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Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A220

BY

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

MARCH 1984

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March 21, 1984

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

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A220 prepared in accordance with our Contract No. 503 agreement dated September 30, 1984 between Converse Consultants, Inc. and Metro Rail Transit Consultants (MRTC). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A220.

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

Respectfully submitted,

Robert M. Pride, Senior Vice President
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RMP:i

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

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

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigations and engineering analyses for the A220 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A220 Design Unit consists of the Wilshire/Normandie and Wilshire/Western Stations and about three miles of tunnel line. (The Wilshire/Crenshaw Station is reported upon in a separate report.) The Stations will be constructed by cut-and-cover methods and extend in depth up to 70 feet below the existing ground surface. The line between the Stations 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 gas and ground water conditions. The report defines the subsurface conditions and provides recommendations for design and construction purposes.

1.1 STATIONS

The subsurface conditions at the station structures consist of 25 to 80 feet of alluvium, primarily silts, clays, clayey sands and silty sands. Underlying the alluvium, the explorations encountered the San Pedro sand and gravel layer varying in thickness between 8 and 20 feet at the Wilshire/Normandie Station but a generally uniform thickness of about 20 feet at the Wilshire/Western Station. The San Pedro sand is in turn underlain by interbedded siltstone, claystone and sandstone of the Puente Formation. Ground water was encountered within the Alluvium at depths of 25 to 42 feet below the existing ground surface at the Wilshire/Normandie Station, and at depths of 15 to 18 feet below the existing ground surface at the Wilshire/Western Station.

Station construction on Wilshire Boulevard will consist of excavations approximately 550 feet long, 60 feet wide, and up to 70 feet deep. The Wilshire/Western Station excavation occurs entirely within alluvial type soils as does the west end of the Wilshire/Normandie Station excavation. The easterly third of the Wilshire/Normandie Station excavation will penetrate the siltstones, claystones and sandstones of the Puente Formation.

Temporary support of the Station excavations 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 below ground water elevations. Lateral pressures and other guidelines for design of temporary support systems are provided in the report.

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

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

1.2 TUNNELS AND CROSS PASSAGE

Subsurface conditions along the A220 tunnel alignment are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavating equipment. The majority of the tunnel alignment will pass through horizons of differing alluvium except at the east end of A220 where the tunnels pass through the Puente siltstones, claystones and sandstones and bedrock-alluvial mixed face tunnelling conditions before entering fully alluvial soil tunnelling conditions to the west. The invert of the tunnels will penetrate the San Pedro Sands, underlying the alluvium, for a significant length of the alignment. Ground water levels lie above the crown of the tunnel the entire length. Therefore, some flowing ground conditions could be encountered at the face, and the potential for blow-outs at the invert should be anticipated. It is, therefore, anticipated that construction shield tunnelling methods will require means for the utilization of fore polling and/or breast boarding techniques to maintain stability of the face. In addition, surface and/or local subsurface dewatering measures will be required to control seepage inflows and to provide for the stability of the soils at the face and invert of the tunnels along certain portions of the tunnel alignment.

Design Unit A220 is considered potentially gassy to gassy per the classification contained in Tunnel Safety Orders issued by the California Division of Industrial Safety and adopted from California Administrative Code, Title 8, page 684.18.

The cross passage between tunnels near Station 436± will encounter saturated, interlayered horizons of cohesive and cohesionless-like soils. The cross passage 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.

At approximately Station 337+00, the crown of the AR tunnel line is anticipated to pass approximately 5 feet below the footings of the southeast corner of the Equitable Life Assurance Company parking structure. The evaluation of the underpinning requirements and the behavior of the tunnel and footings under static and earthquake loading conditions to assure the long-term integrity and stability of the structures will be carried out by others. Similar analyses will be required for the buildings under which the tunnels pass at approximately Stations 324+00 and 332+00.

1.4 SEISMIC CONSIDERATIONS

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

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD report entitled "Guidelines for Seismic Design of Underground Structures" dated March 1984. Seismological conditions which may impact the project and the operating and maximum design earthquakes which may be anticipated in the Los Angeles area are described in the SCRTD report entitled "Seismological Investigations and Design Criteria" dated May, 1983. The 1984 report complements and supplements the 1983 report. Site specific static and dynamic properties for materials in design unit A220 are given in the report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A220. The unit consists of Wilshire/Normandie, Wilshire/Western Stations, and about three miles of subsurface track line proceeding east to west from the west end of the Wilshire/Vermont Station to the east end of the Wilshire/LaBrea Station. The Wilshire/Crenshaw Station, with a double crossover ahead of the Station, and a mid-line ventilation station near the Mullen Avenue intersection are not included in this Design Unit. 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 included stations 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 A220.

- "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 A195", prepared by Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in October, 1983. This report presents our results of the findings for the Wilshire/Vermont 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.

- "Rapid Transit System Backbone Route", Volume IV, Book 1, 2 and 3, prepared by Kaiser Engineers, June, 1962 for the Los Angeles Metropolitan Transit Authority. This report presents the results of a Test Boring Program for the Wilshire Corridor and logs of borings.

The design concepts discussed in this report are based on the "Final Report for the Development of Milestone 10, CBD to North Hollywood Line Plans, Sheets 11 to 43, dated September 1983; and Preliminary Site Plans, Plans and Sections for Wilshire/Normandie and Wilshire/Western Stations, Sheets 44 to 58, dated February, 1983.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

3.1 GENERAL

The existing ground surface elevations along the alignment vary between approximately 230 feet on the east and 195 feet on the west. Local variations from this inferred plane surface occur at Stations 341+00, 387+00, 412+00, 433+00 and 460+00 in the form of broad swales the widths of which may extend over several blocks. Such depressions infer the location of former north-south drainage courses which incised the old alluvium and which are now infilled with young alluvial deposits or man-made fill having a comparatively moderate thickness. Such courses are now marked by development in the form of streets and structures.

The easterly 2000 feet of the alignment lies approximately midway between Wilshire Boulevard and Sixth Street (Stations 318+00 to 340+00), where the tunnels pass beneath buildings of moderate size and height. West of Station 340+00, the design unit alignment follows Wilshire Boulevard. The Wilshire corridor is highly developed on both sides with low, medium and high rise commercial buildings. Several of the buildings have been designated as historic landmarks.

All thoroughfares are paved and underlain by a variety of sensitive utilities and drainage facilities.

The construction features about three miles of twin bore tunnels, between Station locations, having an outside diameter of approximately 19 feet. The minimum depth of cover is approximately 25 feet, and the maximum depth of cover approaches 60 feet. Three Station structures are located at or near Normandie, Western and Crenshaw Avenue intersections. The geotechnical features of the latter Station are discussed in a separate report. The depths to Station structure inverts are approximately 55 and 65 feet at the Western and Normandie Avenue Stations, respectively.

A mid-tunnel vent structure and cross passage is located between Stations 436+26 and 437+56 of the A220 Design Unit alignment.

3.2 WILSHIRE/NORMANDIE STATION SITE

The Wilshire/Normandie Station site will be located beneath Wilshire Boulevard between Ardmore and Normandie streets. A number of high-rise office buildings are located along Wilshire near the station location. The Wilshire Hyatt Hotel is immediately adjacent to the station, and the Ambassador Hotel is one block away. Residential areas are to the north and south of Wilshire. The existing ground surface along Wilshire Boulevard varies from Elevation 226 feet at Ardmore Avenue to Elevation 220 feet at Normandie Avenue.

The Wilshire/Normandie Station will be a reinforced concrete structure about 550 feet long and 60 feet wide (outside wall dimensions). The station has been planned with a mezzanine, and an entrance located on Irolo Street. Ancillary space is proposed at each end of the station. The top of rail varies from about Elevation 167 feet at the east end to about Elevation 168 feet at the west end of the station platform. Assuming the station will be supported on a 4- to 6-foot thick concrete mat, the station area will require an excavation to about Elevation 161 feet. This is approximately 60 feet below the existing grade at the east end of the station, and 65 feet below the existing grade at the west end of the station. After the station is constructed, approximately 3 to 4 feet of fill will be placed above the station end areas, and between 19 and 24 feet of fill will be placed above the majority of the station box. Design loads for this Station structure were not available at the time of this report.

3.3 WILSHIRE/WESTERN STATION SITE

The Wilshire/Western Station site will be located between Manhattan Place and Oxford Street. This area is on the western edge of a high-rise segment of the Wilshire Corridor office core. The remainder of the surrounding area is in residential use. All four corners of the intersection of Wilshire and Western are developed: the historic landmark Wiltern Theater is located on the southeast corner and is undergoing renovation, a Union Bank building is on the southwest corner, the Pierce National Life Insurance Building is on the northwest corner, and a one-story Thrifty Drug Store is on the northeast corner adjacent to the McKinley Building. Existing ground surface along Wilshire Boulevard at the station site is approximately Elevation 200 feet.

The station has been planned with a mezzanine centered over the length of the platform. The northeast corner of the intersection of Western and Wilshire is selected as the entry area to this station. A bus-rail transfer and layover lane is planned north of Wilshire between Western Avenue and Oxford Street. Ancillary space will be located at each end of the station. A traction power substation will be located at grade adjacent to the station entrance.

The Wilshire/Western Station also will be a reinforced concrete structure about 550 feet long and 60 feet wide. The top of rail varies from about Elevation 153 feet at the east end to about Elevation 151 feet at the west end of the station platform. Assuming the station will be supported on a 4- to 6-foot thick concrete mat, the station area will require an excavation to about Elevation 146 feet. This is approximately 55 feet below the existing grade. After the station is constructed, roughly 8 feet of fill will be placed above the station box structure. Design loads for this Station structure were not available at the time of this report.

3.4 TUNNEL ALIGNMENT

As shown on Drawings 2, 3, 4 and 5, the tunnel line in Design Unit A220 is about three miles long, starting at approximately Station 319+16 and ending at approximately Station 474+47. The tunnel continues in an east-west direction from the west end of the Vermont Station and enters into a set of reversing curves to reach Wilshire Boulevard at Alexandria Avenue. From that point, the tunnel continues west directly under Wilshire Boulevard until it reaches the east end of the Wilshire/LaBrea Station.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The information presented in this report is based primarily on the field and laboratory investigations performed in 1981 and 1983. This information was derived from field reconnaissance, borings, geologic reports and maps, ground water measurements, field 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 A220 investigation, 16 borings were drilled along the alignment and at the station sites: three along the alignment, six at the Wilshire/Normandie Station, and seven at the Wilshire/Western Station. The alignment borings are numbered 13A, 13-7 and 13-8. The Wilshire/Normandie Station borings are numbered 14-1 to 14-5. The Wilshire/Western Station borings are numbered 15-1 to 15-5 and 15-A. Borings CEG-13 through CEG-17 which were drilled in 1981 are also included. The locations of the borings are shown on Drawings 2 and 4, and the logs of the borings from the 1981 and 1983 investigations are provided in Appendix A. Ground water observation wells were installed in Borings 14-1, 14-3, 15-1 and 15-3. Section 5.4 presents a summary of ground water level measurements in these wells and others near A220.

Information pertinent to the tunnel alignment for this design unit was also obtained from borings for the Wilshire/Crenshaw Station (Design Unit A240), a vent structure, and the Wilshire/LaBrea Station (Design Unit A245). These borings are identified as 16-1 through 16-6, 16A, 16-B, 17-A, 17-B, 18-2 through 18-7 drilled in 1983. Logs of these borings are also included in this report, and their locations and graphical sections are presented on "Location of Borings and Geologic Sections", Drawings 3, 4 and 5.

In 1962, Kaiser Engineers drilled 30 borings within the Design Unit A220 tunnel alignment section: Borings 44 to 74, inclusive. These borings were spaced about 500 feet apart and ranged from 50 to 80 feet deep at the locations shown on Drawings 2, 3, 4 and 5. Of the 30 Kaiser borings, 26 (Borings 55 through 70) are on the present Metro Rail Project alignment and were used to interpret the depth of soil overlying the bedrock, but they were not used to evaluate ground water conditions. The Kaiser Boring Logs can be examined at the Southern California Rapid Transit District office in Vol. 4, Books 2 and 3, entitled "Test Boring Program" prepared for the Los Angeles Metropolitan Transit Authority, June 1962.

Another source of boring information is the U.S. Geological Survey paper, "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF-866, 1977).

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

4.3 GEOPHYSICAL MEASUREMENTS

Downhole and crosshole compression and shear wave velocity surveys were performed in Borings CEG-14 and CEG-15 which were drilled during the initial 1981 investigation. The CEG-14 boring was drilled about 200 feet east of the Wilshire/Normandie Station, and CEG-15 was drilled about 120 east of the Wilshire Western Station (see Drawings 2 and 3). Appendix B summarizes the field survey procedures as well as the results of the velocity measurements.

4.4 OIL AND GAS ANALYSES

A sulfurous odor was noted in Borings 14-2, 14-3 and 14-4 at the Wilshire/Normandie Station site. The odor was noted at about the time the drilling encountered the Puente Formation.

The Los Angeles City Oil Field is located about 3,000 feet north of the Wilshire/Normandie Station, and the Western Avenue Oil Field is located about 4,000 feet north of the Wilshire/Western Station. The oil fields contain shallow accumulations of petroleum, surface seeps and more than 1250 wells. As discussed in the 1981 Geotechnical Report, these oil fields were discovered in the 1890's, and subsequently produced over a million barrels of oil per year for a few years. No evidence was found to indicate any regional subsidence has occurred due to the oil fields. Most of the wells which were drilled prior to 1900 were not surveyed or accurately located, and the ground surface has since been developed.

4.5 WATER QUALITY ANALYSES

Chemical analyses were performed and selected parameters were evaluated for water samples obtained in Borings 14, 16A, 17, 17A and 17B. The chemical analyses and results of these tests are presented in Appendix D.

4.6 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, 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.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

During the field programs conducted for this and the 1981 investigations, the contact between the Old and Young Alluvium was difficult to identify since the soils in these two deposits can be very similar. While the Young and Old Alluvium may be geologically different, our interpretation of the field and laboratory test data suggests that they do not differ significantly from an engineering standpoint. For the purposes of this report, Young and Old Alluvium have not been differentiated and are simply referred to as Alluvium. Generalized geologic interpretations of subsurface conditions along the proposed route are presented on Drawings 2, 3, 4, and 5.

5.1 WILSHIRE/NORMANDIE STATION

Drawings 2 and 7 show generalized subsurface cross sections through the proposed Wilshire/Normandie Station. Approximately one to 2-1/2 feet of fill overlie alluvial silty clays and silty sand. Within the station limits, an upper layer of granular Alluvium up to 6 feet in depth was encountered. Beneath this fine-grained Alluvium extends to depths varying between 35 feet at the east end and 75 feet at the west end of the station. The Alluvium is underlain by very dense San Pedro Sand. The thickness of this fine- to medium-grained sand varies from 10 to 30 feet between the east end and the west end of the station. Underlying the San Pedro Sand, the bedrock surface at the Wilshire/Normandie Station slopes gently downward from east to west.

Specific descriptions of the soil and rock materials encountered in the borings at the station site include the following:

- <u>Fill</u>: At Borings 14-3 and 14-5, approximately 1.5 feet of clayey sand and sandy clay fill were encountered. The fill in these two borings was dense and stiff. Generally one foot of asphaltic concrete and concrete pavement section existed on Wilshire Boulevard.

- <u>Alluvium</u>: A relatively thin layer (less than 6 feet) of granular Alluvium was encountered beneath the fill. The materials consisted of medium dense to dense silty sand and clayey sand. Two Standard Penetration Tests in this material showed driving resistance of 25 and 27 blows per foot. Based on boring data, the remainder of the Alluvium consisted of clays, clayey silts, sandy clays and silty and clayey sands, primarily very stiff and dense. The borings at the station site encountered some 35 to 75 feet of this unit overlying the sloping San Pedro sand unit. The sampling resistance, unit weight, moisture content, and laboratory test data performed in this unit showed that the clays and silts were stiff to very stiff with low compressibility, and that the sands were dense.

- <u>San Pedro Sand</u>: The borings encountered between 10 and 30 feet of a very dense fine sand and silty sand identified as the San Pedro Formation. The unit essentially consists of a uniform fine sand with less than 5% silt, and with occasional gravelly lenses. Based on the laboratory tests and field Standard Penetration Tests, this sand layer was very dense and low in compressibility.

- Puente Formation: The bedrock underlying the site consisted of thinly interbedded claystone and clayey siltstone of the Puente Formation. The top of the bedrock sloped gently downward toward the west. Bedding, where observed in samples, was between about 20° and 50°. Strike of the bedding could not be determined from the samples. However, the regional trend would be roughly a east-west strike, and a south dip. Occasional thin zones of localized hard cementation were encountered. However, these hard zones are estimated to comprise a small percentage of the Puente Formation.

5.2 WILSHIRE/WESTERN STATION

Drawings 3 and 8 show generalized subsurface cross sections through the proposed Wilshire/Western Station. The subsurface profile at the Station site consists of approximately 2 to 8-1/2 feet of fill over fine-grained Alluvium extending to depths of approximately 60 to 76 feet. Beneath this Alluvium, a layer of very dense San Pedro Sand was encountered. The thickness of this sand layer varied between 20 and 35 feet. At the boring locations within the station limits, a gravelly sand and sandy gravel course of approximately 5 to 8 feet was encountered toward the bottom of the San Pedro Sand layer. The bedrock surface at this Station site sloped slightly downward from west to east.

Specific descriptions of the soil and rock materials encountered in the borings at the site include the following:

- Fill: At Borings 15-1 through 15-4, between 2 and 8-1/2 feet of silty sand and sandy clay fill was encountered beneath the one foot thick pavement section. Boring 15-5 which is located about 275 feet east of the station limits encountered approximately 16-1/2 feet of fill materials. Test results within the fill showed that the materials are dense and stiff.

- Alluvium: The Alluvium consists of silty sand, sandy silt, sandy clay and silty clay. The consistency of this unit showed that the interbedded materials were very stiff and dense to very dense. The borings within the station site showed that the thickness of this layer varied from 60 to 67 feet between the west end and the east end of the station. Detailed descriptions of the unit are shown on Drawing 9.

- San Pedro Sand: The borings encountered between 20 and 25 feet of a uniform fine sand and gravelly sand of the San Pedro Unit. In Boring 15-1 and 15-3 approximately 5 to 8 feet of the lower portion of this unit consisted of coarse-grained sandy gravel. Field and laboratory test results showed that the San Pedro unit is generally very dense and relatively incompressible.

- Bedrock: The bedrock encountered at the Wilshire/Western Station site consists of both thin and thick interbedded siltstone and sandstone of the Puente Formation. The sandstone appears to be weakly cemented. The top of the bedrock slopes very gently toward the east. Dip of the bedding, where observed, was inclined at approximately 30° and 35°. Strike of the bedding could not be determined from the samples. However, the regional trend would be roughly east-west strike and south dip.

