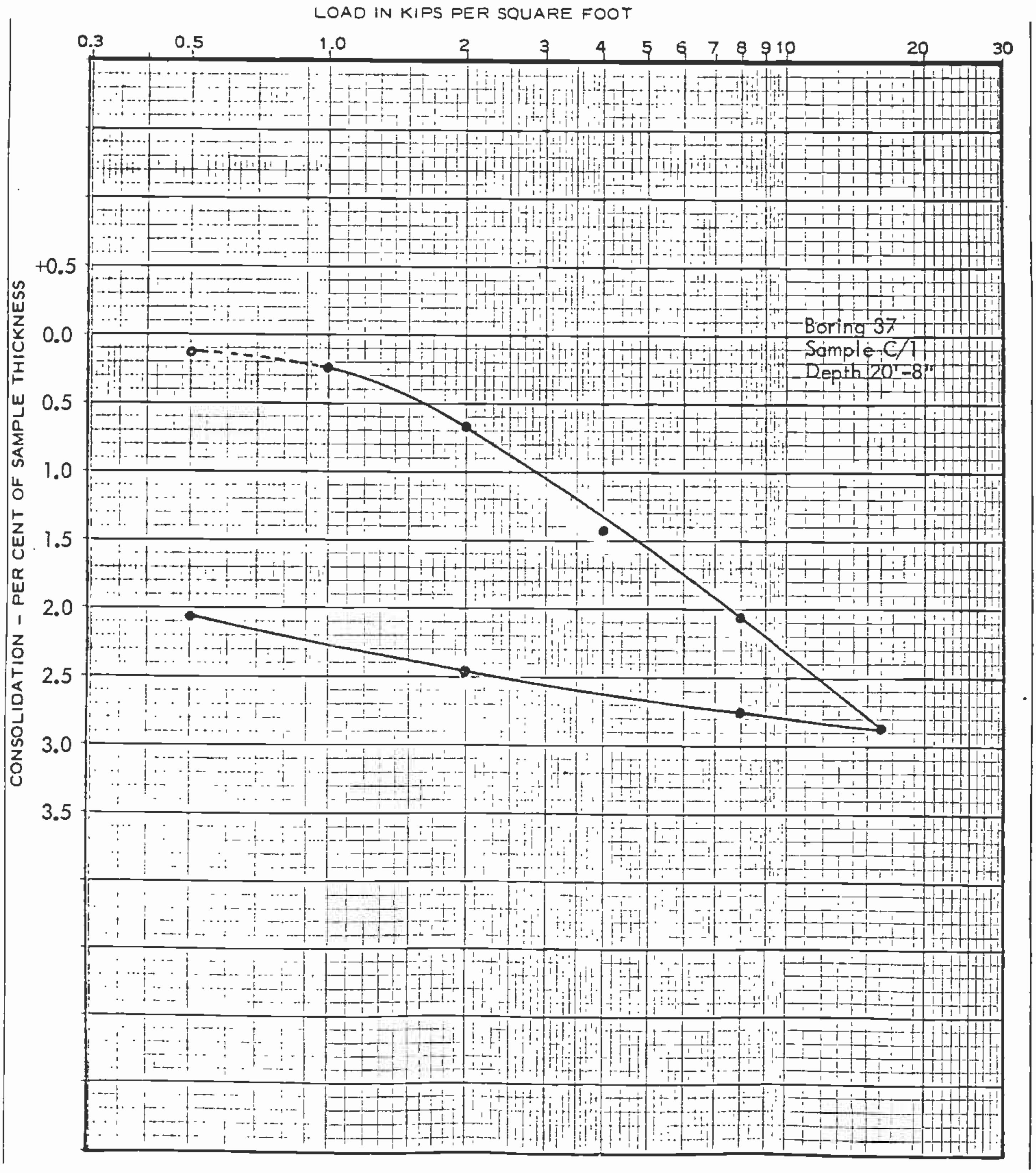
Figure No
C-27



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Project No.
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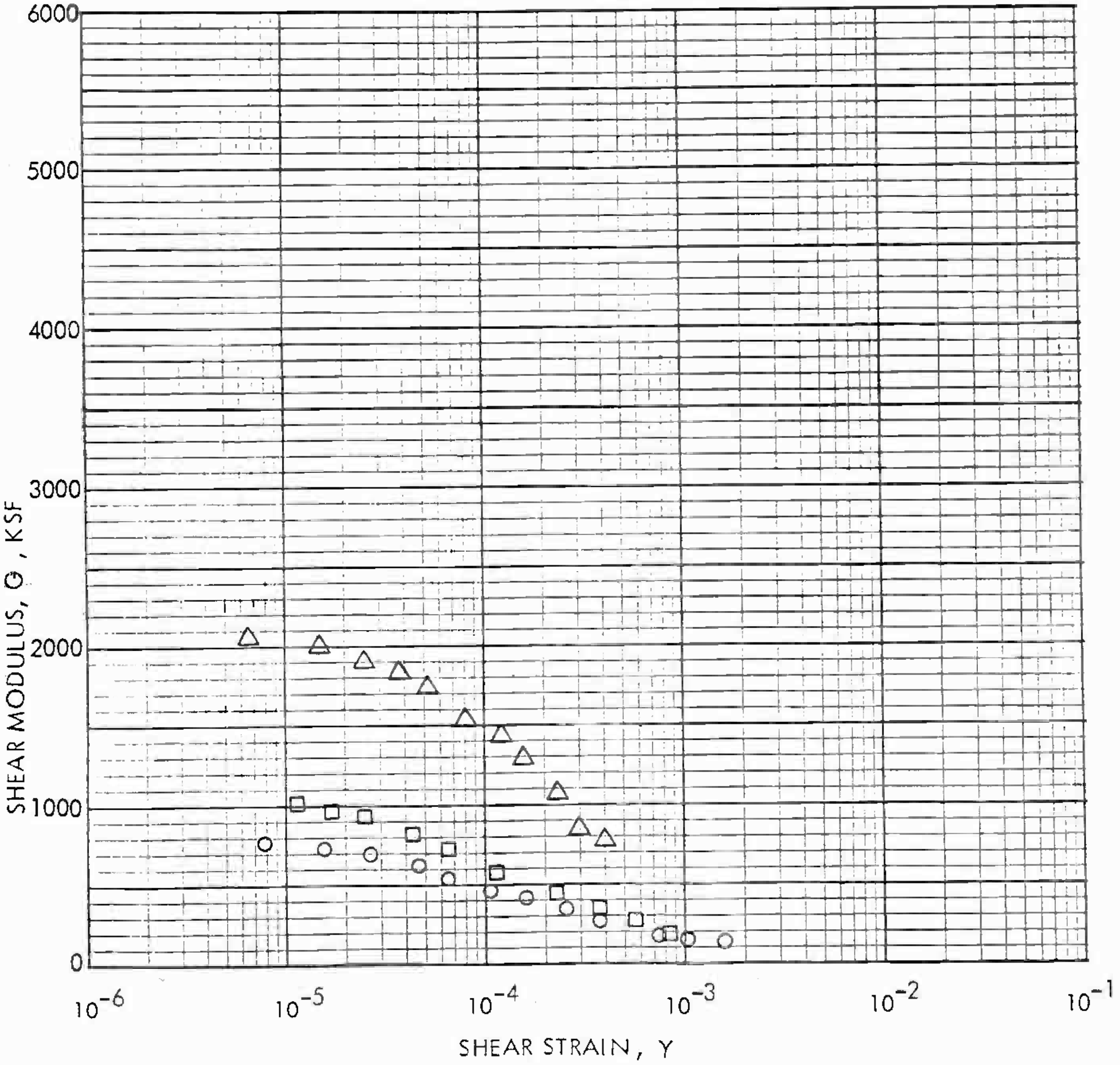
Figure No.
C-28

FORM NO. DI Approved for publication 5/8/91 by

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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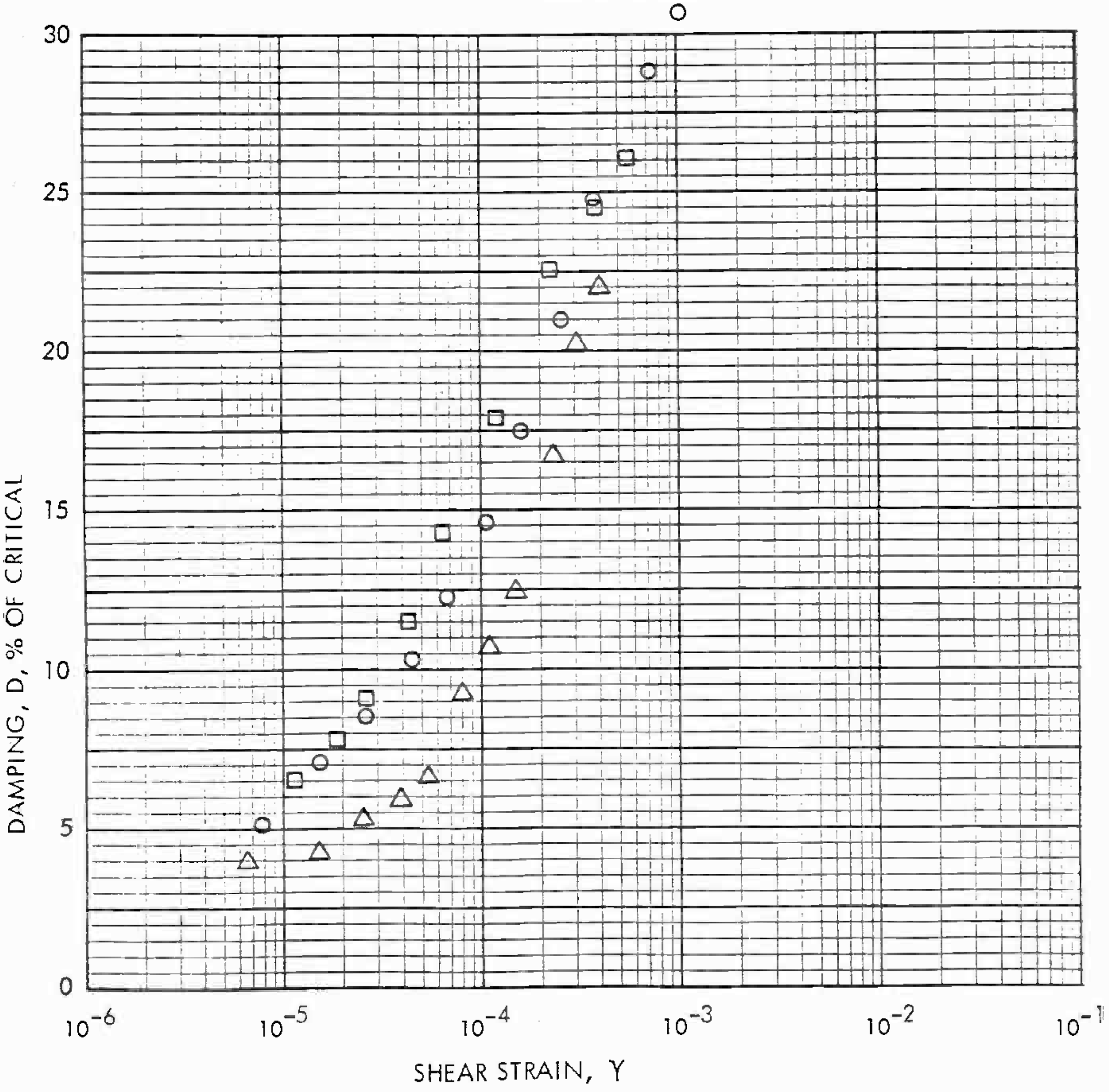
Figure No.
C-29

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

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RESONANT COLUMN TEST

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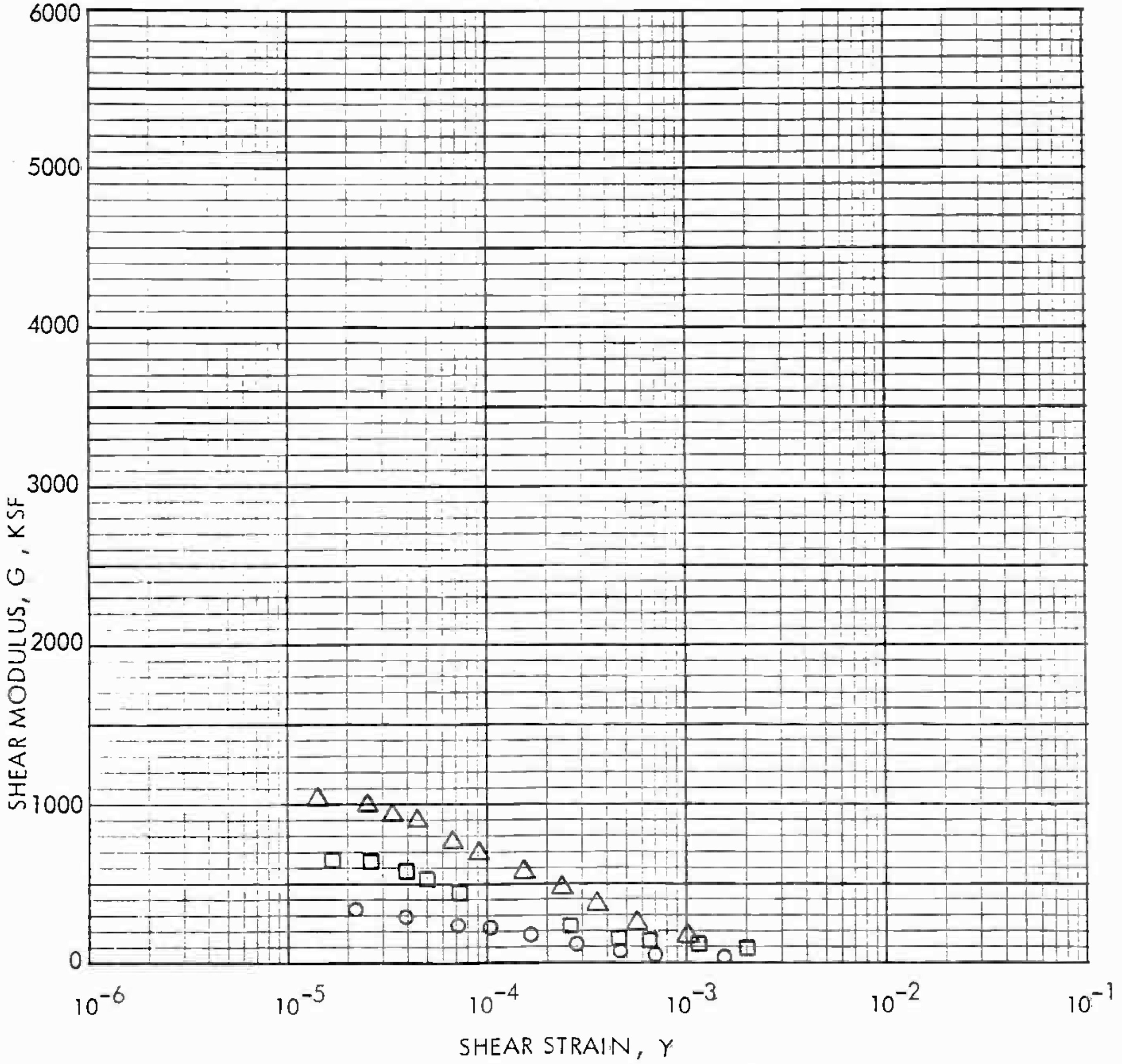
Figure No.
C-30