5.3 TUNNEL ALIGNMENT

About 15% of the A220 tunnel line between Stations 319 and 345 will occur in weak bedrock of the Puente Formation and approximately 85% of the tunnel will be in Alluvium. There is a distinct possibility that the tunnel invert between the Wilshire/Western and Wilshire/Crenshaw Stations will encounter the San Pedro (sand) Formation.

Mixed-faced tunnel conditions should be anticipated exiting the Wilshire/Vermont Station (Stations 320± to 328) and entering the Wilshire/Normandie Station at about Station 344. A general description of the anticipated geologic units along the tunnel alignment follows:

- Alluvium: Alluvium consists of a mixture of clays, clayey silts, sandy clays, silt and clayey sands. The materials are primarily stiff to very stiff and dense with low compressibility. Large boulders are not anticipated. Ground water occurs at depths ranging from 9 to 40 feet below the existing ground surface. Below the ground water level, the granular alluvium may be expected to flow at the face of the excavation. This is expected due to the higher permeability of the granular soils. Clayey soils are expected to produce only minor water inflow.

- San Pedro Sand: The San Pedro Sand generally consists of a uniform fine sand with less than 5% silt, with occasional gravelly lenses. The San Pedro Sand is very dense in-situ with a low compressibility. In our opinion, the San Pedro Sand should be considered saturated for tunnelling purposes. This is based on the wet flowing nature of the sand as observed in man-sized auger Borings 15-A, 16-A, 16-B and 17-B.

- Puente Formation: Bedrock of the Puente Formation consists of well stratified claystone and siltstone with interbeds of sandstone. The Puente 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 Puente Formation is classified as "soil-like" bedrock or "soft ground" tunneling material. Locally, the Puente Formation contains very hard sandstone beds ranging from less than 1 inch to 3 feet in thickness, with an estimated unconfined compressive strength ranging from 5000 to 15,000 psi. Based on surface outcrops located about one mile east of the Wilshire/Normandie Station, bedding planes strike nearly east-west, with attendant dips of 13° to 40° southward. This corresponds to bedding observed in man-sized auger Boring 11-A near MacArthur Lake; i.e., strike N85°E, dip 33° to 45° south and man-sized auger Boring 13-A with strike N70°E, dip 25°S.

No tar or oil was encountered in the borings in Design Unit A220. However, man-sized auger Boring 17-A, located on the west side of Mullen Street about 200 feet south of Wilshire Boulevard, and opposite tunnel line Station 435, encountered gas under pressure for the depth interval from 38 to 42 feet. The gas detector read 100% Lower Explosive Limit (LEL) immediately after encountering the 38-foot depth and 20% LEL after 1 hour. Gas issued from the bottom of the hole so vigorously that it churned the water and white-colored vapor was visible. Knowing that the San Pedro Sand Formation and the Puente/Fernando Bedrock Formations contain oil and gas, and because Design Unit A220 is located near the Los Angeles City Oil Field and the Salt Lake Oil Field, Design Unit A220 should be considered potentially gassy to gassy. Gas was not detected by the gas meter in man-sized auger Borings 12-A (Station 303±), 13-A (Station 321±), 15-A (Station 374±), 16-B (Station 416±), and 17-B (Station 470±).

The tunnel will pass beneath the southeast corner and the south edge of the Equitable Life Assurance Building. Based on building drawings provided by MRTC, the bottom of the garage wall footing is at about Elevation 164 feet. The crown of the tunnel at this location corresponds to about Elevation 159 feet. Therefore, there is only approximately 5 feet of cover above the crown of the tunnel.

5.4 GROUND WATER

Regionally, ground water has been measured both at shallow depths within the alluvium and at deep levels within the bedrock. The alluvial ground water occurs at depths of about 30 to 40 feet at the Wilshire/Normandie Station, and 15 to 20 feet at the Wilshire/Western Station. Ground water levels within the bedrock at the station sites are estimated to be about 150 feet below the ground surface. For design purposes, it is assumed that the bedrock above the lower ground water level is not submerged.

The following Table 5-1 presents ground water levels and fluctuations measured in piezometers and man-sized auger borings within the limits of A220.

TABLE 5-1
GROUND WATER OBSERVATION WELL DATA

	GROUND WATER ELEVATION*					
	1981	1982	1983			1984
BORING	JAN.	APRIL	OCT.	NOV.	DEC.	MARCH
13A				222**		
14			192	192		
14-1			186	186	186	186
14-3			186		186	187
15-1				182		182
15-3				180		180
16	176	167		173		
16A				173**		
16B				187**		
16-2			176			174
16-5				174	174	174
16-6				175	175	175
17	171	168	destroyed			
17A			187**			
17B			180**			
18-7			180	179	176	179

* Rounded to the nearest foot

** No piezometer installed; water level measured during drilling

It appears that the ground water level varies across the Wilshire/Normandie Station site, ranging from about Elevation 192 feet in CEG-14 (located 200 feet east of the Wilshire/Normandie Station) to about Elevation 186 feet in Boring 14-3 (located just east of Ardmore Avenue). The ground water level varies across the Wilshire/Western Station site, ranging from about Elevation 182 feet in Boring 15-1 (located east of Manhattan Place) to about Elevation 180 feet in Boring 15-3 (located west of Oxford Street). The piezometer data represent a ground water gradient of about 0.017 across the Wilshire/Normandie Station site in the westward direction, and 0.003 across the Wilshire/Western Station in the eastward direction.

A sulfur odor was noted in Borings 14-2, 14-3 and 14-4 at the Wilshire/Normandie Station site. The odor was noted after the borings had encountered the Puente bedrock.

5.5 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.5.1 General

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

5.5.2 Fill

Fill soils encountered at the Wilshire/Normandie and Wilshire/Western Station sites included stiff sandy clays, and dense silty sands and clayey sands. Generally, fill was encountered to relatively shallow depths at the Wilshire/Normandie Station site. Greater fill thickness was encountered at the Wilshire/Western Station site. None of the borings within the station sites encountered building debris. Due to possible variability of old fills however, the presence of undesirable materials or soft/loose zones should be anticipated. Strength tests performed on representative samples of the fill indicate that the fill at the station sites is either stiff and/or dense.

5.5.3 Alluvium

The Alluvium encountered at both station sites consisted of clay, silty clay, sandy clay, clayey sand, silty sand and gravelly sand. Standard Penetration Test (SPT) results, laboratory densities and strength tests indicate that the fine-grained and the coarse-grained alluvium are, respectively, stiff and dense.

Strength tests performed on the alluvial soils included both direct shear and triaxial compression tests. Considering the relative high permeability of the coarse-grained alluvium and the random occurrence and lenticular nature of the fine-grained and the coarse-grained materials, drained (effective) strength parameters are considered appropriate for static design. These parameters are presented in Table 5-2.

5.5.4 San Pedro Formation Sand

At both station sites, a uniform fine sand, gravelly sand and sandy gravel layer of the San Pedro Formation was encountered. SPT results and laboratory densities indicate that this sand unit is very dense. This unit is below the water level.

Recommended moist and saturated densities are presented in Table 5-2. Permeability of the sands is expected to vary somewhat between the fine sand materials (10^{-2} to 10^{-3} cm/sec) and gravelly lenses or layers (5×10^{-2} cm/sec) which may be encountered. The permeability values are estimates based on results of the laboratory tests combined with engineering judgement.

Strength tests performed on the sands included both direct shear and triaxial compression tests. Considering the relatively high permeability of the sands, drained (effective) strength parameters are considered appropriate.

Elastic properties for the sands were based on the laboratory triaxial and consolidation tests combined with published data and engineering judgement. Modulus data on soil samples from this site and similar soil samples from other Design Units were evaluated. The data indicate that the modulus increases linearly with confining pressure. This characteristic is consistent with published data. The modulus value is presented on Table 5-2 in terms of the effective overburden pressure.

5.5.5 Puente Formation Bedrock

The weak Puente Formation claystone and siltstone were considered to be very stiff to hard overconsolidated fine-grained soil for engineering purposes. These materials were encountered below the water level which is within the alluvium and are assumed to be saturated but not submerged.

Due to the nature of the bedrock materials and the various loading conditions, both the drained (effective) and undrained (total) strength parameters were considered in developing design recommendations. Strength parameters presented in Table 5-2 should be considered to be representative of the relatively fresh bedrock encountered about 5 feet below the bedrock surface and were based on interpretation of triaxial, unconfined compression, and direct shear tests combined with our engineering judgement. The total stress data indicate a relatively high undrained friction angle. However, experience and principles of soil mechanics predict that the undrained strength of the bedrock should approach that of a cohesive material.

Bedrock elastic properties were selected based on consideration of field performance data, laboratory test data and published information combined with engineering judgement. For this study, the bedrock material was considered to

TABLE 5-2
MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	FILL	FINE-GRAINED ALLUVIUM	GRANULAR ALLUVIUM	SAN PEDRO SAND	PUENTE[c] BEDROCK
Moist Density Above Ground Water (pcf)	130	130	130	130	120
Saturated Density (pcf)	-	130	130	130	120
Effective Stress Strength					
ø' (degrees)	-	35	35	35	35
c' (psf)	-	0	0	0	0
Total Stress Strength[a]					
ø (degrees)	-	20	-	-	10
c (psf)	-	1000	-	-	4000
Unconfined Compressive Strength (psf)	-	2000	-	-	8000
Permeability (cm/sec)	-	10^{-3} to 10^{-6}	10^{-2} to 10^{-4}	$5x10^{-2}$ to 10^{-3}	10^{-6} to 10^{-7}
Initial Vertical Tangent Modulus (psf)	-	$180 \cdot \sigma_v'$[b]	$300 \cdot \sigma_v'$[b]	$300 \cdot \sigma_v'$[b]	$2x10^6$
Poisson's Ratio (non-saturated)	-	0.40	0.35	0.35	0.35

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

b σ_v' is the effective overburden pressure (psf) (equal to effective density times overburden depth). Moist density should be used to determine σ_v' above the water table and submerged density (saturated density minus water density) should be used for the effective density of soils below the water table.

c For relatively fresh bedrock.

have no significant modulus increase within the range of depth affected by the proposed stations. The apparent variation of modulus values at low confining pressures indicated by the laboratory data may be due to several factors including the effects of sample disturbance and sample expansion after insitu stresses were removed.

Section 6.0

Geotechnical Evaluation and Design Criteria for Stations

6.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA FOR STATIONS

6.1 GENERAL

In general terms, construction of the A220 Stations will involve deep excavations through stiff and dense alluvium to depths varying between 55 and 70 feet below the ground surface. At the east end of the Wilshire/Normandie Station the excavation will penetrate up to about 30 feet of siltstone/claystone bedrock. Construction problems will be similar at both sites. The existence of high ground water levels will require either dewatering or tight shoring for the construction excavations. The permeable San Pedro Sand layer below the alluvium must be dewatered or cut-off to prevent basal heave or blow-out.

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

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

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

The permanent Station structures will, in essence, be concrete boxes supported on and retaining the surrounding soils and/or bedrock. As shown on Drawing 9, the subgrade condition at the Wilshire/Western Station generally will be uniform. However, at the Wilshire/Normandie Station (Drawing 7), the subgrade will vary from bedrock at the east end to alluvium at the west end. Significant differential settlement is expected to occur between the two extreme subgrade conditions at the Wilshire/Normandie Station; however, the subgrade transition is gradual enough (Drawing 7 exaggerates the vertical scale) that estimated angular distortions in the longitudinal direction are small.

The following subsections present our further evaluations and recommendations for design and construction of the A220 Station structures.

6.2 EXCAVATION DEWATERING

6.2.1 General Evaluation

The construction of both the Wilshire/Normandie and Wilshire/Western Stations will require excavations extending 30 to 40 feet below the measured ground water levels and may require areal construction dewatering if tight shoring is not used. As discussed in Section 5.0, the subsurface conditions at both sites generally consist of predominately fine-grained alluvium, overlying the San Pedro Sand Formation which in turn overlies siltstone bedrock. At the Wilshire/Normandie site, the bedrock and San Pedro Formation slope down toward the west and, therefore, the permeable San Pedro Sand strata will be exposed in both the sidewalls and bottom of the excavation (see Drawing 7). At Wilshire/Western site, the bedrock surface and overlying San Pedro Sand strata are relatively flat lying, and the bottom of excavation will be within the fine-grained alluvium about 10 to 15 feet above the San Pedro Sands (see Drawing 9).

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

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

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

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

As an alternative to dewatering, tight shoring such as slurry wall construction penetrating into the bedrock underlying the A220 sites could provide an effective ground water barrier. Chemical grout may also be considered to establish a ground water cut off within the San Pedro Sands in conjunction with a soldier pile system.

6.2.2 Possible Dewatering System

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

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

6.2.3 Criteria for Dewatering Systems

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

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

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

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

6.3 UNDERPINNING

6.3.1 Common Underpinning/Support Methods

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

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

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

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

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

6.3.2 Underpinning Considerations

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

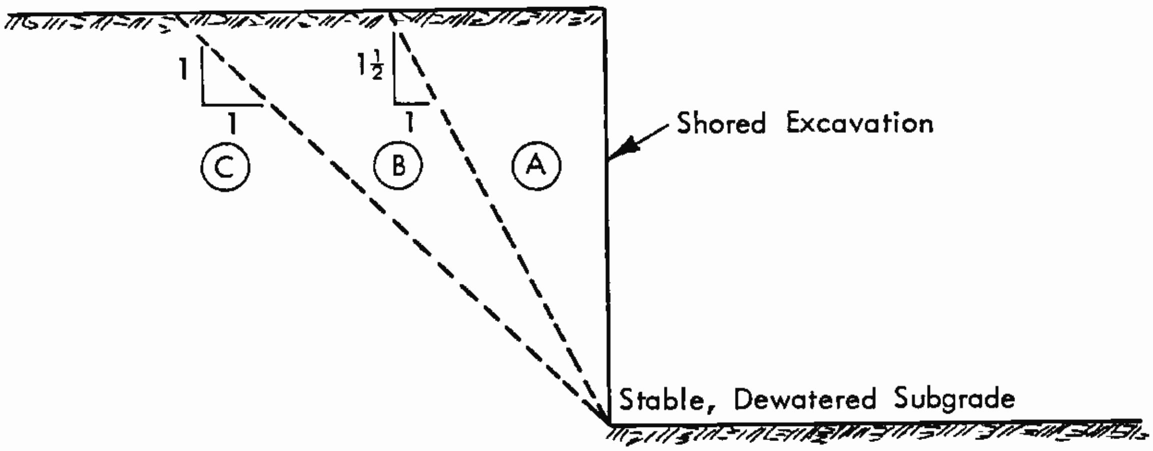
From an engineering standpoint, the need to underpin is evaluated on the basis of expected ground movements and potential for structural damage. Figure 6-1 presents general guidelines for evaluating if a structure may be within the influence zones of the excavation; however, further evaluation of expected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

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

6.3.3 Design Criteria

Figures 6-2 through 6-7 present design criteria for jacked piles and slant drilled piles without dragdown loads. 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, 6-4 and 6-7 present design parameters for underpinning based on the expected subsurface conditions at the Wilshire/Normandie Station. Figures 6-5 and 6-6 present underpinning design data for deep alluvium conditions at the Wilshire/Western Station.

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



NOTES:

1. These guidelines are applicable only for stable ground conditions. Other soil and/or foundation conditions may require further analyses.
2. Settlement due to dewatering also must be considered.
3. For structure foundations bearing in zones A, B, or C, the following guidelines are presented:

ZONE (A) Special Provisions Required for Important Structures:

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

ZONE (B) Generally No Special Provisions Required:

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

ZONE (C) No Special Provisions

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UNDERPINNING GUIDELINES

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Figure No. 6-1

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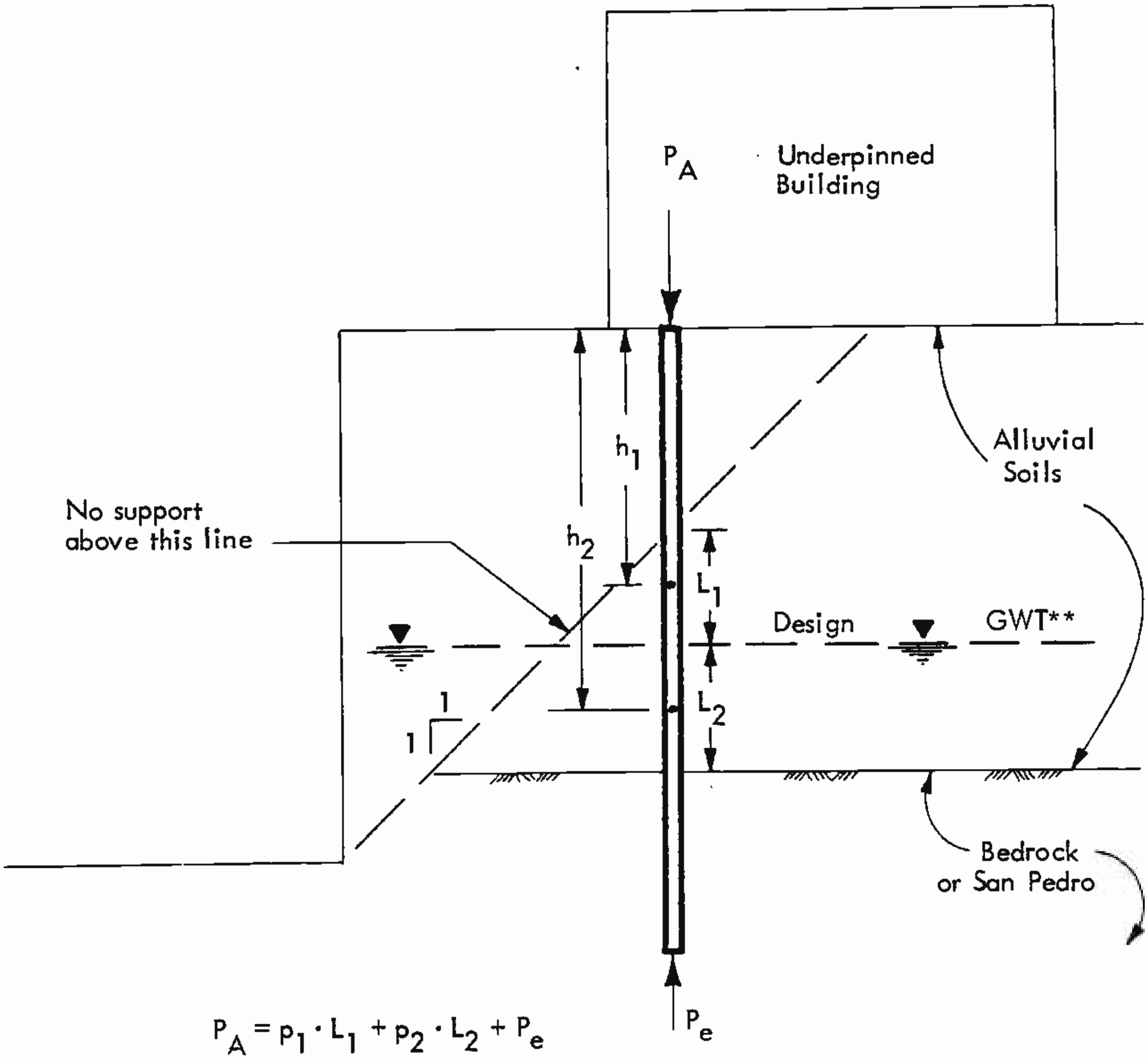

$$P_A = p_1 \cdot L_1 + p_2 \cdot L_2 + P_e$$

WHERE: $*p_1$ = average frictional resistance at h_1

$*p_2$ = average frictional resistance at h_2

$**P_e$ = end resistance

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

** For alluvium use P_e values given on Figure 6-3 through 6-6.

For San Pedro Sands use $P_e = q \times A_e$

where: $q = 80D$ (ksf)

D = pile diameter (ft.)

A_e = pile tip area (ft.^{2})

For Bedrock use P_e values given on Figure 6-7.

UNDERPINNING - DESIGN CAPACITY CRITERIA

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Figure No. 6-2

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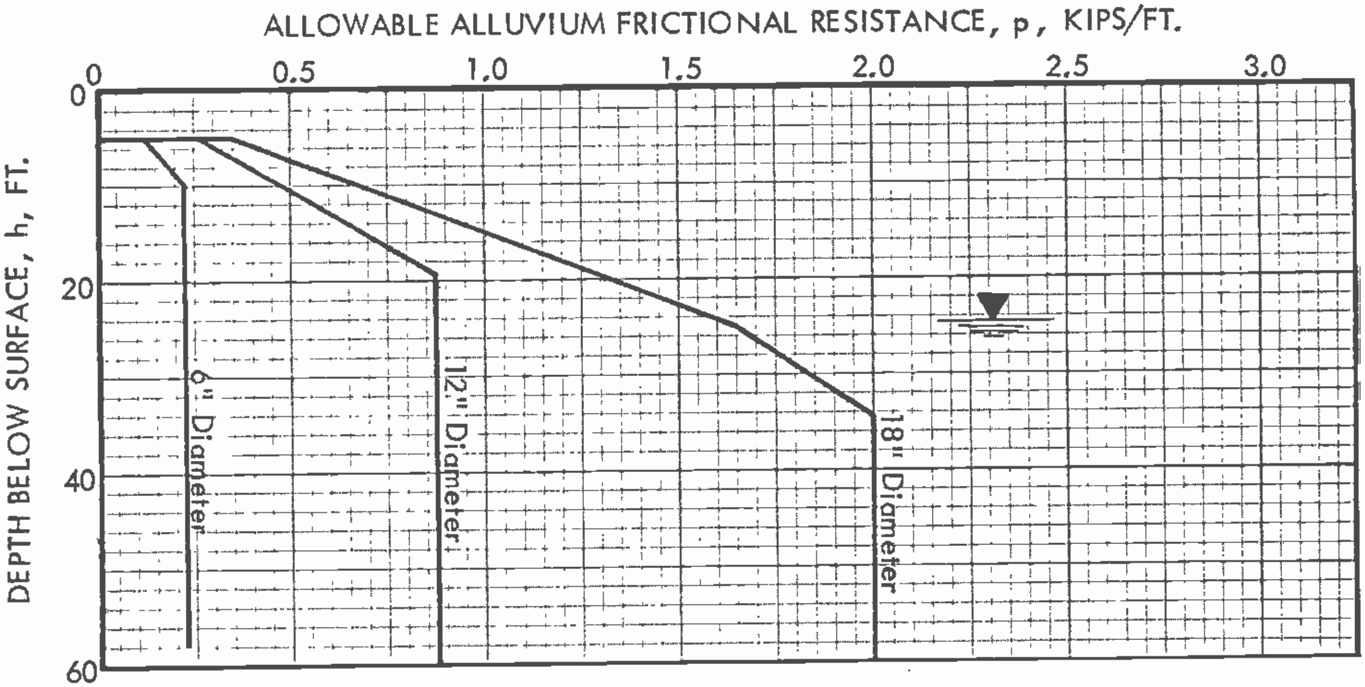

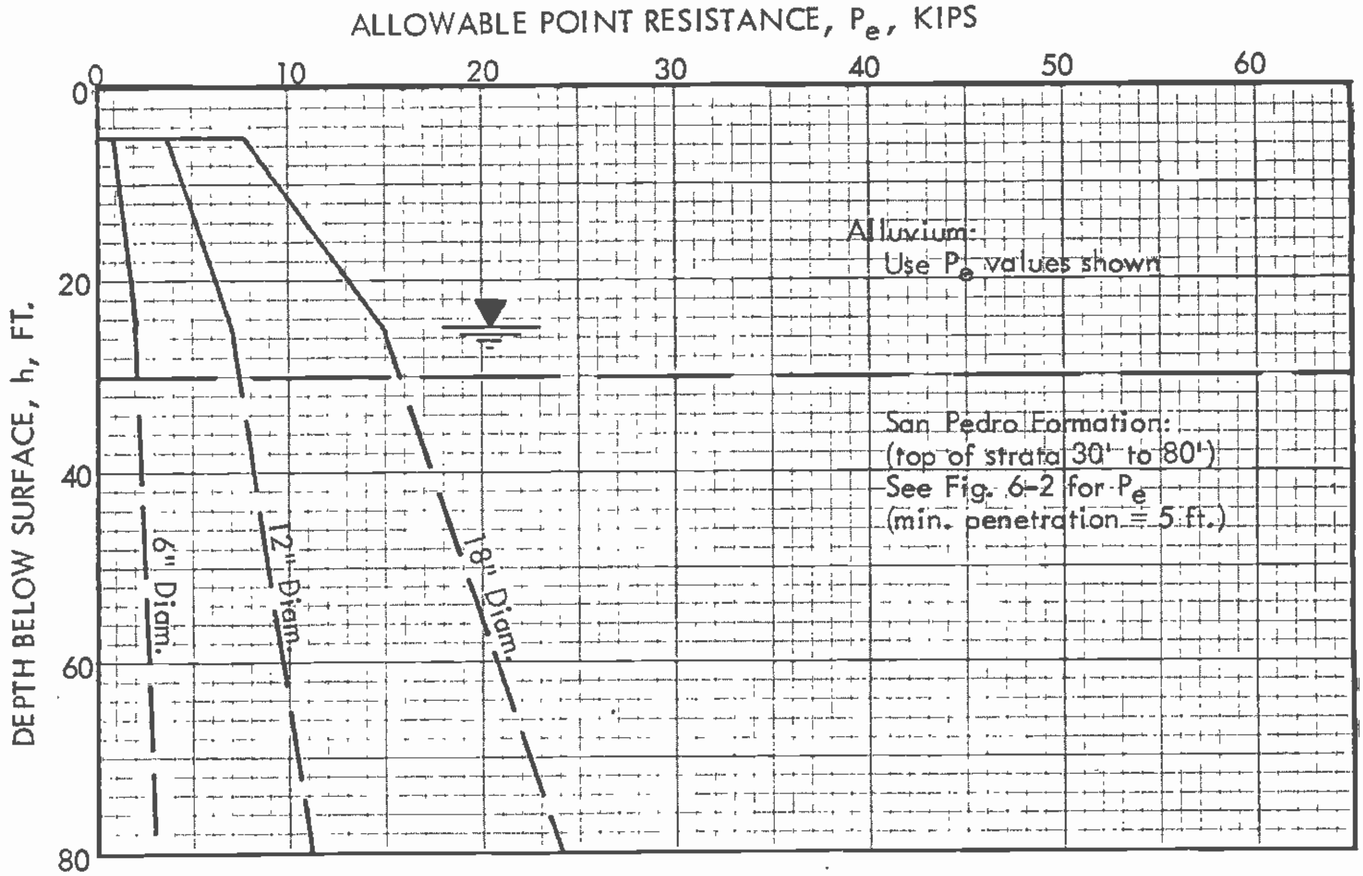

See Figure 6-2 for Determination of Total Capacity.

UNDERPINNING - JACKED PILE DESIGN PARAMETERS (NORMANDIE)

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Figure No. 6-3

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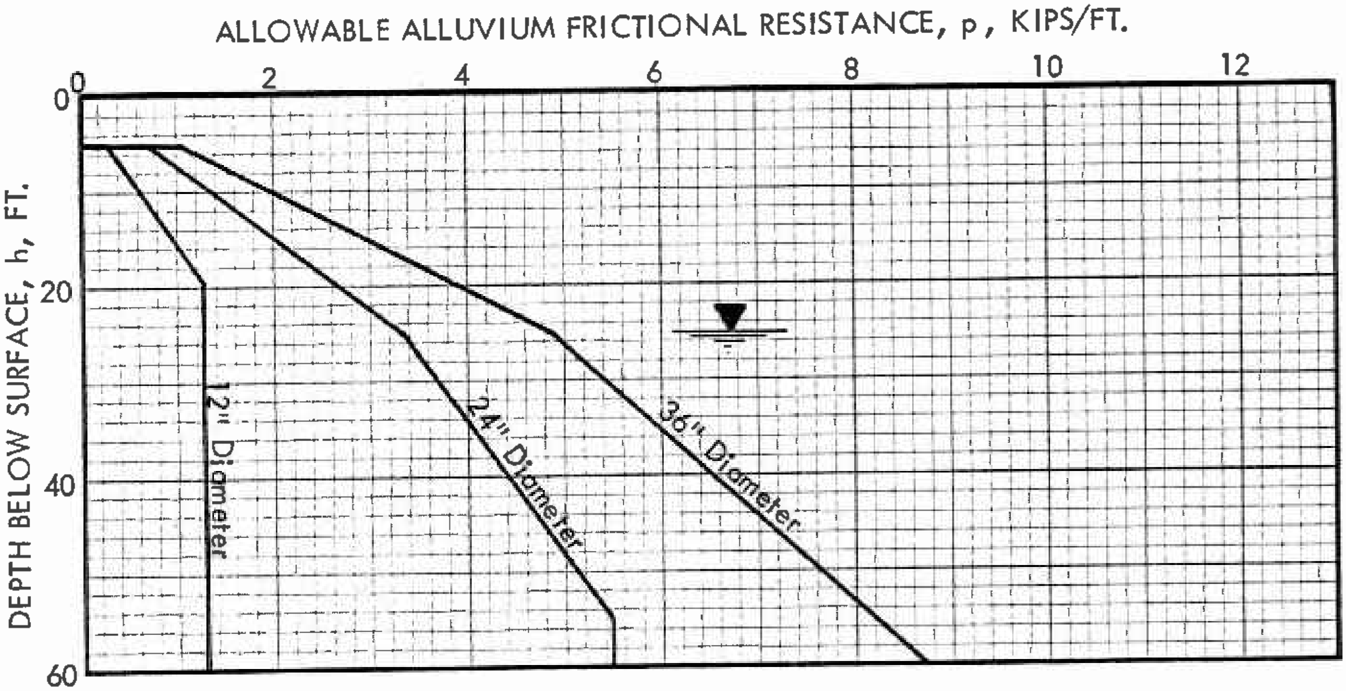

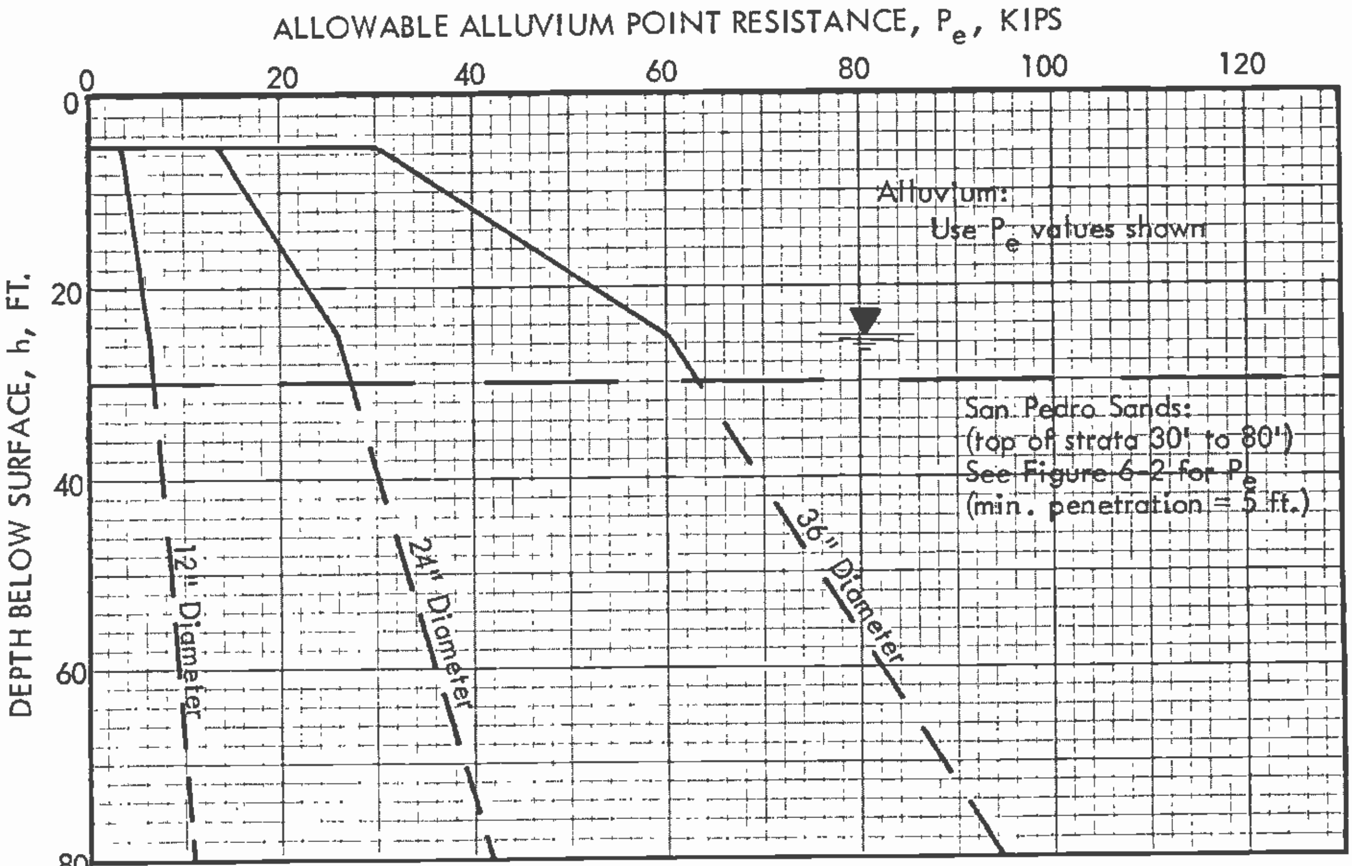

See Figure 6-2 for Determination of Total Capacity.

Approved for publication 3/84 by [illegible]

UNDERPINNING-CAST-IN-PLACE PILE DESIGN PARAMETERS (NORMANDIE)

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Figure No. 6-4

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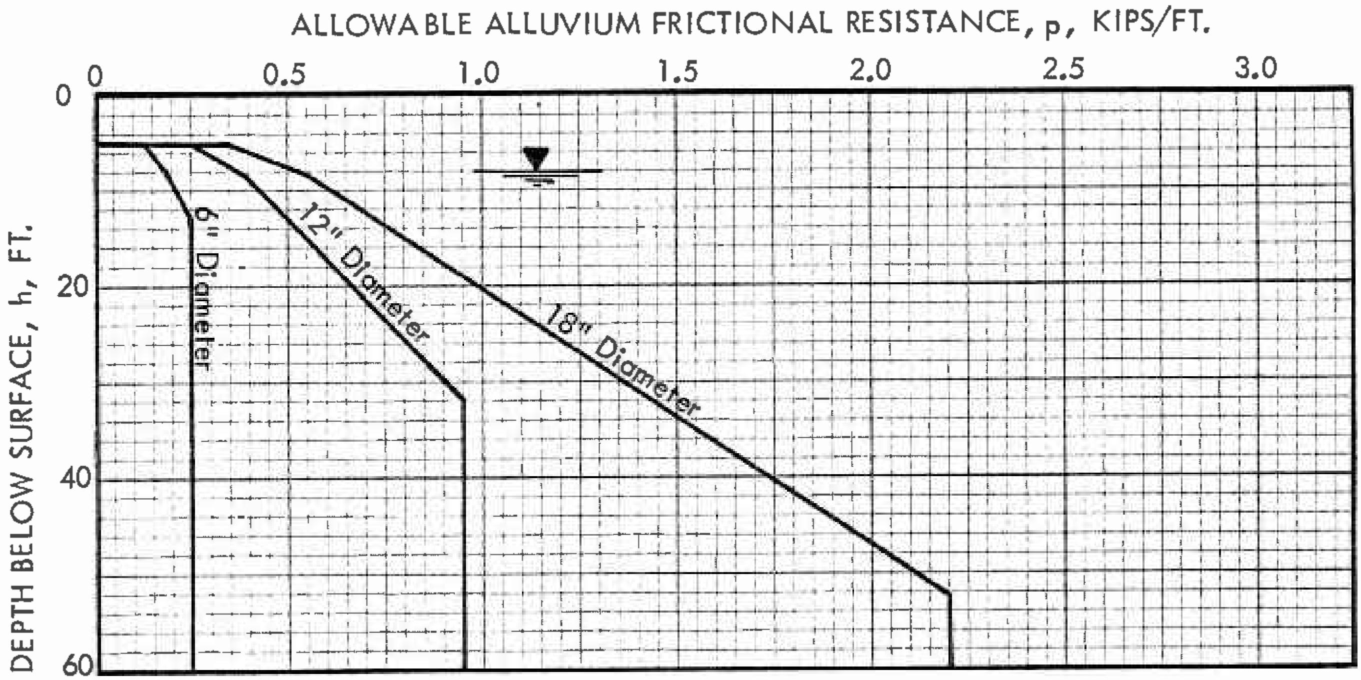