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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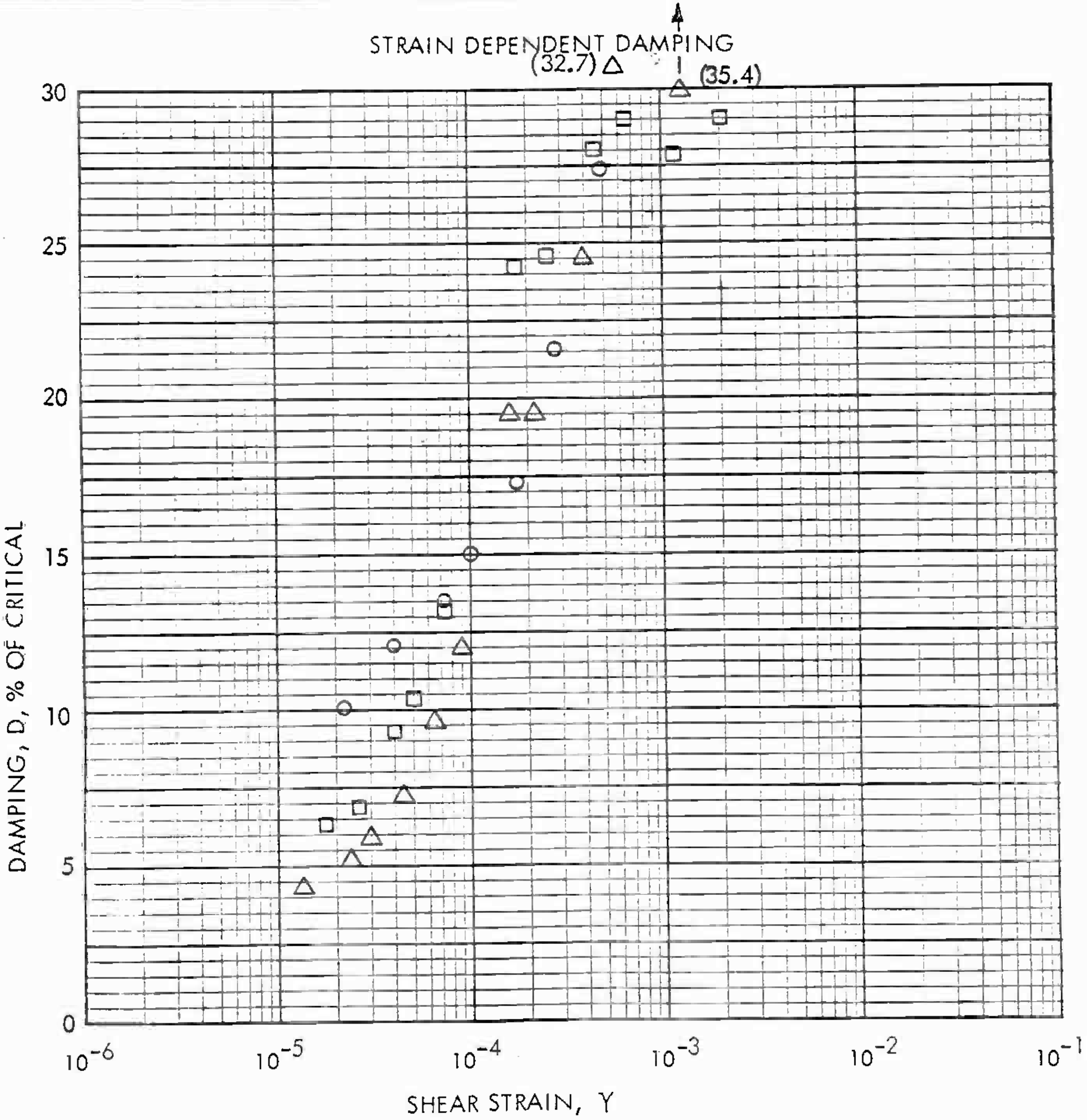
Figure No.
C-31



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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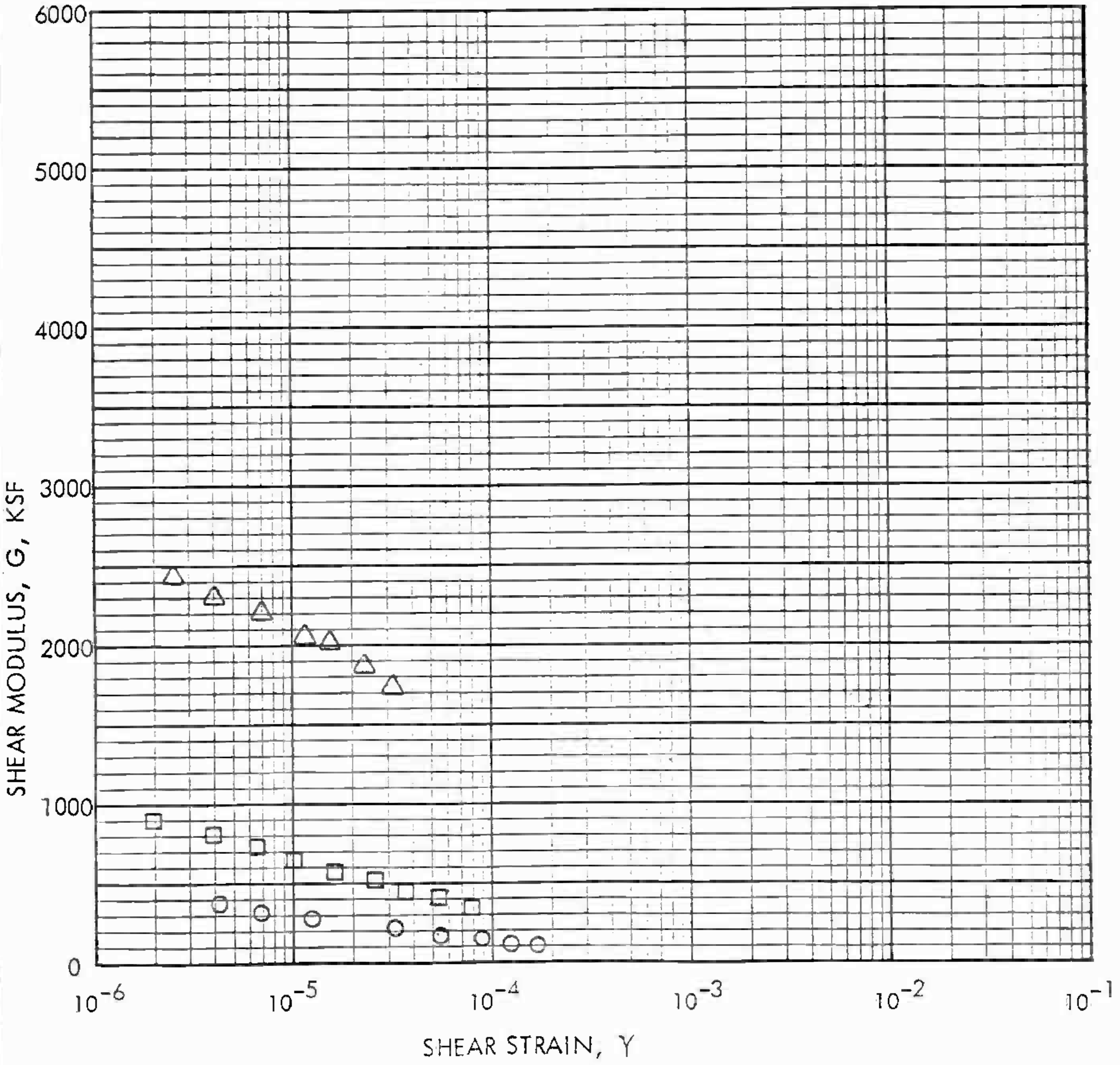
Figure No.
C-32

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

Approved for publication 5/[illegible] by [illegible]