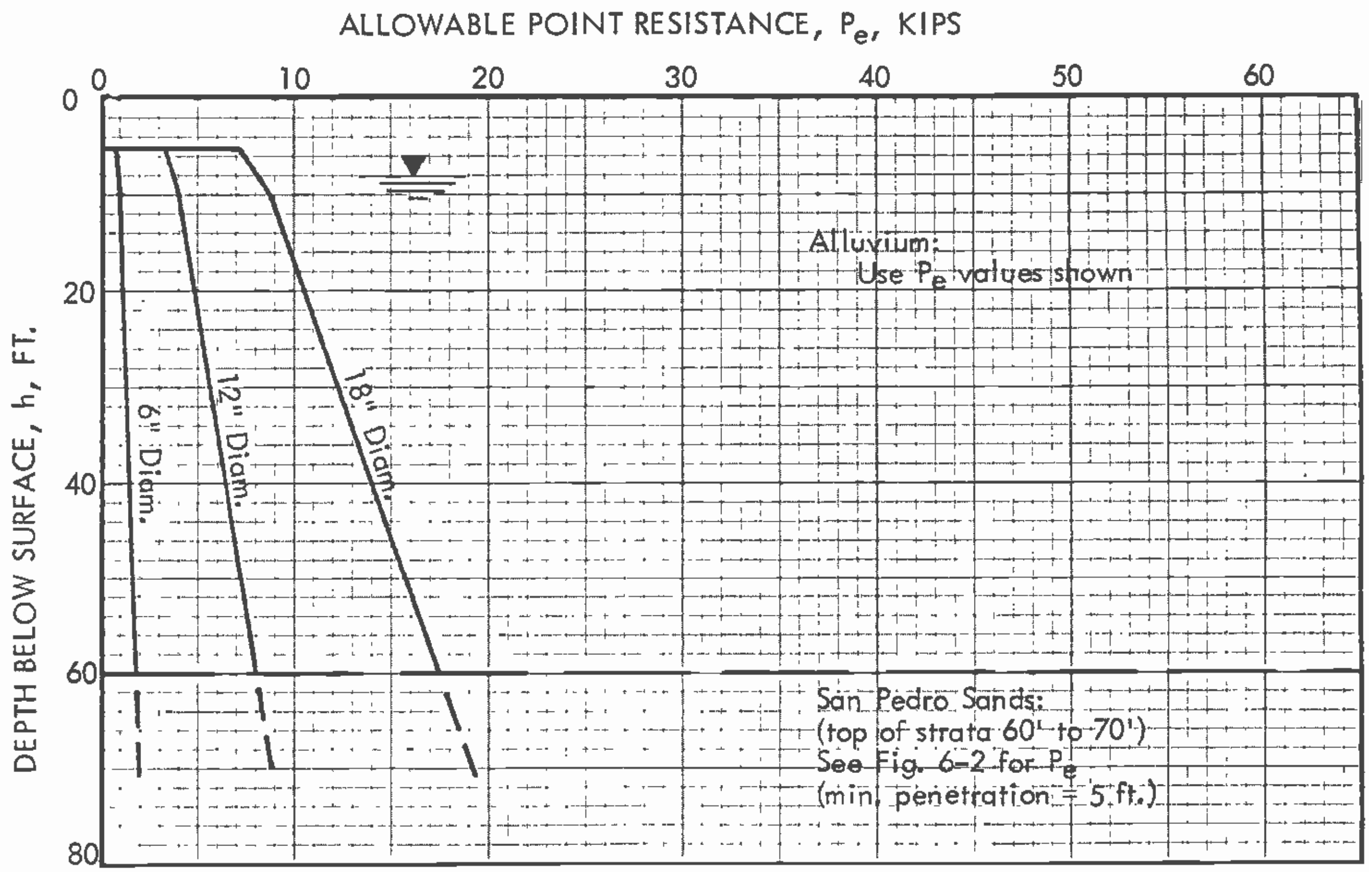

See Figure 6-2 for Determination of Total Capacity.

UNDERPINNING - JACKED PILE DESIGN PARAMETERS (WESTERN)

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Figure No. 6-5

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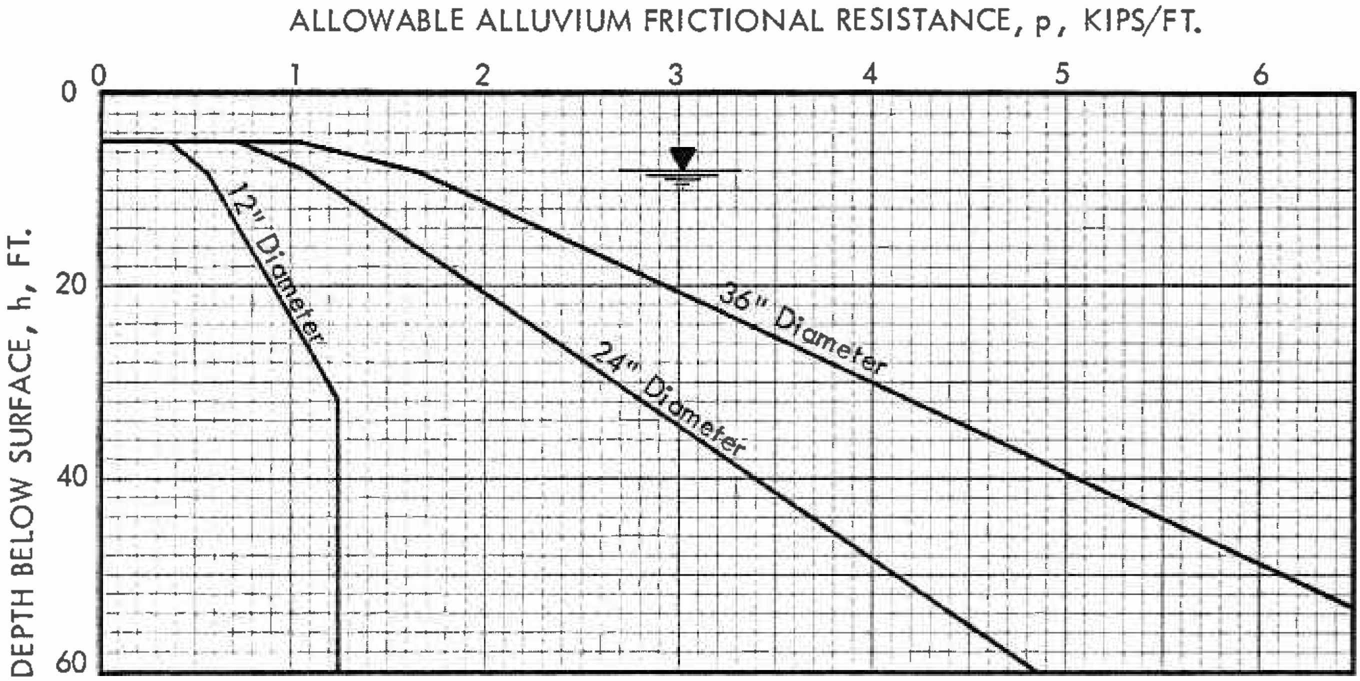

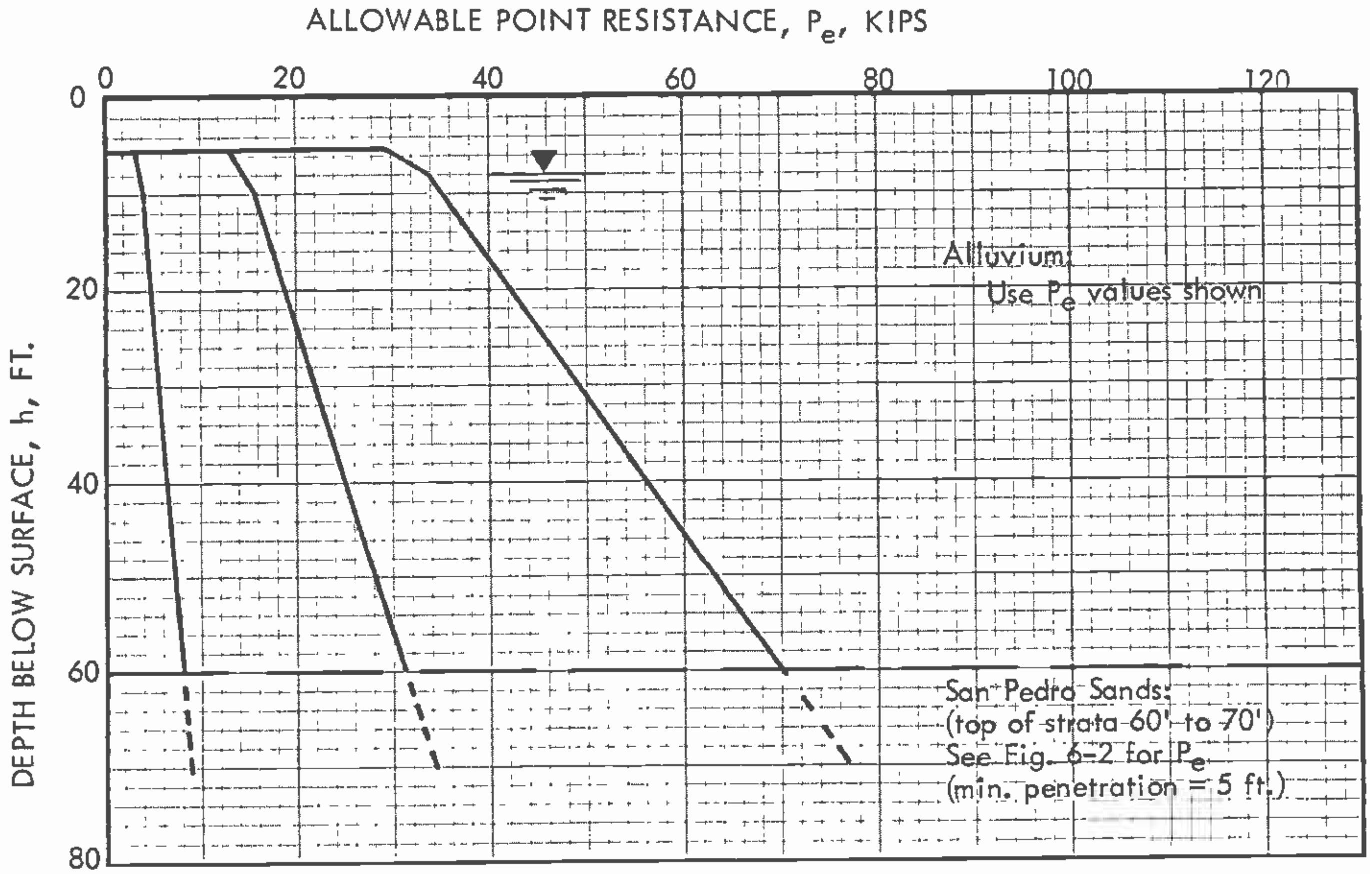

See Figure 6-2 for Determination of Total Capacity.

UNDERPINNING-CAST-IN-PLACE PILE DESIGN PARAMETERS (WESTERN)

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Figure No. 6-6

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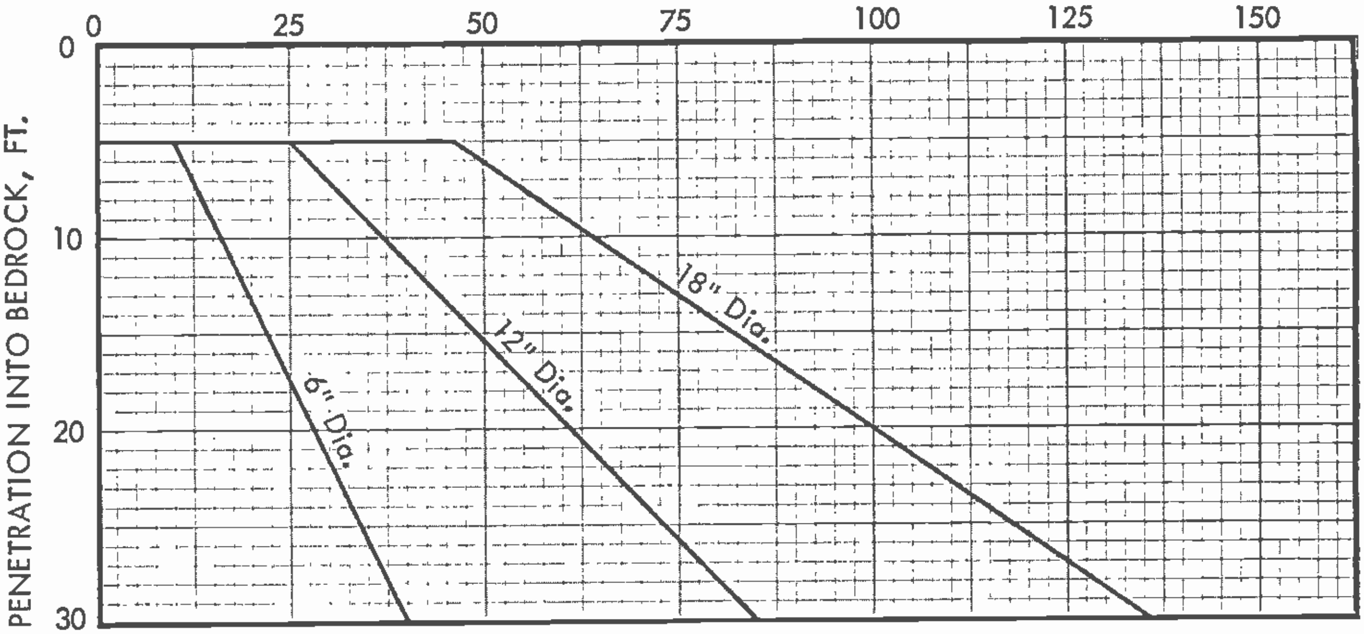
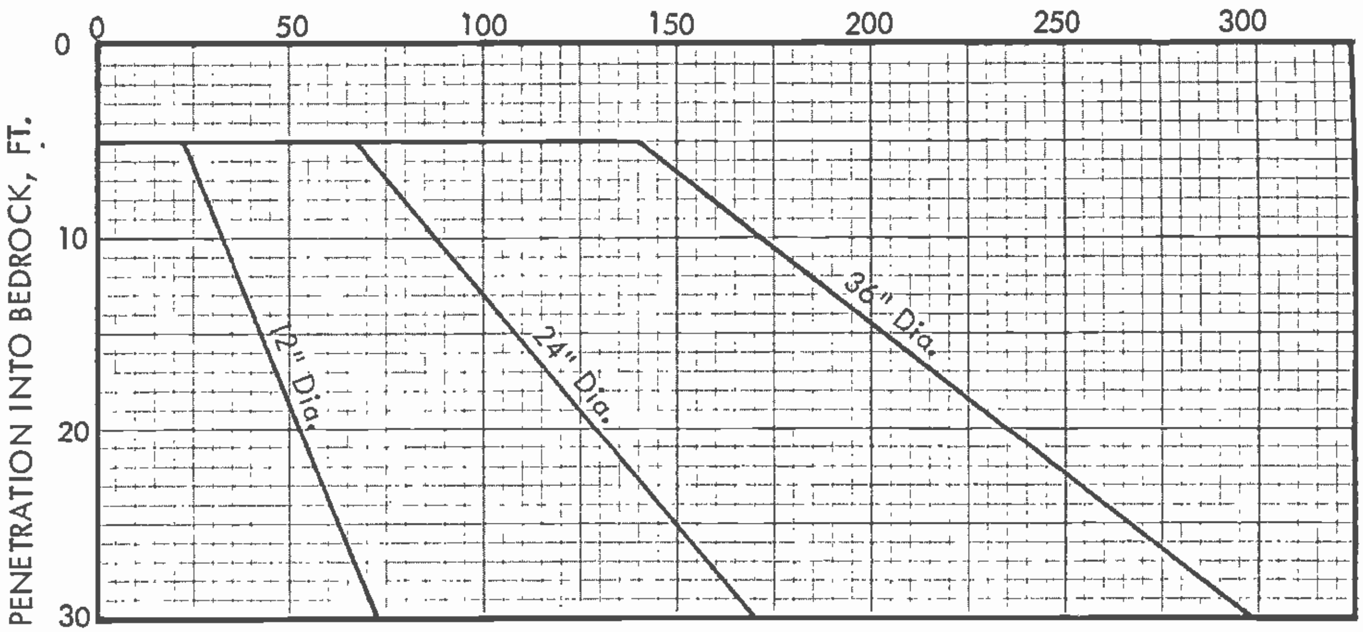
JACKED PILE-ALLOWABLE RESISTANCE, P_e, KIPS



CAST-IN-PLACE PILE-ALLOWABLE RESISTANCE, P_e, KIPS



See Figure 6-2 for Determination of Total Capacity.

Approved for publication 3/84 by JAD

UNDERPINNING-BEDROCK SUPPORT (NORMANDIE)

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Figure No. 6-7

Total capacity of hand-dug, lagged piers should be limited to end bearing only and must extend below the "no support" zone shown on Figure 6-2. All piers are assumed to be 36-inch square or larger in section. For design, an allowable bearing pressure of 7 ksf may be used for piers which bear on undisturbed alluvium and penetrate at least 10 feet below the ground surface. For piers which penetrate at least 5 feet into the San Pedro sand but are at least 5 feet above the bedrock surface, an allowable bearing pressure of 20 ksf may be used. Piers bearing on bedrock may be designed based on 15 ksf. These values apply only if the bearing surface is properly prepared and approved by a qualified engineer.

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

1. The upper 3/4 of the alluvium thickness (including soils within the "no load" zone) should be assumed to be the downdrag zone. The alluvium thickness may be estimated from Drawings 7 and 9 and should not include the San Pedro Sands.

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

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

6.3.4 Underpinning Performance

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

6.3.5 Underpinning Instrumentation

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

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

6.4 TEMPORARY EXCAVATIONS

6.4.1 General

The required A220 station excavations will extend approximately 55 to 70 feet below the existing ground surface and 30 to 40 feet below the water table. A primary consideration in the selection of the shoring system should be the effects of dewatering as discussed in Sections 6.1 and 6.2. Dewatering of the site may result in significant areal subsidence in the site vicinity which could cause downdrag and differential settlements of underpinned structures. However, this condition could be mitigated by a conservatively designed shoring system which could minimize underpinning or by a "tight" shoring system which could eliminate the need for site dewatering. There are several currently used shoring methods which include soldier piles and lagging, slurry wall construction and sheet piles. Bracing systems are generally either tieback anchors or internal bracing. We understand that the excavation system will be chosen and designed by the contractor in accordance with specified criteria and subject to the review and acceptance by the Metro Rail Construction Manager.

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

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

6.4.2 Soldier Pile Shoring Systems

A soldier pile and lagging shoring system consisting of soldier piles installed in predrilled holes is a common method of shoring deep excavations in the Los Angeles area. Both conventional and conservative soldier pile shoring systems may be used at these sites. The conservative wall should be designed for higher soil loads to reduce ground movements behind the wall. Appendix D.1 summarizes several case studies in the Los Angeles area involving soldier pile excavations to depths exceeding 100 feet.

Soldier piles have been installed in the Los Angeles area in soils similar to those encountered at the proposed A220 Station sites. In granular soils, particularly below the ground water table, caving can be a problem. The contractor should recognize that caving conditions may be encountered in construction of soldier piles or other drilled shaft elements.

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

6.4.3 Shoring Design Criteria

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

At the east end of the Wilshire/Normandie Station the shoring will penetrate the Puente bedrock. The dip of the bedrock bedding planes is approximately 40° south. It is our opinion that no variation in shoring pressure is required to account for bedding. However, passive bedrock resistance below the east end of the Wilshire/Normandie excavation will be affected by the bedding, and reduced values are recommended for the south side of that excavation.

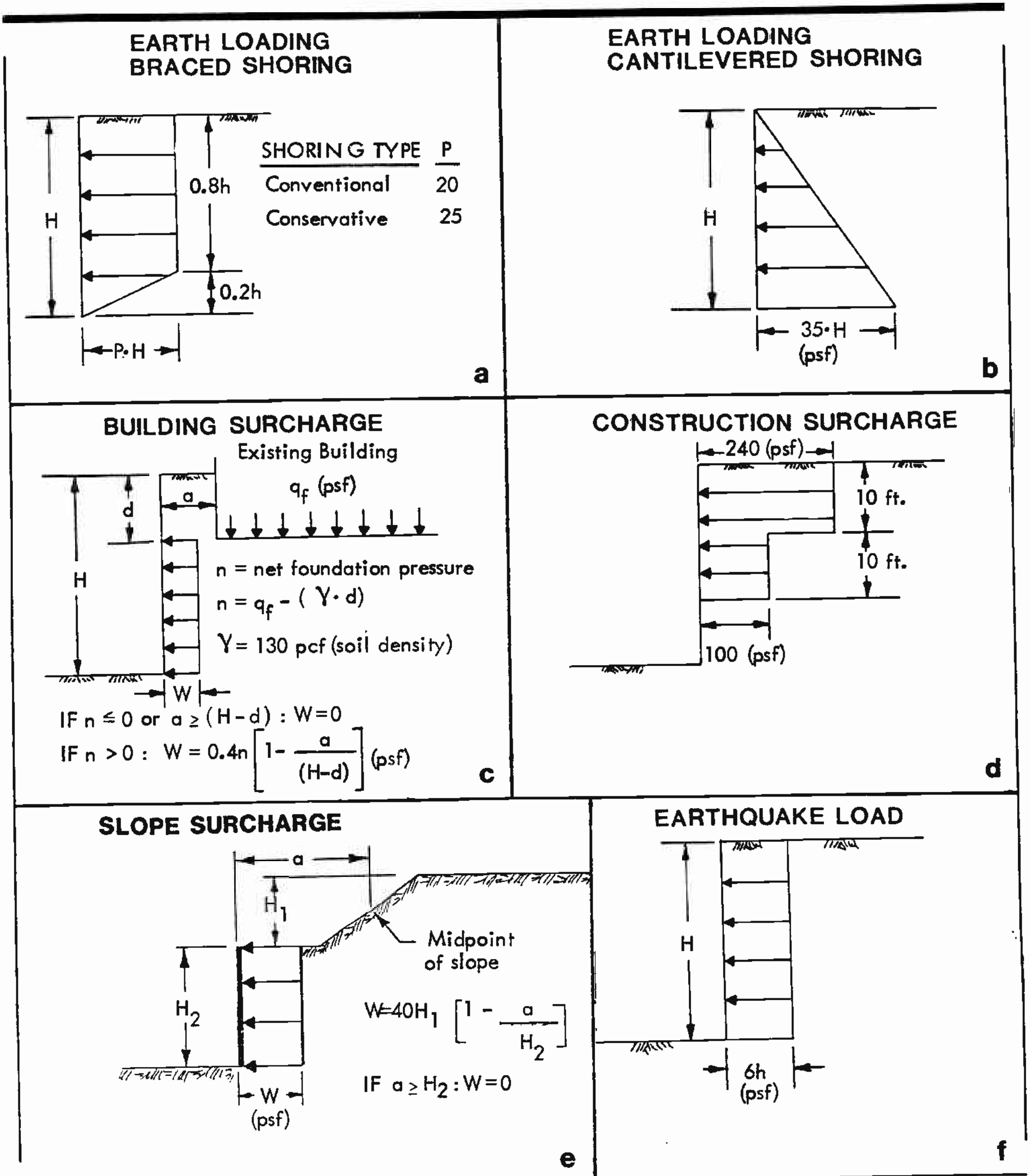
Specific shoring design criteria include:

- Design Wall Pressure: Figures 6-8a and 6-8b present the recommended lateral earth pressure on the temporary shoring walls. Design lateral pressures for both conventional and conservative shoring systems are presented in Figure 6-8a. Figure 6-8e also includes the case of partial slope cuts. Appendix D.2 provides technical support for the recommended seismic pressure of Figure 6-8f. 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 the allowable vertical loads shown on Figures 6-9 and 6-10. Figure 6-9 should be used for piles penetrating bedrock. Where the pile tip is within 5 feet vertically of the bedrock surface shown on Drawings 7 and 9, both Figures 6-9 and 6-10 should be considered and the lower capacity used. Figure 6-10 should be used for all other piles and it should be noted that all piles should penetrate at least 5 feet into the San Pedro bearing stratum.

  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-8 minus the support from tiebacks or internal bracing. The required depth of embedment to satisfy lateral loads should be computed based on the net allowable passive resistance



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No. 6-8

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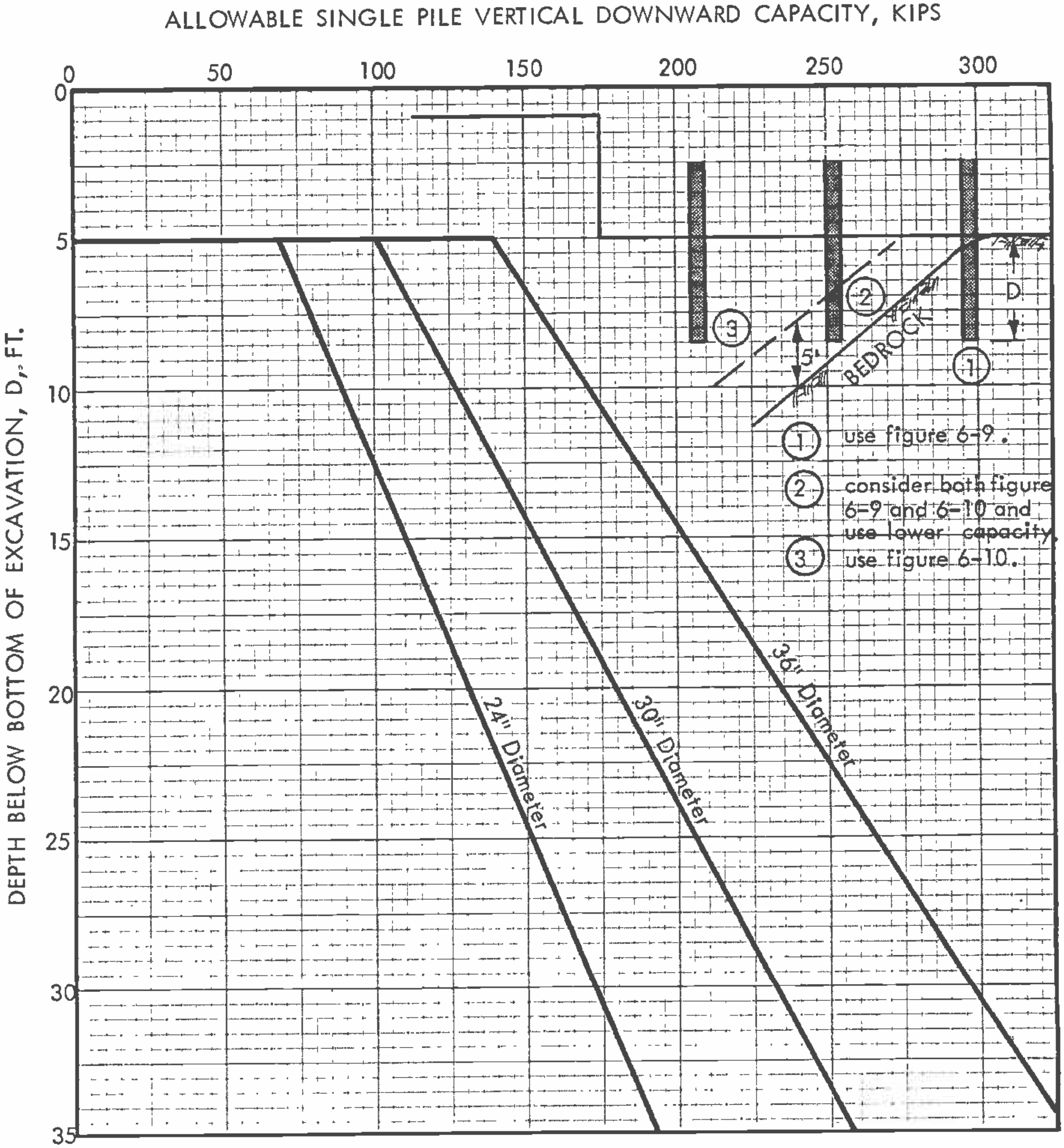

NOTES: 1.) For seismic design, capacities may be increased 33%.
2.) Applicable only for drilled shaft piles.

VERTICAL CAPACITY OF PILES FOR SHORING & DECKING (BEDROCK)

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Figure No.
6-9

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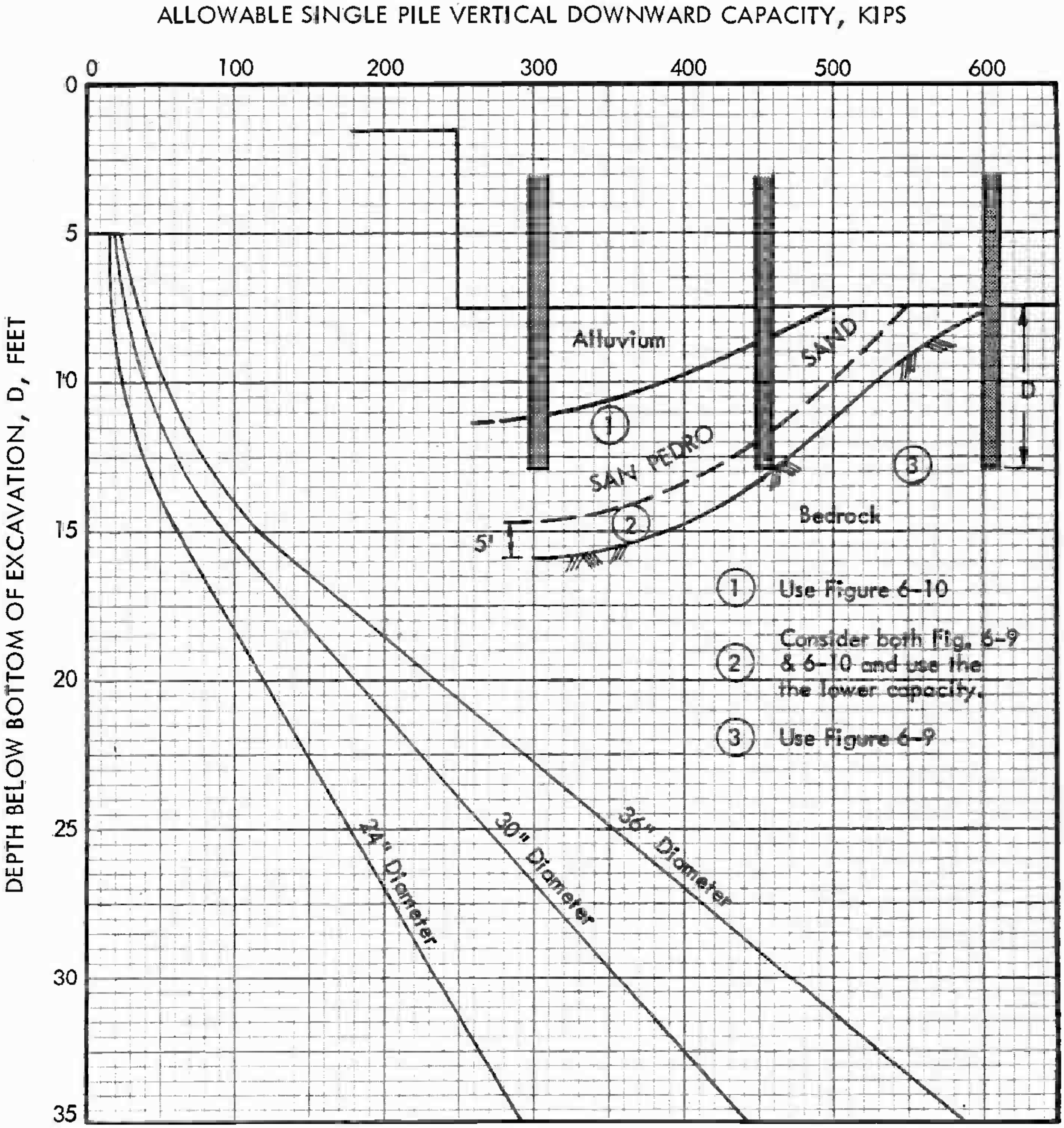

Notes: 1) All piles must penetrate at least 5 feet into the San Pedro Sand.

2) For seismic design, capacities may be increased 33%.

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VERTICAL CAPACITY OF PILES FOR SHORING & DECKING (SAN PEDRO)

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Figure No. 6-10

(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. Figures 6-11 and 6-12 indicate the recommended method to compute net passive resistance. Figure 6-11 should be used for piles penetrating Puente bedrock. A reduced maximum passive resistance is recommended for the south side of the east Wilshire/Normandie excavation due to expected adverse bedrock bedding. Figure 6-12 should be used for all piles which do not penetrate bedrock.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. At the A220 Station sites the alluvial soils encountered were generally clayey. However, occasional silty sands layers may be exposed and these soils would be subject to ravelling and sloughing. Thus, it is recommended that the pile spacing be limited to about 8 feet and that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles. The contractor should limit the temporary exposed height of sandy soil to less than 3 feet to control ravelling problems, especially in the dewatered zone.

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

6.4.4 Internal Bracing and Tiebacks

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

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

6.4.4.3 Internal Bracing: The contractor should not be allowed to extend the excavation an excessive distance below the lowest strut level prior to installing the next strut level. The maximum vertical distance depends on several specific details such as the design of the wall and the allowable ground movement. These details cannot be

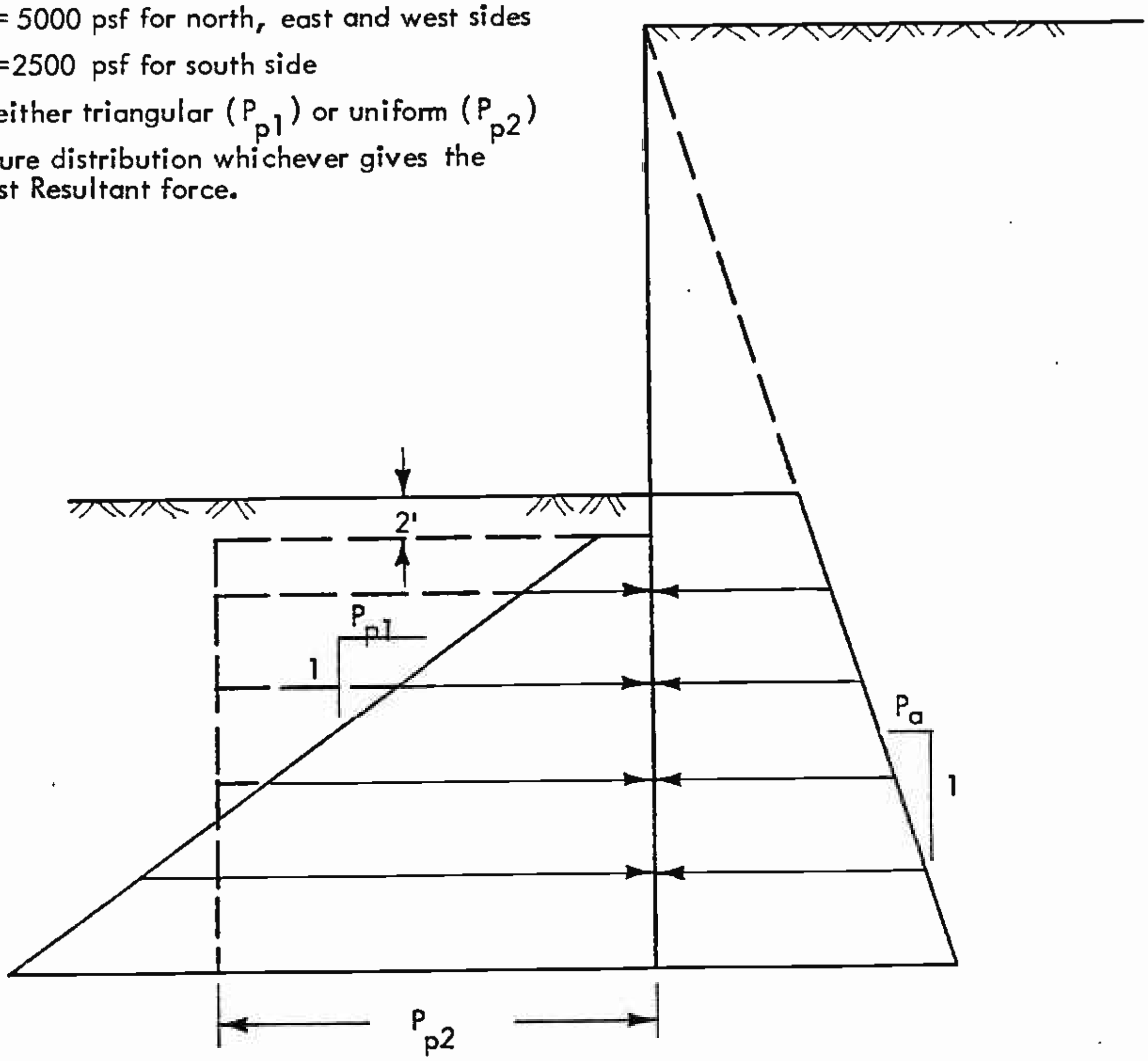
Recommended Unit Pressures

P_a = 35 psf/ft

P_{p1} = 450 psf/ft

P_{p2} = 5000 psf for north, east and west sides
=2500 psf for south side

Use either triangular (P_{p1}) or uniform (P_{p2}) pressure distribution whichever gives the lowest Resultant force.