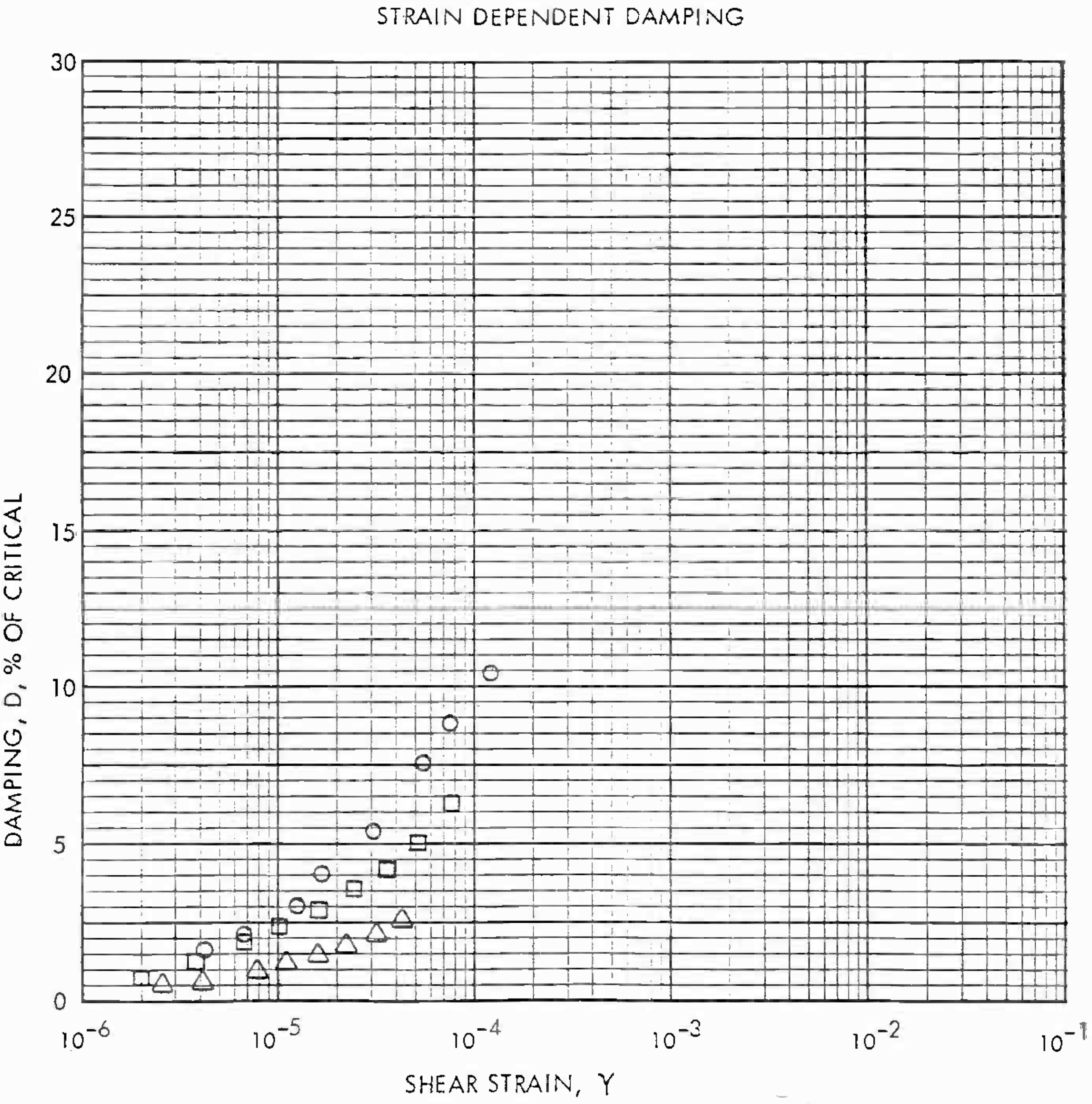
RESONANT COLUMN TEST

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-33



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

Approved for publication by

RESONANT COLUMN TEST

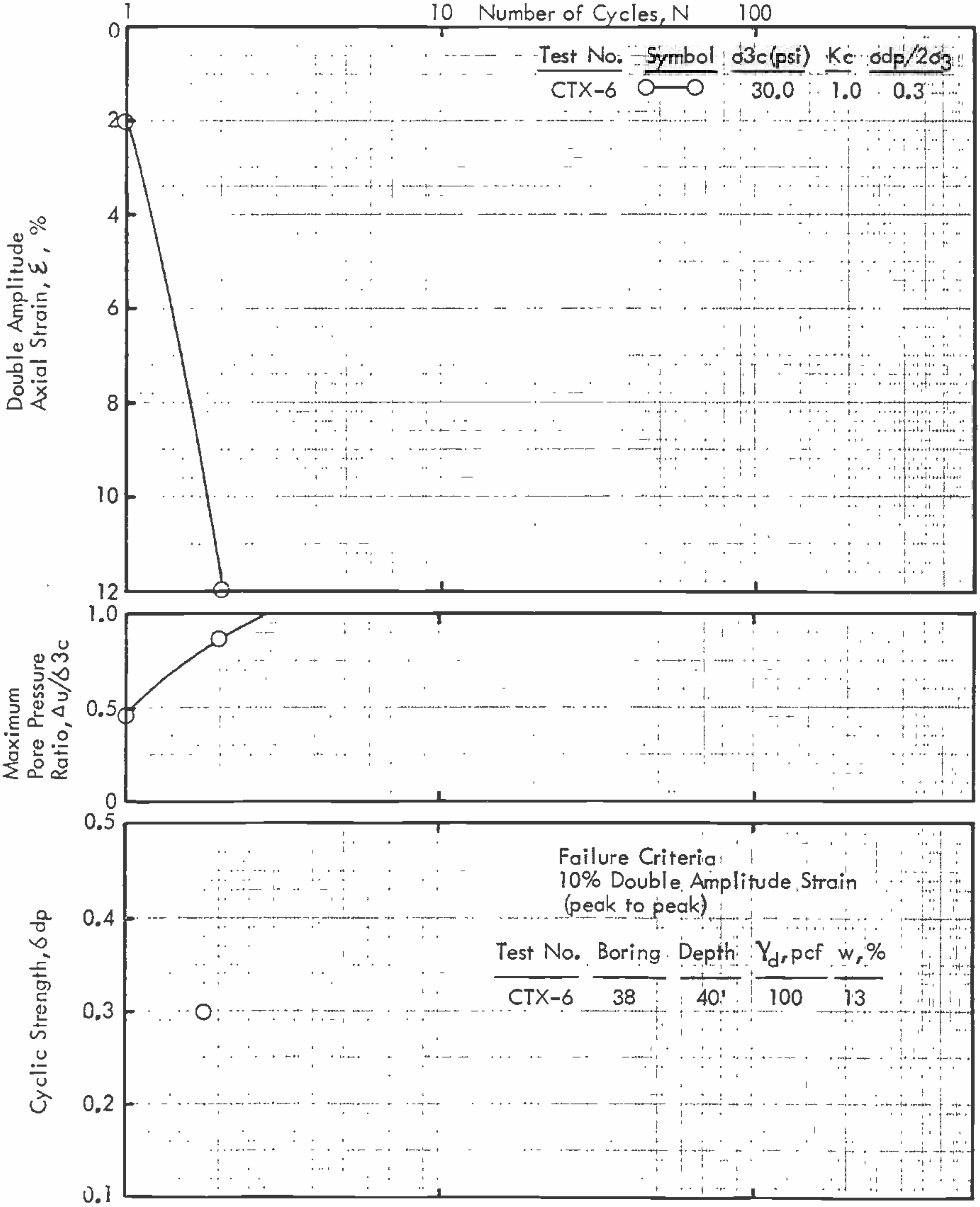
DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-34

Approved for publication 5/14 by



CYCLIC TRIAXIAL COMPRESSION TEST RESULTS

DESIGN UNIT A430
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METRO RAIL PROJECT