Where: P_p = Total allowable unit passive pressure

P_a = Unit active pressure

NOTE: 1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive - Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-8

SOLDIER PILE PASSIVE RESISTANCE (IN BEDROCK)

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Figure No. 6-11

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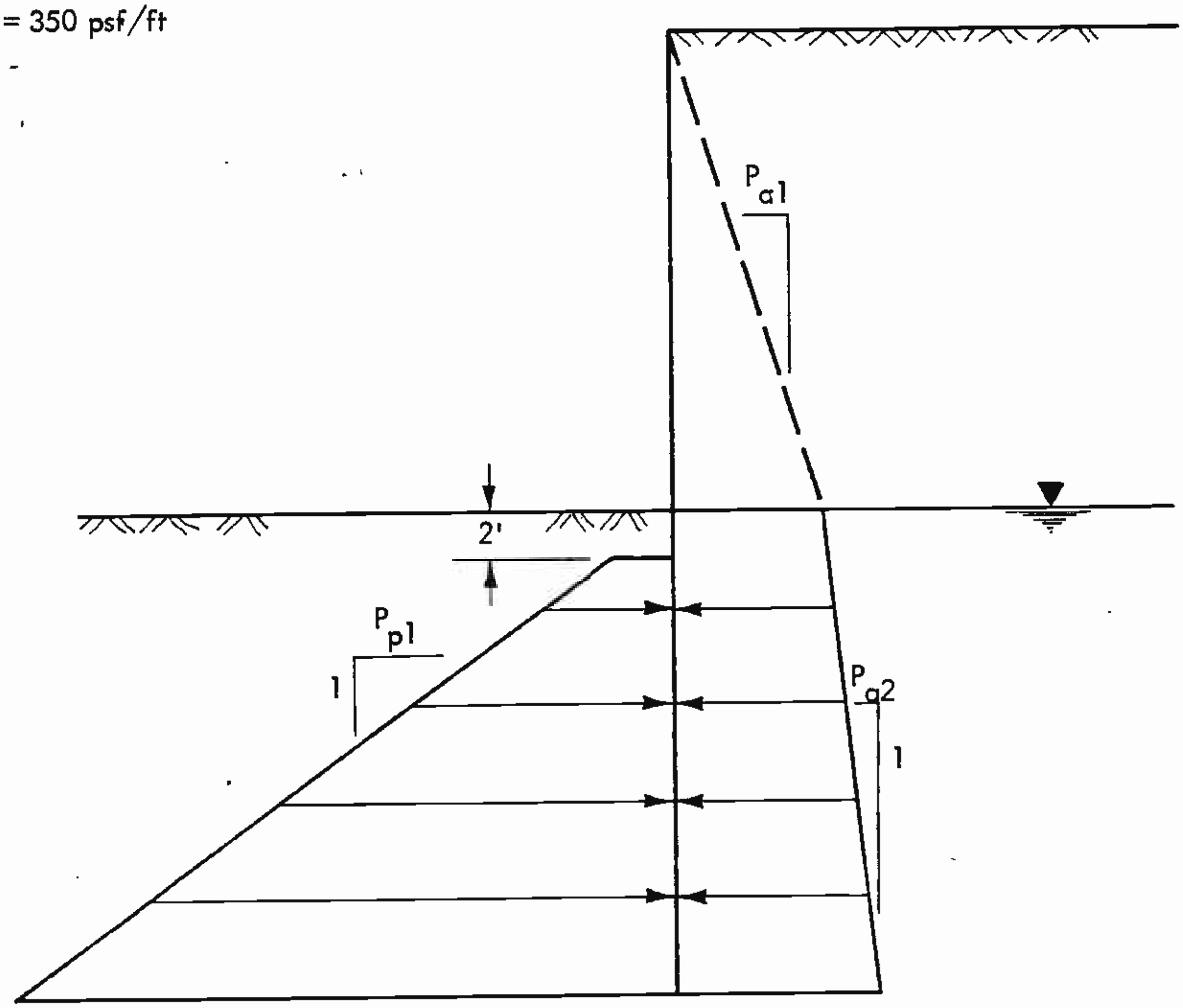
Recommended Unit Pressures

P_{a1} = 35 psf/ft

P_{a2} = 19 psf/ft

P_{p1} = 350 psf/ft



Where: P_p = Total allowable unit passive pressure

P_a = Unit active pressure

NOTE:
1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive − Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-8

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SOLDIER PILE PASSIVE RESISTANCE (IN ALLUVIUM)

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Figure No. 6-12

generalized. However, as a guideline, we recommend consideration of the following maximum allowable vertical distances between struts:

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

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

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

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

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

$$P = \pi DLq$$

Where:

P = allowable anchor design load in pounds
D = 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:

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

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

Where:

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

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

Figure 6-13 illustrates the tieback anchor parameters.

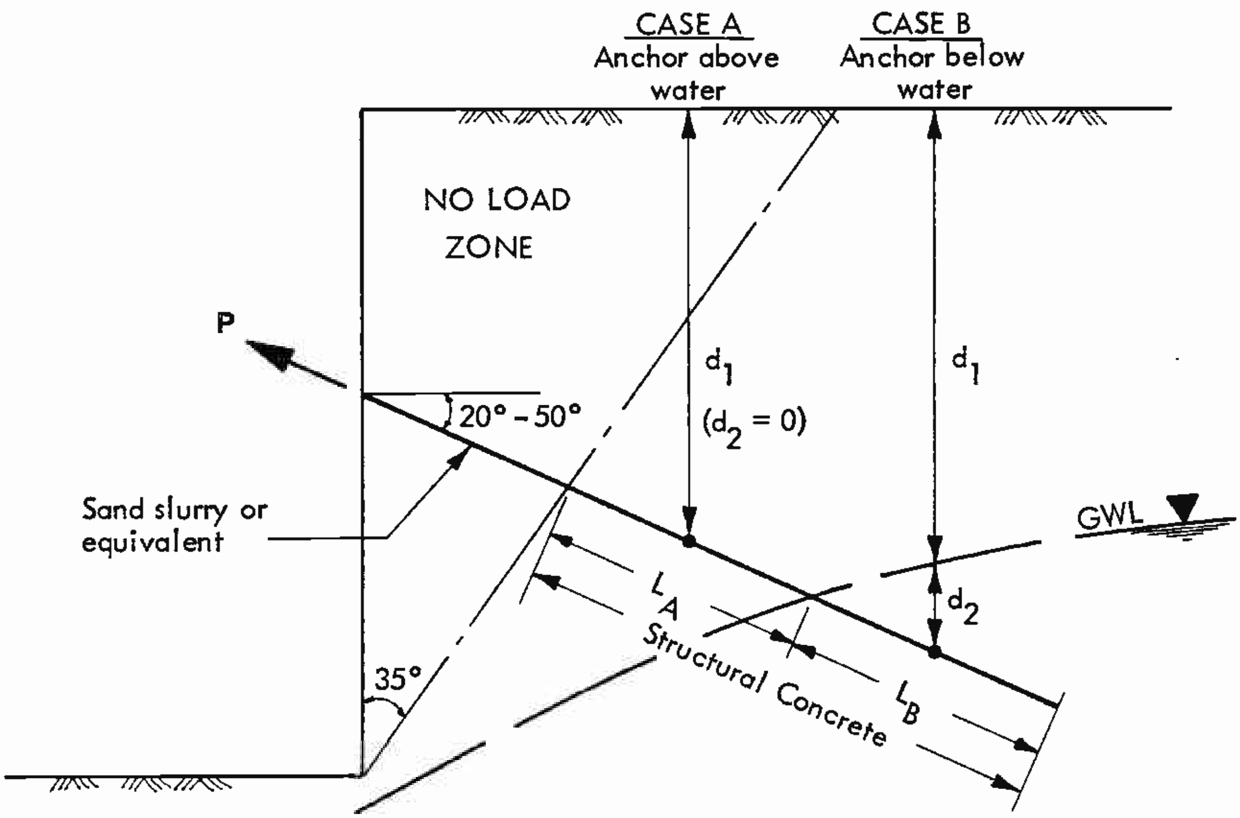
Allowable anchor capacity/length relationships for tieback types other than straight shaft friction anchors cannot be generalized. Design parameters for anchors such as high pressure grouted anchors and high pressure regroutable anchors must be based on experience in the field and on the results of test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

For design purposes, it should be assumed that the potential wedge of failure behind the shored excavation is determined by a plane drawn at 35° with the vertical through the bottom of the excavation for alluvial soil conditions. The failure plane for the Puente bedrock on the north side of the Wilshire/Normandie Station should be assumed parallel to the dip of the bedding planes. Only the frictional resistance developed beyond the no-load zone should be assumed effective in resisting lateral loads.

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

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

It is recommended that each tieback anchor be test loaded to 150% of the design load and then locked off at the design load. At 150% of the design load, the anchor deflection should not exceed 0.1 inches over a 15-minute period. In addition, 5% to 10% of the anchors should be test-loaded to 200% of the design load and then locked off at the design load. At 200% of design load the anchor deflections should not exceed 0.15 inches over a 15-minute period. The rate of deflection should consistently decrease during the test period. If the rate of deflection does not decrease the test should not be considered satisfactory.



NOTE:

The design adhesion value, q, can be evaluated by

$q = 750$ psf (in all bedrock)

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

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

The total anchor capacity can be estimated by:

$$P = \pi D L_A q_A + \pi D L_B q_B$$

See also Section 6.4.4.4

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STRAIGHT SHAFT TIEBACK ANCHOR CAPACITY

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Figure No. 6-13

6.4.5 Anticipated Ground Movements

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

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

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

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

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

6.4.6 Historical Shoring Pressure Diagrams - Los Angeles

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

6.5 SUPPORT OF TEMPORARY DECKING

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

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that he may use similar piles to support the center decking. Accordingly, we evaluated the allowable loads on these types of piles for several typical diameters. The recommended allowable design loads are shown on Figures 6-9 and 6-10. These values include both end bearing and shaft friction.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed A220 Station excavations 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. 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 zones 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 station excavation. Inclinometers should be located on each side of the excavation. The casing could be installed within the soldier pile holes or in separate holes immediately adjacent to the shoring wall. Baseline readings of the inclinometers should be made immediately upon installation. Subsequent readings should be made at regular time intervals during excavation and construction.

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

  We recommend that heave gages be installed prior to construction 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.

° Measurements of Strut Loads: If internal bracing is used, we recommend that the loads on at least four struts at each support level be monitored periodically during the construction period. These measurements provide data on support loads and a forewarning of load reductions which would result in excessive ground movements. There are several methods to obtain these data. A commonly used method involves vibrating wire strain gages mounted on studs welded to the struts. For full measurements of maximum stresses, a minimum of three gages is needed on a pipe strut and four on a wide flange strut. However, two gages are often used to simplify the installation and monitoring effort with acceptable results. There should be a means of measuring the strut temperature at the time of the strain 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 STATION STRUCTURES

The proposed excavations will substantially change the ground stresses below and adjacent to the excavations. The proposed 65-foot excavation at Wilshire/Normandie will decrease the vertical ground stresses by about 7000 psf. The proposed 55 foot deep excavation at Wilshire/Western will result in a ground stress reduction of about 4800 psf. Stress reduction caused by the excavation will result in rebound or heave of the alluvium and bedrock below the excavations. This response is not due to the occurrence of any type of swelling soils but simply an elastic response to stress unloading. In addition, even with a suitable shoring system, shear stresses will develop tending to cause the bedrock adjacent to the walls to heave upward. Since the excavations will be open for an extended period, the heave is expected to be completed prior to construction of the Stations. The Station structures and subsequent backfilling will reload the soil. We estimate that the net Station loads will be about 2000 to 4000 psf. This load 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 both excavations will be on the order of 1½ to 3 inches. We also believe that the majority of this will occur while the excavation is being made. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (consolidation heave) within the bedrock underlying the proposed excavation. Due to the hard consistency of the bedrock, the majority of the deformation will be elastic rebound. These values agree well with observed behavior in similar excavations in the Los Angeles area (Evans, 1968).

It was computed that the estimated imposed loads from the structures and backfill will induce settlements on the order of 1 to 2-1/2 inches. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/3 inches over the width of the structures. This correlates to an angular rotation of only about 1:1100. At the Wilshire/Normandie Station differential settlement between the alluvial supported west end the San Pedro Formation in the central portion and the bedrock supported east end could be one inch. However, the maximum longitudinal angular distortion is estimated to be only about 1:1800. Differential settlements at the Wilshire/Western Station should be equal or less than the values estimated for Wilshire/Normandie.

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

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

6.8 FOUNDATION SYSTEMS

6.8.1 Main Stations

It is understood that the proposed Stations will be supported on thick base slabs which will function as massive mat foundations. We estimate that the net mat foundation bearing pressures will be about 2000 to 4000 psf. In our opinion the stations can be adequately supported on mat foundations bearing on undisturbed soil/bedrock subgrade materials. Section 6.7 presents estimated settlements for the proposed station structures.

6.8.2 Support of Surface Structures

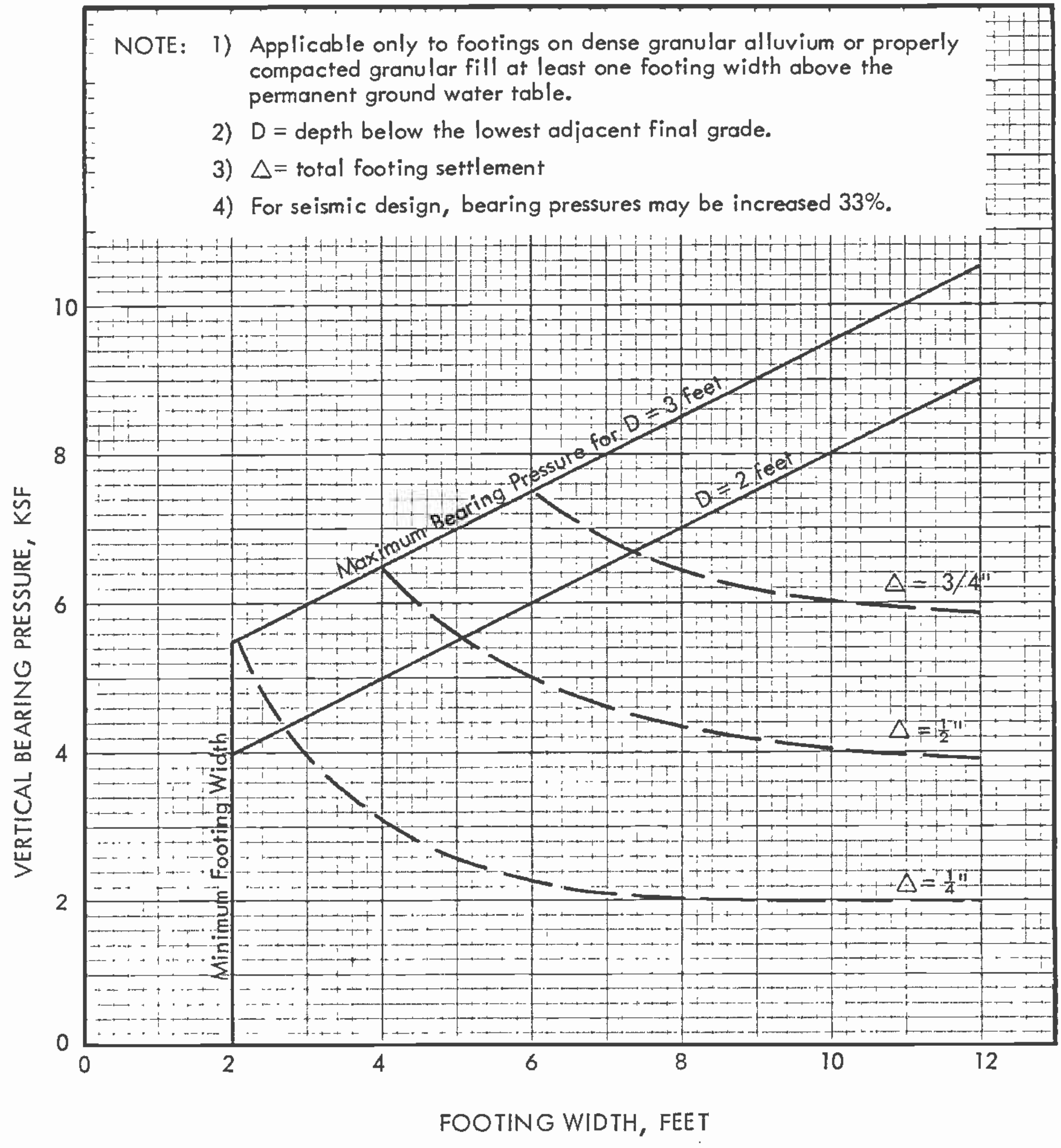
Surface structures can be generally supported on conventional spread footings founded on undisturbed stiff or dense natural soils. If suitable natural soils do not exist at the surface structure site, footings may be founded on a zone of properly compacted structural fill (see Appendix E). Allowable bearing pressures and estimated total settlements of spread footings bearing on the natural alluvium or compacted fill can be determined based on Figures 6-14 and 6-15. These figures are based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed information on structural loadings and site conditions at the surface structure location. Detailed site specific studies should be performed to provide final design recommendations for specific structures.

All spread footing foundations should be founded at least 2 feet below the lowest adjacent final grade and should be at least 2 feet wide. The bearing values shown on Figures 6-14 and 6-15 are for full dead load and frequently applied live load. For transient loads, including seismic and wind loads, the bearing values may be increased by 33%. Differential settlements between adjacent footings should be estimated as 1/2 of the average total settlements or the difference in the estimated total settlements shown on Figures 6-14 and 6-15, whichever is larger.

For design, resistance to lateral loads for surface structures may be assumed to be provided by passive earth pressure and friction acting on the foundations. An allowable passive pressure of 350 psf/ft may be used for the sides of footings poured neat against dense or stiff alluvium or properly compacted fill. Frictional resistance at the base of foundations should be determined using a frictional coefficient of 0.4 with dead load forces.

6.9 PERMANENT GROUND WATER PROVISIONS

We understand that the stations will be designed to be water-tight and to resist the full permanent hydrostatic pressures. We recommend that full waterproofing be carried at least 5 feet above the anticipated maximum ground water levels given in Section 6.10 for the Wilshire/Normandie Station. The entire structure at Wilshire/Western will require waterproofing because of the high ground water condition.



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SPREAD FOOTING BEARING/SETTLEMENT ON GRANULAR SOILS

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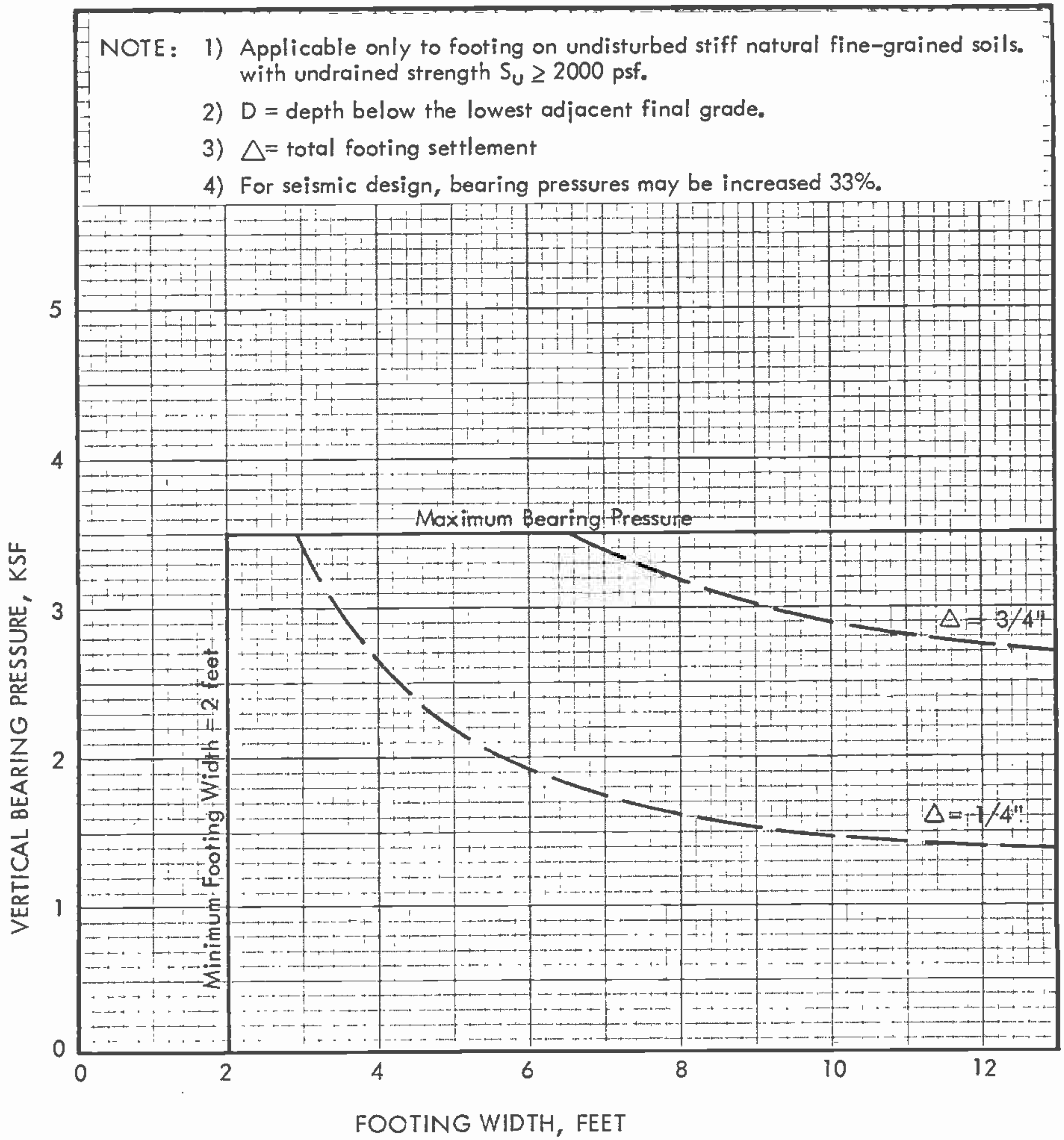
Figure No. 6-14



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SPREAD FOOTING BEARING/SETTLEMENT ON FINE-GRAINED SOILS

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METRO RAIL PROJECT

Project No. 83-1140

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Figure No. 6-15

6.10 LOADS ON SLAB AND WALLS

6.10.1 Hydrostatic Pressures

As discussed in Section 5.4, the existing ground water levels as measured at the boring locations were at about Elevation 186 at the Wilshire/Normandie site and about Elevation 180 to 182 at the Wilshire/Western site. The winter of 1983 was one of the five wettest years in the past 100 years and, therefore, the measured levels are considered to represent near maximum levels. It is recommended that the following ground water levels be assumed for determining hydrostatic pressures:

LOCATION	ELEVATION (ft)
Wilshire/Normandie Station	196
Wilshire/Western Station	190

6.10.2 Permanent Static Earth Pressures

Figure 6-16 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. A total unit weight of 130 pcf may be used.

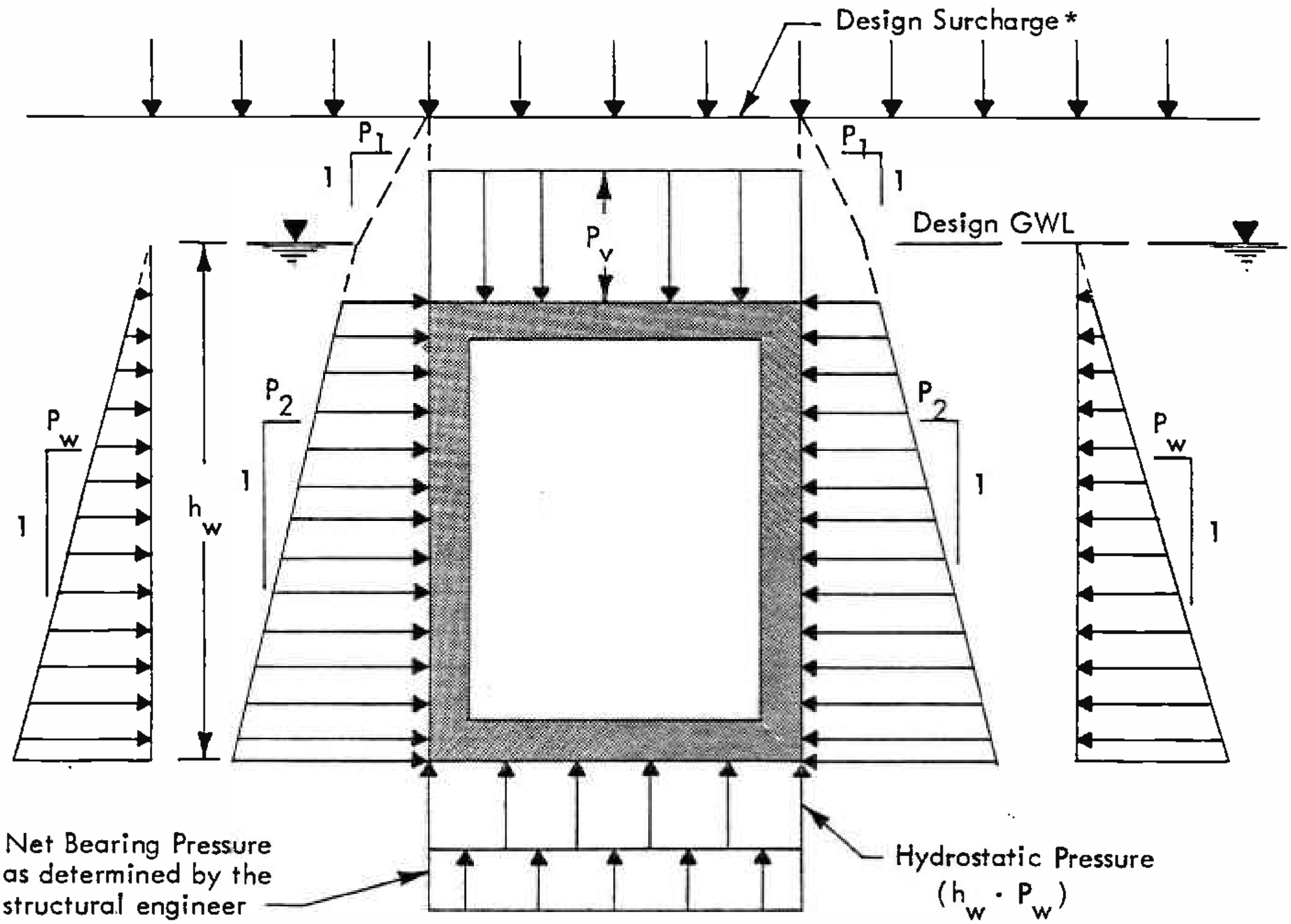
6.10.3 Surcharge Loads

Lateral surcharge loads from existing buildings not underpinned must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-8. Vertical surcharge loads due to surface traffic, etc. should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.

6.11 SEISMIC CONSIDERATIONS

6.11.1 General

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



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	40	20	62.4	*	**
Long Term	60	30	62.4	*	***
Side sway †	40/60	20/30	62.4	*	**

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

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

*** Elev. 196 ft.at Wilshire/Normandie Sta. and elev. 190 ft. at Wilshire/Western Sta.

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

LOADS ON PERMANENT WALLS

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Figure No. 6-16

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6.11.2 Dynamic Material Properties

Values of apparent wave propagation velocities for use in travelling wave analyses have been previously recommended in the May, 1983 Seismological Investigation and Design Criteria Report. Other dynamic soil parameters required for input into the various types of analyses recommended in the seismic design criteria report are also given. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels.

TABLE 6-1

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		ALLUVIUM	SAN PEDRO SAND	PUENTE BEDROCK
Average Compression Wave Velocity, V_c (ft/sec)	- moist	4000		5700
	- saturated	5000	5000	
Average Shear Wave Velocity, Vs (ft/sec)		950	960	1300
*Poisson's Ratio		0.40	0.35	0.35
**Young's Modulus, E, (psi)	- moist	207,000		530,000
	- saturated	185,000	185,000	
**Constrained Modulus, E_c, (psi)	- moist	450,000		850,000
	- saturated	700,000	700,000	
**Shear Modulus, G_{max}, (psi)		25,000	25,000	45,000

* For saturated alluvium, use value of 0.45.

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

Average values of compression and shear wave velocities based on interpretation of limited downhole geophysical surveys performed in Boring CEG-14 and CEG-15 and other borings in similar materials during the 1981 investigation are presented in Table 6-1. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish modulus values at low strain levels. Computed modulus values for the various geologic units present at the Station sites, are also tabulated in Table 6-1.

The variation of dynamic shear modulus, expressed as the ratio of G/G_{max}, with the level of shear strain is presented in Figure 6-17 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 6-18.

6.11.3 Liquefaction Potential

The generalized subsurface cross section has been described in Section 5.0 and is shown in Drawings 2,3,7 and 9. The ground water level appears to have a slight westward gradient declining from Elevation 192 in Boring CEG-14 to Elevation 185 in Boring 15-1. These ground water elevations correspond to depths of below the ground surface of about 35 feet at Wilshire/Normandie and about 15 feet at Wilshire/Western. The soils which are below the ground water level and, therefore, must be evaluated for liquefaction potential include the alluvial soils and the San Pedro Formation Sand.

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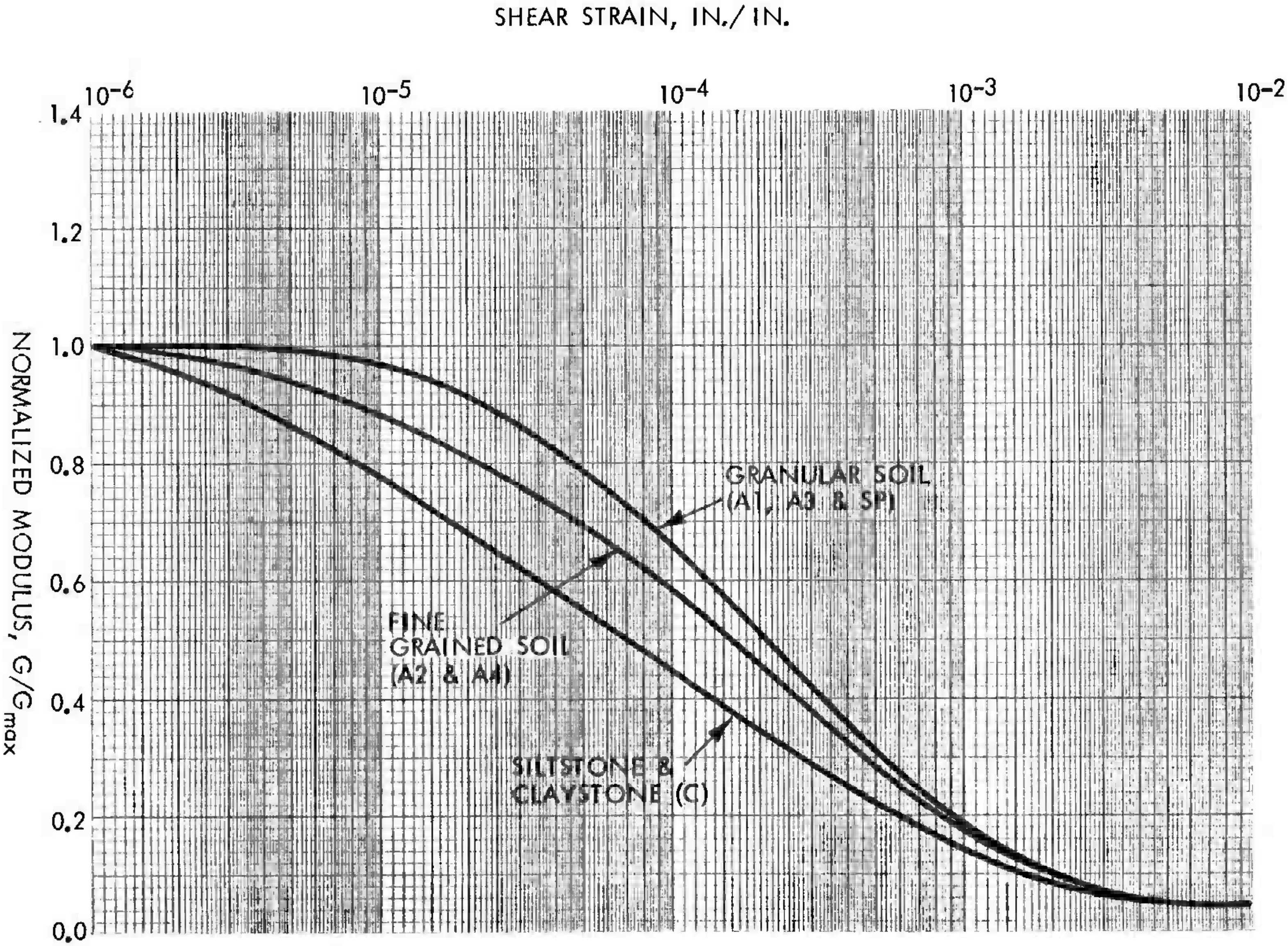

RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No. 6-17

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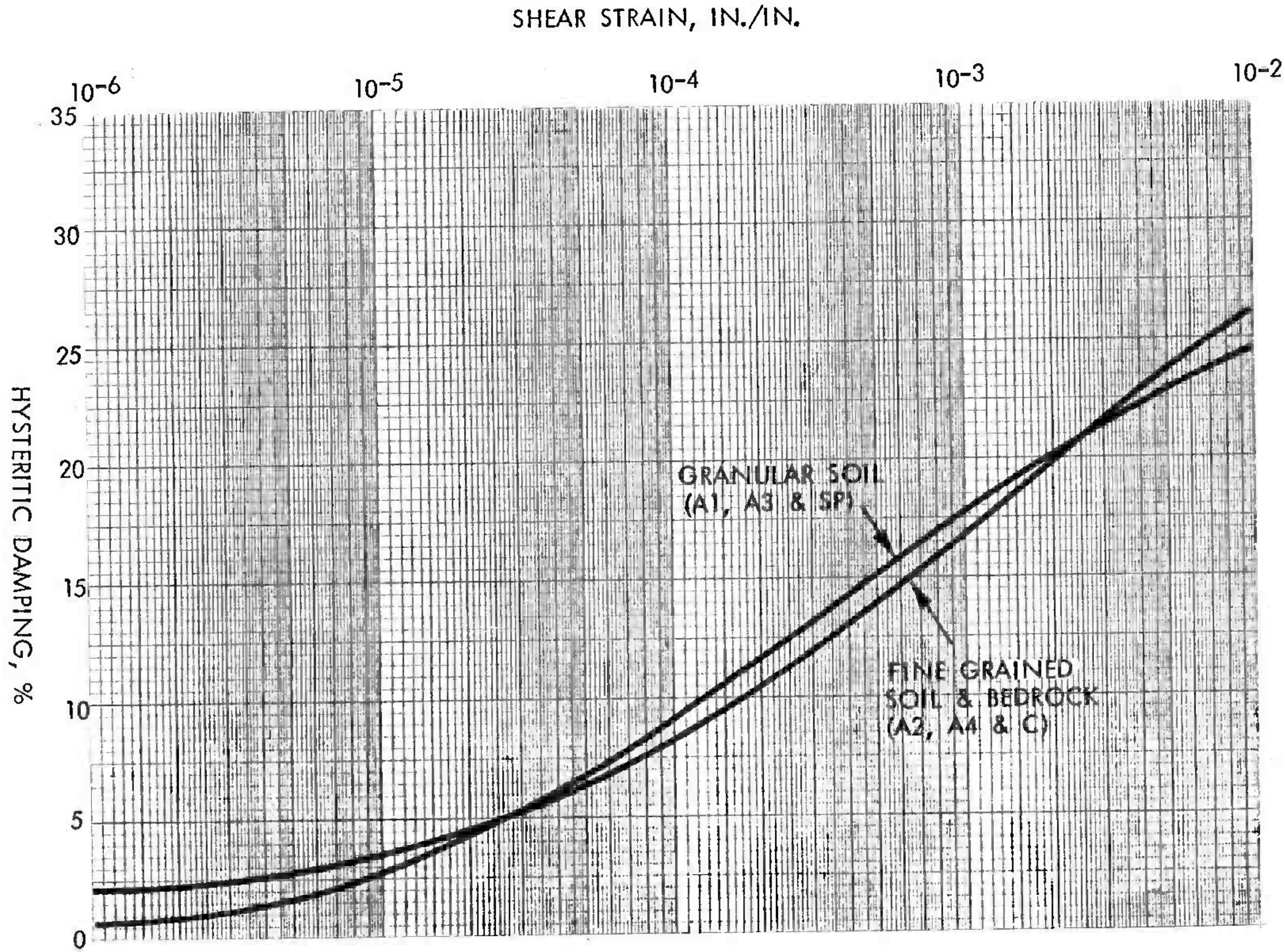

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No. 6-18

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Our liquefaction evaluation was based on procedures and correlations published by Seed et al (1983) which utilized index soil properties and performance data for soils during previous earthquakes. Field Standard Penetration Tests (SPT), available field geophysical data from CEG-14 and CEG-15, and laboratory classification test data were all used in our evaluation of liquefaction potential (see Appendix E).