Project No.
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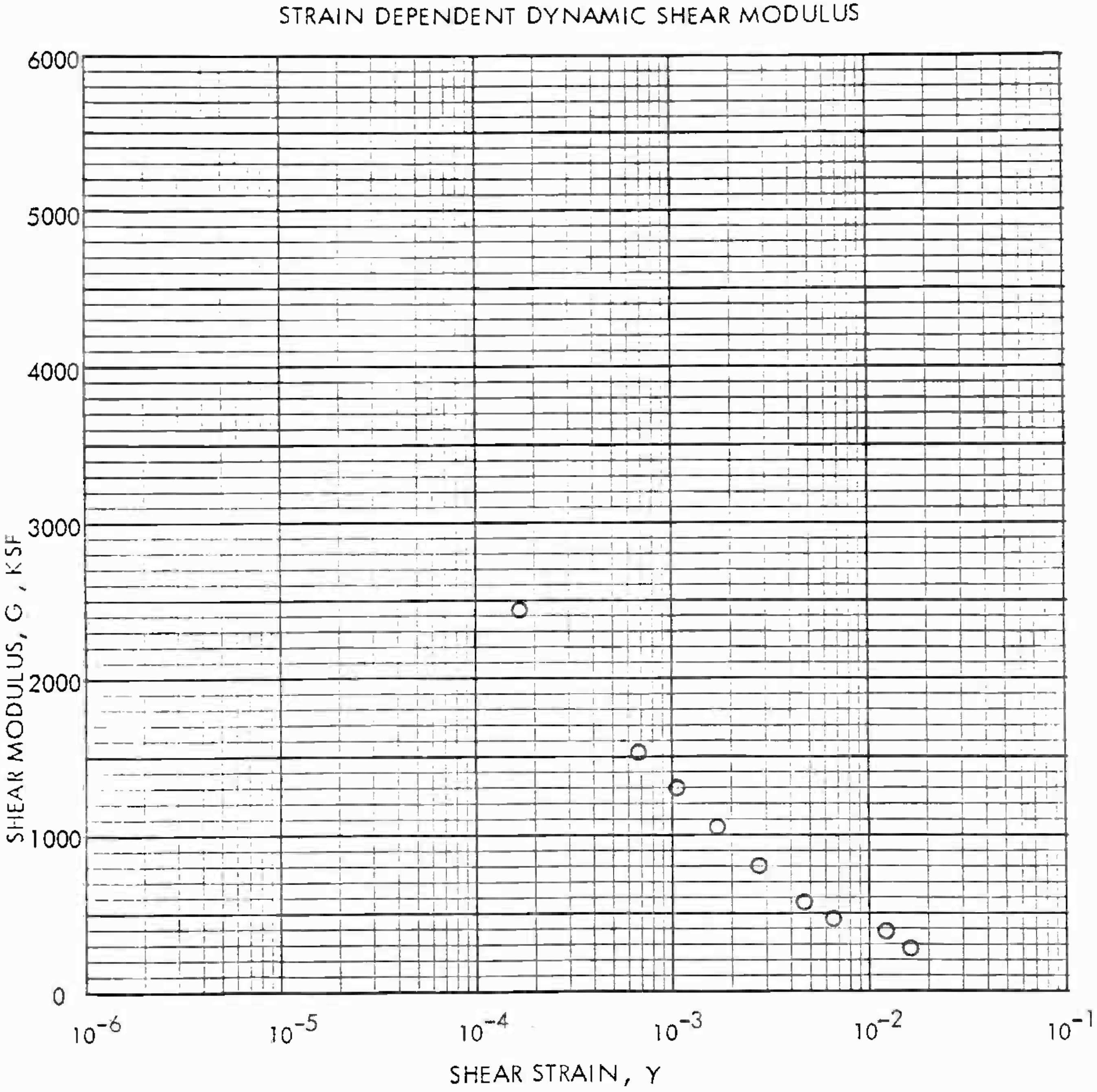
Figure No.
C-35



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o (%)	$\bar{\sigma}_c$(PSI)
37	C-3	60	112	19	30

Sample Description: Brown Silty Sand; dense, fine to medium graded

Approved for publication 5/84 by TAD

DYNAMIC TRIAXIAL TEST

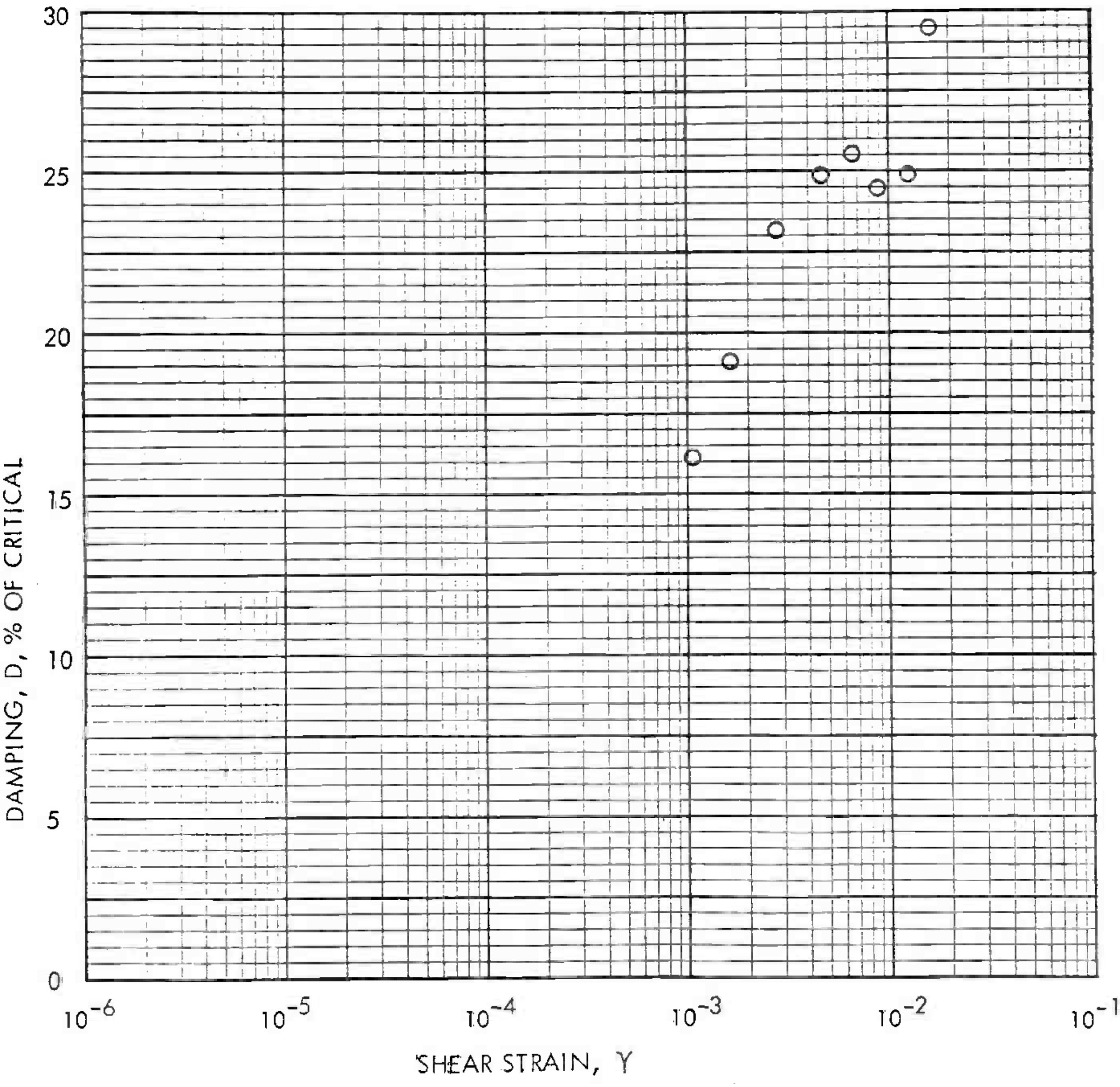
DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-36

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
37	C-3	60	112	19	30

Sample Description: Brown Silty Sand; dense, fine to medium graded

pproved for publication 5/[illegible] by FAD

DYNAMIC TRIAXIAL TEST

DESIGN UNIT A430
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-37

Appendix D

Pump Test

APPENDIX D PUMP TEST

D.1 SITE CONDITIONS

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

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

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

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

D.2 WELL CONSTRUCTION AND DEVELOPMENT

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

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

D.3 PUMP TESTING PROCEDURE

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

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

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

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

D.4 TEST INTERPRETATIONS

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

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

A check interpretation is shown on Figure D-3 which shows distance drawdown plots for both tests. The first test was influenced by boundary effects resulting in a relatively low transmissivity. The second test is probably high in terms of transmissivity. However, the average of these two interpretations is probably not far off. Table D-1 below summarizes the more reliable test results.

TABLE D-1

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

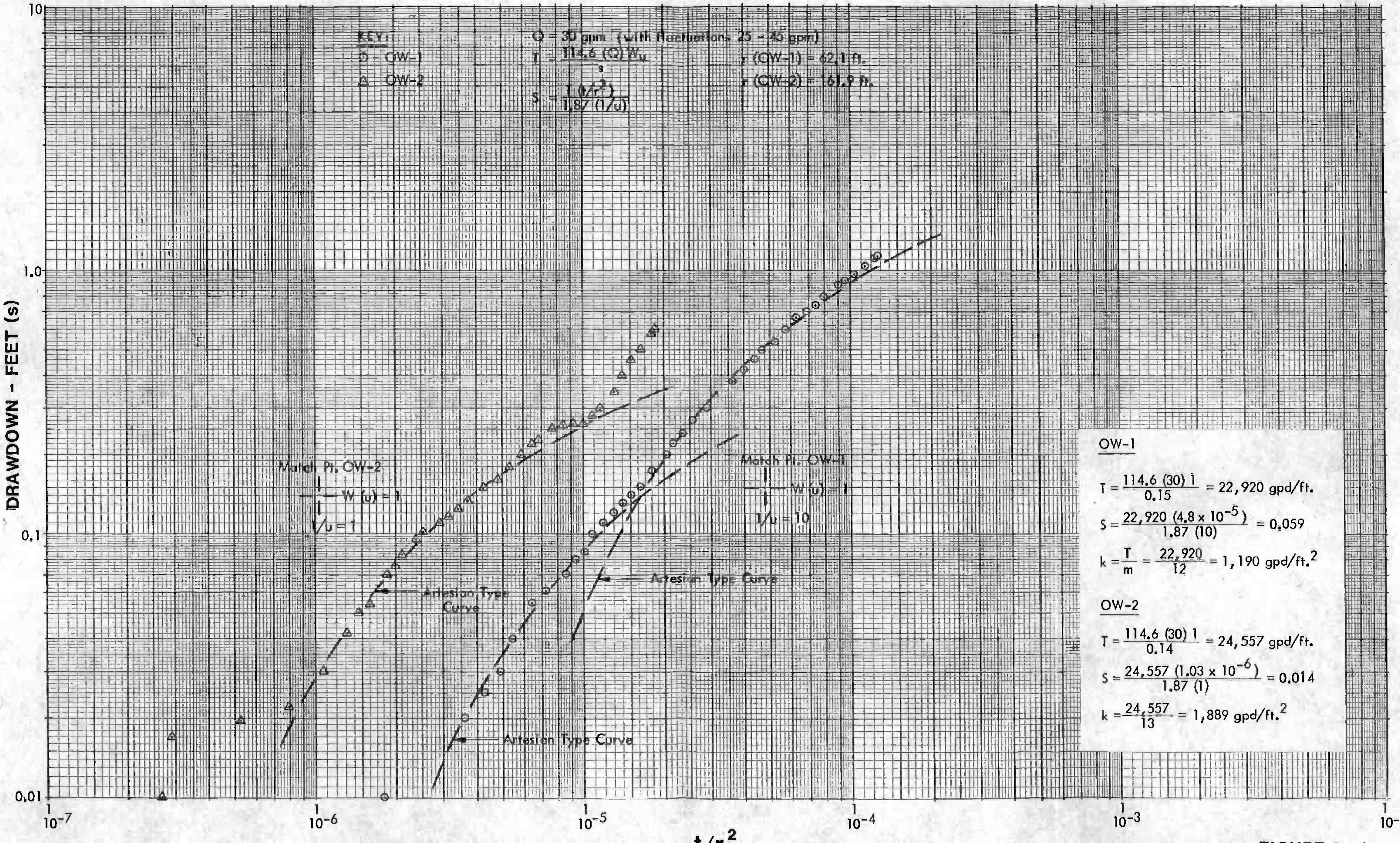
The mean transmissivity from the above summary is approximately 24,000 gpd/ft^2 and the mean hydraulic conductivity is about 1,900 gpd/ft^2 (about 8.5 x 10^{-2} cm/sec). Storativities are relatively high for initial responses suggesting unconfined conditions. As these deposits are dewatered, a specific yield value will apply that is considerably higher than the computed values of storativity. Specific yields of 0.20 to 0.25 are probably reasonable.

D.5 COMMENTS ON TEST RESULTS

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

The transmissivity data and average hydraulic conductivities appear quite reasonable in spite of delayed responses of OW-1 and less stress on the aquifer than planned. Prior to well development, the anticipated pumping rates were several hundred gallons per minute and observation well spacings were determined on that basis. In retrospect spacings of about 50 and 25 feet would have been better for the 30 gpm pumping rate and the thinner than expected aquifer. Aquifer thickness is expected to be greater near the Universal City Station. It is recommended that the computed average hydraulic conductivity of 1,900 gpd/ft^2 be used. Transmissivity can be estimated by multiplying the hydraulic conductivity times the aquifer thickness (clean sands and gravels). The silts and clays will of course have much lower hydraulic conductivities (by several orders of magnitude).

If limited aquifer thickness prevails at the construction site, well points would appear applicable. If wells are used, regardless of type, the limited available drawdown will require development of efficient wells. This requires well screens with adequate open areas and good well development techniques.



OW-1

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

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

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

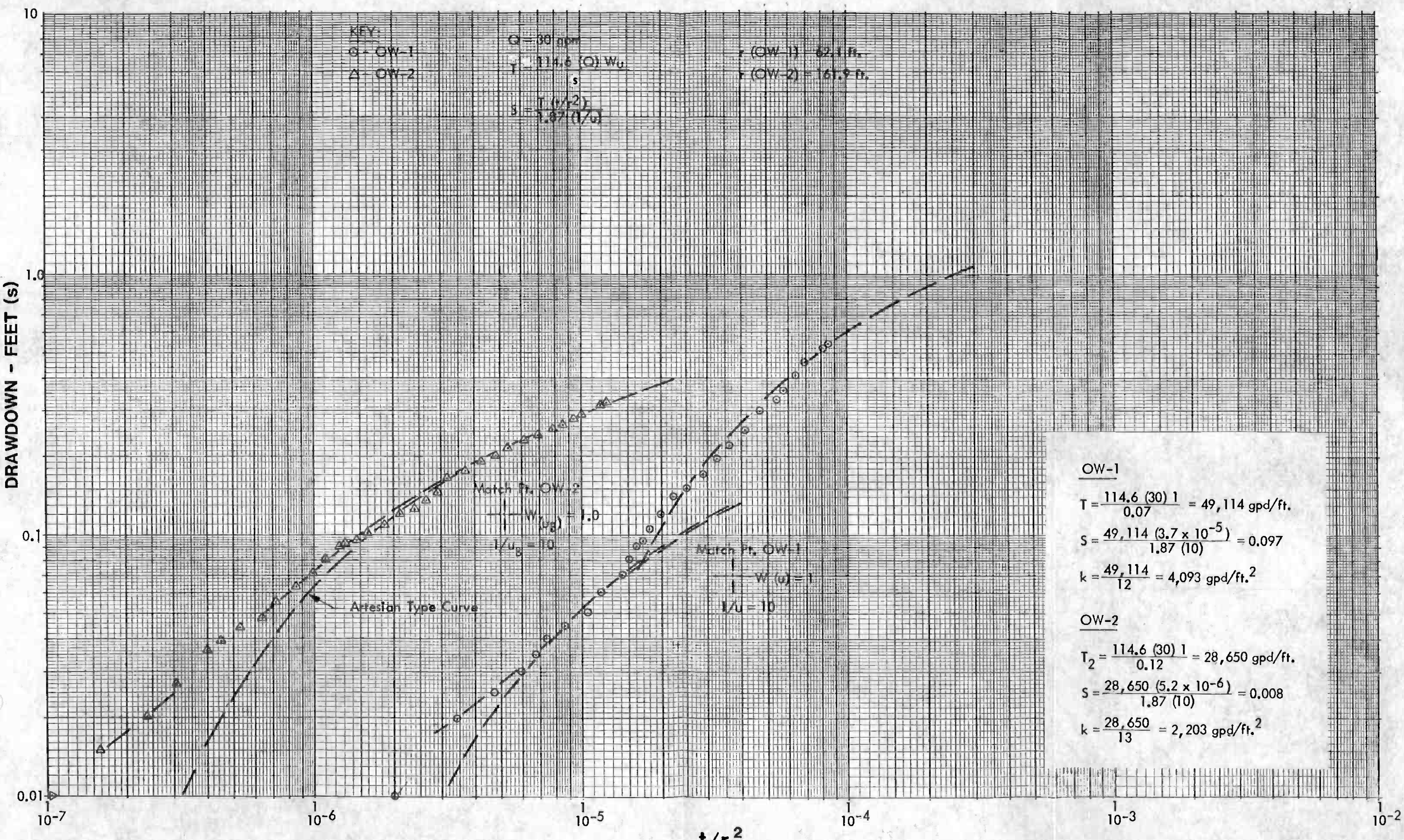
OW-2

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

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

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

FIGURE D-1



OW-1

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

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

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

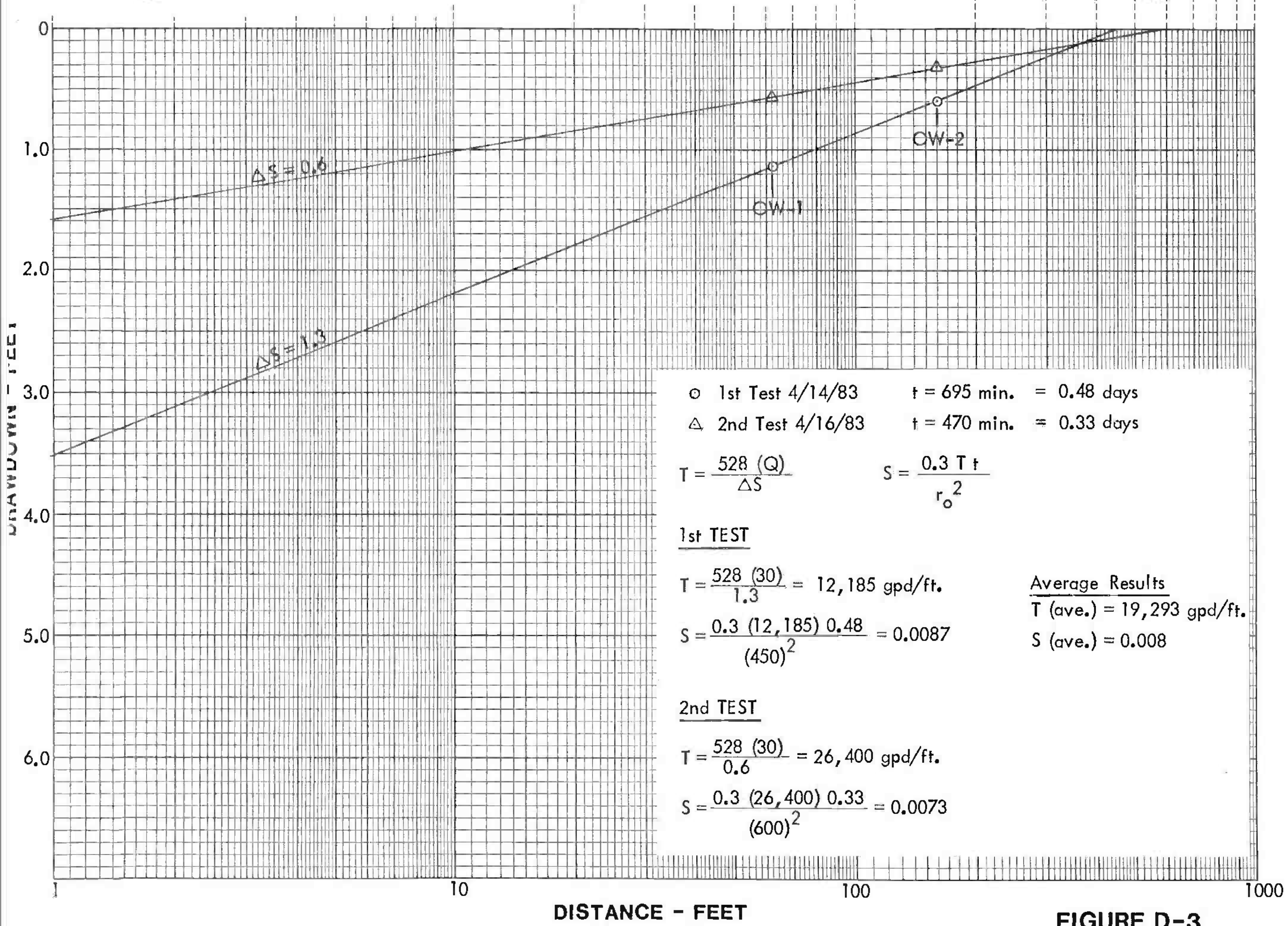
OW-2

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

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

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

FIGURE D-2



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

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

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

1st TEST

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

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

2nd TEST

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

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

Average Results

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

S (ave.) = 0.008

FIGURE D-3

WELL TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Universal City Station

Date of Test 04/14/83

Static Water Level 17.95

Observed By TDH

Radius from Pumped Well 62.1

Average Discharge 30 gpm

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

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

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Universal City Station

Date of Test 04/14/83

Static Water Level 15.61

Observed By TDH

Radius from Pumped Well 161.9

Average Discharge 30 gpm

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

Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
11:45	245	1.70x10^{-1}	6.49x10^{-6}	15.829	0.219	
12:00	260	1.81x10^{-1}	6.91x10^{-6}	15.836	0.226	
12:30	290	2.01x10^{-1}	7.67x10^{-6}	15.860	0.250	
1:00	320	2.22x10^{-1}	8.47x10^{-6}	15.868	0.258	
1:30	350	2.43x10^{-1}	9.27x10^{-6}	15.871	0.261	
2:00	380	2.64x10^{-1}	1.01x10^{-5}	15.871	0.261	
2:30	410	2.85x10^{-1}	1.09x10^{-5}	15.889	0.279	
3:00	440	3.06x10^{-1}	1.17x10^{-5}	15.911	0.301	
4:00	500	3.47x10^{-1}	1.32x10^{-5}	15.956	0.346	
4:30	530	3.68x10^{-1}	1.40x10^{-5}	16.010	0.400	
5:15	575	3.99x10^{-1}	1.52x10^{-5}	16.070	0.460	
6:00	620	4.31x10^{-1}	1.64x10^{-5}	16.120	0.510	
7:00	680	4.72x10^{-1}	1.80x10^{-5}	16.190	0.580	
7.15	695	4.83x10^{-1}	1.84x10^{-5}	16.210	0.600	

AQUIFER TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Universal City Station

Date of Test 04/16/83

Static Water Level 18.04

Observed By TDH

Radius from Pumped Well 62.1

Average Discharge 30 gpm

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

[illegible] 1 Page [illegible] of 2

Time	t min.	t days	t/r^2	Water level feet	Drawdown, s feet	Remarks
1:00	260	$1.81x10^{-1}$	$4.69x10^{-5}$	18.340	0.300	
1:30	290	$2.01x10^{-1}$	$5.21x10^{-5}$	18.370	0.330	
2:00	320	$2.22x10^{-1}$	$5.76x10^{-5}$	18.400	0.360	
2:35	355	$2.47x10^{-1}$	$6.40x10^{-5}$	18.450	0.410	
3:00	380	$2.64x10^{-1}$	$6.85x10^{-5}$	18.490	0.450	
4:05	445	$3.09x10^{-1}$	$8.01x10^{-5}$	18.560	0.520	
4:30	470	$3.26x10^{-1}$	$8.45x10^{-5}$	18.580	0.540	

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Universal City Station

Date of Test 04/16/83

Static Water Level 15.52

Observed By TDH

Radius from Pumped Well 161.9

Average Discharge 30 gpm

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

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

Appendix E

Water Quality Analysis

APPENDIX E WATER QUALITY ANALYSIS

E.1 RESULTS

Water samples were taken from Borings CEG-35, CEG-36, CEG-37, CEG-38 and 35-B. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for future tunneling activities. The chemical constituents test results are attached.

E.2 FIELD PROGRAM

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

Converse Ward Davis Dixon

Lab No. P81-02-142-7

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

Sample labeled: HOLE 35-1", 175'

Conductivity: 4,640 μ mhos/cm

Turbidity: NTU

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

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

Converse Ward Davis Dixon

Lab No. P81-02-123-4

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

Sample labeled: HOLE 36

Conductivity: 1,170 μ mhos/cm

Turbidity: NTU

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

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	65	3.24
Magnesium, Mg	33	2.71
Sodium, Na	125	5.44
Potassium, K	5.2	0.13
		Total 11.52
Anions determined:		
Bicarbonate, as HCO_3	286	4.69
Chloride, Cl	66	1.87
Sulfate, SO_4	253	5.27
Fluoride, F	0.3	0.02
Nitrate, as N	2.3	0.16
		Total 12.01
Carbon dioxide, CO_2, Calc.	10	
Hardness, as $CaCO_3$	298	
Silica, SiO_2	32	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.28	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	732	

Converse Ward Davis Dixon

Lab No. P81-02-123-1

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

Sample labeled: HOLE 37

Conductivity: 1,220 µ mhos/cm

Turbidity: NTU

pH 7.0 @ 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	132	6.60
Magnesium, Mg	34	2.80
Sodium, Na	100	4.35
Potassium, K	5.8	0.15
		Total 13.90
Anions determined:		
Bicarbonate, as HCO_3	192	3.15
Chloride, Cl	49	1.39
Sulfate, SO_4	418	8.71
Fluoride, F	0.5	0.03
Nitrate, as N	7.1	0.51
		Total 13.79
Carbon dioxide, CO_2, Calc.	28	
Hardness, as $CaCO_3$	470	
Silica, SiO_2	25	
Iron, Fe	0.02	
Manganese, Mn	0.10	
Boron, B	0.56	
Total Dissolved Minerals, (by addition: $HCO_3 \to CO_3$)	877	

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	

GENERAL MINERAL ANALYSIS*

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

Log No. P83-10-130

Date Sampled Varies
Date Received 10-19-83
Date Reported 11-29-83

Page 2 of 2

Converse Consultants

Reported To: Buzz Spellman

cc.

Laboratory Director

Sample Description BH 35B Lankershim/Moorpark St. A430

Anions	Miligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligram per liter
Nitrate Nitrogen (as NO_3)	58	0.94	Hydroxide Alkalinity (as $CaCO_3$)	-0-	pHs @ 60°F	6.9
Chloride	36	1.01	Carbonate Alkalinity (as $CaCO_3$)	-0-	pHs @ 140°F	6.2
Sulfate (as SO_4)	160	3.33	Bicarbonate Alkalinity (as $CaCO_3$)	390	Langelier Index @ 60°F	0.8
Bicarbonate (as HCO_3)	480	7.81	Calcium Hardness (as $CaCO_3$)	310	Langelier Index @ 140°F	1.5
Carbonate (as CO_3)	-0-	-0-	Magnesium Hardness (as $CaCO_3$)	120		
Total Milliequivalents per Liter		13.09	Total Hardness (as $CaCO_3$)	430		
Cations	Milligrams per liter	Milliequiv. per liter	Iron	< 0.09		
Sodium	86	3.73	Manganese	< 0.04		
Potassium	3.9	0.09	Copper	< 0.07		
Calcium	120	6.18	Zinc	< 0.015		
Magnesium	30	2.47	Foaming Agents (MBAS)	< 0.1		
Total Milliequivalents per Liter		12.47	Dissolved Residue, Evaporated @ 180°C	760		
			Specific Conductance, micromhos @ 25°C	1100	pH	7.7

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

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Appendix F

Technical Considerations

APPENDIX F TECHNICAL CONSIDERATIONS

F.1 SHORING PRACTICES IN THE LOS ANGELES AREA

F.1.1 General

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

F.1.2 Atlantic Richfield Project (Nelson, 1973)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

F.1.5 Design Lateral Load Practices

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

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

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

TABLE F-1

SHORING LOADS IN LOS ANGELES AREA

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

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

Note:

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

F.2 SEISMICALLY INDUCED EARTH PRESSURES

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

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

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

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

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

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

Where:

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

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

γ = unit weight of soil

ϕ = angle of internal friction of soil

i = angle of soil slope to horizontal

β = angle of wall slope to vertical

k_h = horizontal earthquake coefficient

K_v = vertical earthquake coefficient

δ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes,

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

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

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

Where:

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

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

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

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

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

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

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

Appendix G

Earthwork Recommendations

APPENDIX G EARTHWORK RECOMMENDATIONS

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

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

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

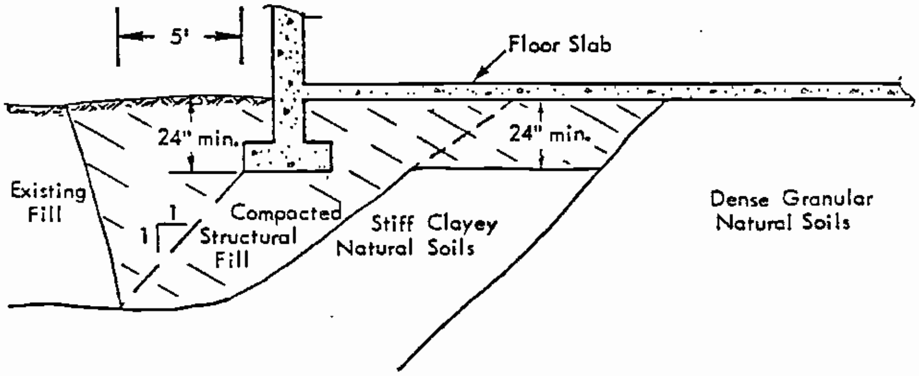
° Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed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

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

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

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

FOUNDATION/SUBGRADE PREPARATION



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

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

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

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

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

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

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

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

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

Appendix H

Geotechnical Report Refrences

APPENDIX H GEOTECHNICAL REPORT REFERENCES

REPORT No.	REPORT DATE	LOCATION	CONSULTANT
43	05/82	Cetty Plaza - southeast corner Lankershim Boulevard and Hollywood Freeway	Woodward-Clyde
44	07/27/46	Universal Pictures, Inc. - Sound Stage C	L.T. Evans
45	09/29/61	Revue Studios - Lankershim Boulevard	L.T. Evans
46	10/27/65	Tower No. 2, Universal City Studios - Lankershim Boulevard	L.T. Evans
47	08/06/74	Universal City Studios - 80 Lankershim Boulevard	L.T. Evans
48	06/03/76	Universal City Studios - 70 Lankershim Boulevard Office Building and Parking Structure	L.T. Evans
49	12/18/50	Southeast corner, Lankershim Boulevard and Magnolia Avenue	L.T. Evans