Published correlations of SPT data and liquefaction potential have historically been made for granular soils. Measured SPT "N" values in the San Pedro Sands were all greater than 100 blows (refusal) and, therefore, these materials are considered to have a very low liquefaction potential even under the maximum design earthquake. Corrected "N" values (normalized to 2 ksf overburden pressure) for 20 SPT tests in saturated granular alluvium ranged from 10 to 82 with an average of about 47. Determination of dynamic strength was based on an M7.0 (maximum design) earthquake event. Only one SPT value (N=10) indicated a potential for liquefaction of the granular alluvium.

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

Considering the above discussed results, it is our opinion that the potential for liquefaction at the A220 Station sites is low.

6.12 EARTHWORK CRITERIA

Site development is expected to consist primarily of excavation for the subterranean structure but will also include general site preparation, foundation preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations and dewatering are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill are presented in Appendix F. Recommended specifications for compaction of fill are also presented in Appendix F. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. The excavated existing fills, fine-grained soils and bedrock material are not considered suitable because these fine-grained materials will make compaction difficult and could lead to fill settlement problems after construction. If the granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the geotechnical engineer.

6.13 PAVEMENT SECTION

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

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

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

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

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

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 A220 tunnel alignment is shown on Drawings 2, 3, 4 and 5. The tunnels occur between Station 319+00 and Station 474+00, a distance of about 2.5 miles, deducting for the stations within this Design Unit.

The average depth of ground cover above the crown of the tunnels is 35 feet, varying between a minimum of 25 feet near Station 458+00 and a maximum of 57 feet near Station 337+00, except where the AR line tunnel passes beneath the southeast corner of the Equitable Life Assurance Company building. The crown of the tunnel is always below the recorded water level in the alluvium.

7.1 STRATIGRAPHY, GROUND WATER AND TUNNELLING CONDITIONS

The geologic units existing along the tunnel alignment consist of cohesionless and cohesive alluvium (A_3/A_4); San Pedro Sands (SP) and bedrock-type materials of the Puente/Fernando Formation (C). 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 cut-and-cover stations and at significant changes in subsurface stratigraphy and/or conditions.

7.1.1 Station 319+50 and Station 345+50 (2600 feet - Drawing 2)

The tunnels leaving the Vermont Street Station will primarily pass through the Puente bedrock formation except that the crowns of the tunnels may encounter mixed-face conditions for a distance of approximately 300 feet east of the Vermont Station. The rock-alluvium interface may vary locally from that shown on Drawing 2, wherein 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 silts and/or sands overlying soft weathered Puente siltstone, claystone materials. The ground water level above the crown is about 17 feet between the west end of the Vermont Station and Station 322+50. It is anticipated that flowing ground conditions may be encountered at the crown 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 silt-stones and claystones of the Puente formation with occasional hard sandstone beds.

West of approximately 322+50, the tunnels enter the Puente Formation completely, attaining a maximum cover of 35 feet of Puente material at approximately Station 337+00. Near Station 325± (624 Berendo Street apartment building), Boring 13-7 records about 15 feet of Puente bedrock over the crown. Near Station 332± (630 Kenmore Street apartment building), Boring 13-8 indicates there is about 30 feet of Puente bedrock over the crown.

At approximately Station 337±, the AR line tunnel passes beneath the southeast corner of the Equitable Life Assurance Company building parking structure with only some 5 feet of Puente material between the crown of the tunnel and the underside of the wall footing. The exact elevation of the bottom of the wall footing will need to be established prior to the start of construction.

At approximately Station 345+00, it is anticipated that the crown of the tunnels will again encounter mixed-face conditions similar to those west of the Vermont Station, prior to arriving at, or egressing from, the east wall of the Normandie Station structure. The soil and ground water conditions here are expected to be similar to those at the west end of the Vermont Station, except that the natural head of water at the crown may be less.

We believe that ground conditions between the Vermont and Normandie Stations are suitable for the use of soft ground tunnelling techniques utilizing an open-face shield with mechanical excavation equipment. Because of the nature of the mixed-face conditions, we do not believe that methods of tunnel construction not employing a shield will be successful through this segment. The mixed-face segments are expected to require fore polling and/or breast boarding techniques within the alluvium to maintain stability of the face, prevent loss of ground and avoid surface settlement along such portions of this alignment.

When entirely within non-weathered Puente materials, the subsurface conditions are expected to be favorable for open-face shield tunnelling methods utilizing suitable excavation equipment. The average unconfined compressive strength of the siltstone, claystone fraction of the Puente Formation is 70 psi. The exception to this average compressive strength could be a few sandstone beds, 1 inch to 3 feet in thickness, which can have unconfined compressive strength ranging from 5,000 to 15,000 psi. The nearby Sacatella Tunnel was driven in the Puente formation and encountered several of these hard beds which could not be excavated with a claw-type excavator (see Appendix E.4). Our borings in Design Unit A220 suggest there are only a few such beds. No significant inflows of water are anticipated within the Puente Formation, where the tunnel crowns are well below the interface with the alluvium. The strike and dip of the bedding planes of the Puente vary appreciably because of the folded nature of this sedimentary formation. The general strike, however, is believed to be approximately east-west, sometimes parallel to the tunnel alignment, sometimes at an acute angle with the alignment. The observed dips are southward, varying between 10° and 55° below the horizontal. Because of the structural orientation of the Puente and the direction of the tunnel alignment, it is possible that the tunnels will encounter single or multiple beds of harder sandstone in the face of the tunnel excavation for certain lengths of the tunnel alignment occurring within the Puente Formation.

7.1.2 Station 349+75 and Station 368+25 (1850 feet, Drawing 3)

The tunnels between the Normandie and Western Stations will encounter alluvial materials consisting entirely of interbedded horizons of saturated cohesive- and cohesionless-like alluvial soils. The depth of cover above the crown varies between 40 feet near the Normandie Station and 35 feet near the Western Station.

The ground water level varies between a few feet above the crown at the west end of the Normandie Station and 15 feet above the crown at the east end of the Western Station. It is anticipated that flowing ground conditions may be encountered between these two stations, assuming no dewatering systems are installed. Below the springline, in general, it is anticipated that more impervious cohesive materials will predominate.

The following conditions were recorded in the noted borings through tunnel horizons.

- Boring 14-1 indicates clayey sand and clayey silt at crown elevation with a minor hydrostatic head. Below the crown and invert, the tunnelling medium consists of sandy clay and clayey sand with varying amounts of sand and clay throughout. Clayey silt exists below the invert to a depth of 76 feet, at which depth the San Pedro Sands are encountered.

- Boring 15-4 indicates sandy clays, silty sands and clayey silts as being present within the zone of the tunnel crown with a hydrostatic head of approximately 15 feet. The potential for flowing ground at this end of this tunnel segment appears more probable than at Boring 14-1. Sandy clay occurs below the zone of the crown extending some 4 feet below the invert. A 2- to 3-foot thick, fairly clean sand 2 1/2 feet below the invert, may be under significant hydrostatic pressure, and the potential for a blow-out at the invert of the tunnels should not be overlooked.

Similar variations in soil stratigraphy and ground water conditions can be anticipated between Borings 14-1 and 15-4.

We believe that the soil conditions between the Normandie and Western Stations are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavation equipment. Because of the 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. The contractor should be prepared to search for, and relieve excessive hydrostatic uplift pressures below tunnel invert to prevent local blow-outs at the invert of the tunnels.

7.1.3 <u>Station 372+50 and Station 400+25 (2775 feet, Drawing 3)</u>

The tunnels between the Western and Crenshaw Stations will encounter alluvial materials consisting of interbedded horizons of saturated cohesive and cohesionless-like materials with a more or less equal distribution of occurrence over the face of the tunnels.

The ground water level varies between approximately 20 feet above the crown of the tunnel at the west end of the Western Station and 5 feet above the crown at the east end of the Crenshaw Station. A comparison with Section 7.1.2, above, indicates that this tunnel segment will also encounter flowing ground conditions, but more probably in the zone between the crown and invert of the tunnels.

Typical soil conditions which may be encountered by the tunnel construction along this segment are noted at the following locations:

- Boring 15-1 indicates sandy clay at the crown, clayey silt and silty sand above and below the springline, and sandy clay to 1 foot below the invert. The San Pedro Sands will occur immediately below the sandy clay.

- Boring 16-6 indicates the crown will encounter sandy clay soil/sands above and below the springline, and interbedded sand, silts and clays through to the invert. It is anticipated that the invert will penetrate the San Pedro Sands.

- Boring 16-5 suggests the crown is expected to pass through interbedded sandy clay, clayey silt and silty sand horizons, the latter of which may flow. From above the springline to the invert, saturated sands are expected to occur in the face of the tunnel excavation. The invert at this location may just be above the San Pedro Sands.

It is pointed out here that the elevation of the surface of the San Pedro Sands (Drawing 3) may vary from that shown between the borings and, therefore, the invert may encounter this sand for longer distances than that shown on the drawing.

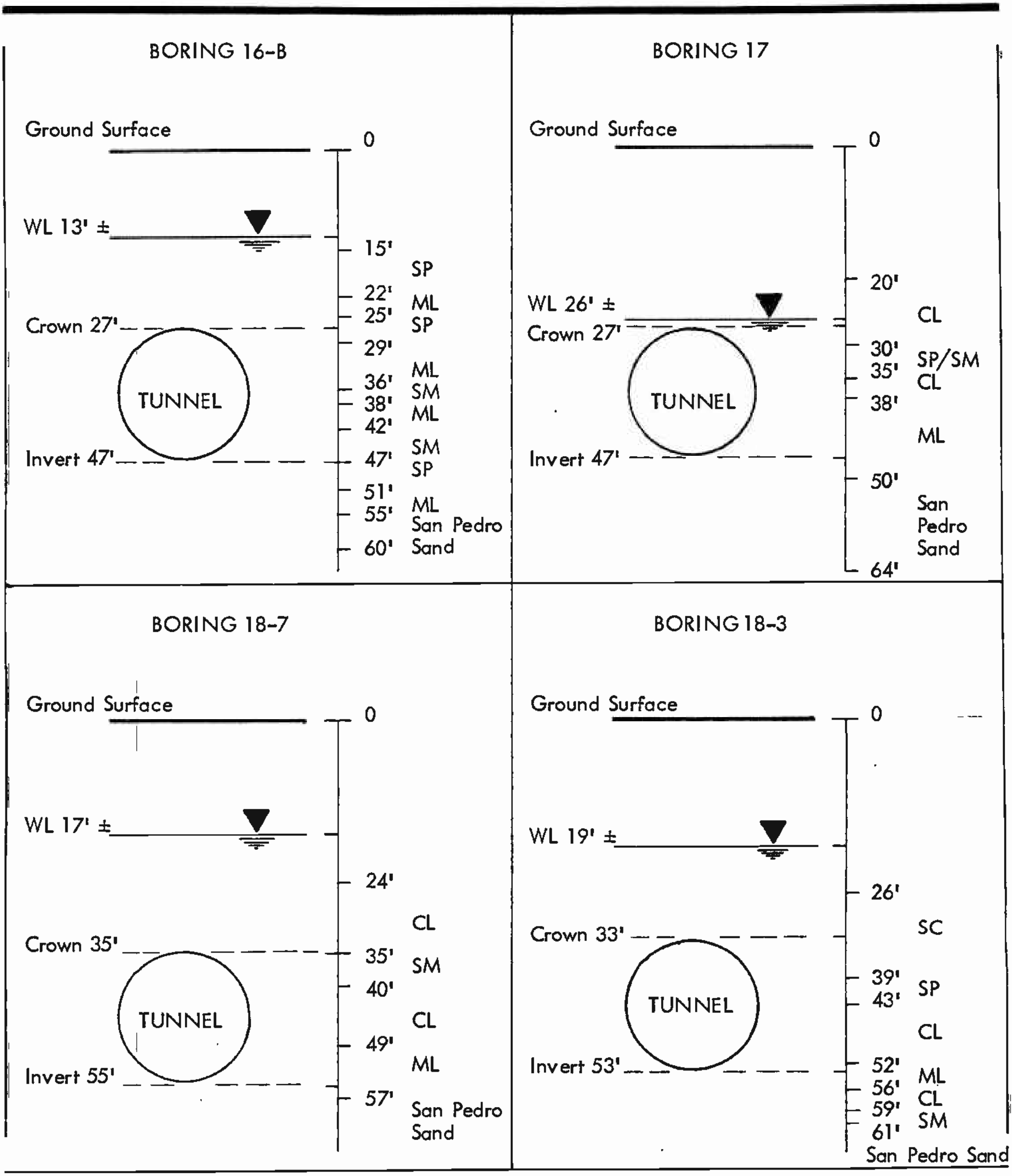
We believe that the soil conditions between the Western and Crenshaw Stations are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavating equipment. We do not believe that tunnelling without a shield would be successful in these soils and ground water conditions described in this segment. Shield tunnelling methods are expected to require means by which the face of the tunnel excavations can be supported. Control of seepage flows may require the installation of dewatering systems ahead of the face of the tunnel excavation - the primary function of which would be to reverse the hydraulic gradients and, therefore, flows to the face and the invert of the tunnel excavation. Grouting of the San Pedro Sand Formation is believed feasible utilizing chemical and/or cement injection methods. However, grouting of the more cohesionless fractions of soils above the San Pedro is expected to be more difficult in most cases because of the significant silt and clay fractions within such soil horizons.

7.1.4 <u>Station 405+75 and Station 474+25 (6850 feet, Drawings 4 & 5)</u>

The tunnels between the Crenshaw and LaBrea Stations will encounter saturated alluvium throughout consisting of interbedded or interlayered horizons of cohesive and cohesionless-like soils.

The ground water level in the alluvium is consistently above the crown of the tunnels varying between 7 feet above at Boring 16-1, 15 feet above at Boring 16-B, 1 foot at Boring 17, and 17 feet above at Boring 18-7. It is conceivable that some flowing ground conditions may be encountered during the construction of this tunnel segment, as suggested by the variable stratigraphic conditions at tunnel grades (see Figure 7-1).

The heterogeneous nature of the tunnelling media not withstanding, we believe that the soil conditions between the Crenshaw and LaBrea Stations are suitable for the use of soft ground tunnelling techniques utilizing a shield with hand and/or mechanical excavating equipment. We do not believe that tunnelling without a shield would be feasible in the soil and ground water conditions described in this segment. Shield tunnelling methods are expected to require means by which the face of the tunnel excavation can be supported. The heterogeneous and non-continuous nature of the alluvial soils suggests that a general dewatering system in the alluvium may or may not be successful. A



EXAMPLES OF STRATIGRAPHIC VARIATIONS

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83-1140

Figure No.
7-1

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similar conclusion would be viable for grouting the more pervious alluvial horizons along this segment. Grouting of the San Pedro Sand Formation is believed feasible utilizing chemical and/or cement methods.

7.2 GROUND WATER - INFLOWS AND MINERAL ANALYSES

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

Ground water inflows from saturated alluvial materials in the entire segment of this tunnel, in our judgement, are likely to be significant inflows with attendant caving problems, based on the performance of man-sized auger Boring Nos. 13A, 15A, 16A, 16B, 17A and 17B (see Appendix A). The ground water inflows/caving conditions are summarized in Table 7-1.

The entire alluvial interval below the water level is considered saturated. Although there are many fine-grained, tight, clay and silt beds, there are several relatively pervious sand horizons that could contribute considerable amount of water into the face of the tunnel excavation. Good examples of this are Boring 16B which recorded an inflow of 50 gpm for the interval 13 to 22 feet, Boring 15A which recorded a 15 gpm inflow for the interval 17 to 22 feet, and Boring 17B which recorded a 5 gpm inflow for the interval 28 to 64 feet.

Mineral analyses of the alluvial ground water from Boring Nos. 14, 16A, 17, 17A and 17B indicate the total dissolved solids (TDS) are less than 1000 parts per million (ppm). This is considered good quality water compared to mineral analyses of bedrock ground water from Boring 16, a sodium chloride type water containing a TDS of 6926 ppm. Ground water originating from the bedrock would be considered corrosive to metals and cement. For details on corrosion, refer to studies performed for SCRTD by Waters Consultants (Professional Services Group, Inc.), San Diego, California.

7.3 ENGINEERING PROPERTIES OF TUNNELLING MATERIALS

The engineering properties of alluvium, San Pedro Sand and Puente bedrock Formation, as applied to tunnelling, are similar to those described in Section 5.5 and in Table 5-2, "Material Properties Selected for Static Design".

Squeezing of Unit C should not be a particular stability problem in normal shield tunnel construction operations because the average unconfined compressive strength is 70 psi. In general, the alluvial material should not squeeze, although there could be a slight tendency for squeezing of local, saturated, cohesive interlayers. Such behavior of the cohesive material should not impede shield tunnelling operations.

7.4 CROSS PASSAGES

Southern California Rapid Transit District Drawings CSK-8 (Sheet 2 of 7) and CSK-9 (Sheet 3 of 7) dated January 12, 1984, indicate 15 cross passages are

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
13A	321	35 - 55	none	26	N/A	none	No caving 0 to 60 ft; slight caving 23 to 27 ft; 5 gpm inflow at 26 ft
15A	374	35 - 55	55 - 60	15	N/A	none	10 to 15 gpm inflow from confined sand layer 17 to 23 ft; San Pedro Sand probably caving from 55 to 60 ft
16A	398	43 - 63	30 - 33	42	914/7.9	slight sulfur odor	2 gpm ± inflow from San Pedro Sand at depth of 69 ft; caving San Pedro Sand 42 to 72 ft
16B	416	27 - 47	13 - 22 55 - 60	13	N/A	none	50 gpm inflow from confined flowing sand layer 15 to 22 ft and San Pedro Sand 55 to 60 ft
17A	435	33 - 53	38 - 42	18	850/7.8	gas	100% LEL by gas detector; 1 gpm from 13 ft, 3 gpm from 26 ft gas caused water to foam
17B	470	35 - 55	flowing ground 56 - 64	18	670/7.9	none	caving from 48 to 64 ft; 5 gpm inflow from 28- to 64-ft interval; San Pedro Sand 56 to 64 ft

planned at tunnel line stations 325+66, 332+16, 338+66, 356+35, 361+96, 380+47, 388+05, 413+03, 420+78, 428+53, 436+26 (vent structure), 443+15, 450+98, 458+80 and 446+63 (see Drawings 2, 3, 4 and 5). According to 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.

Cross passages at Stations 325+66, 332+16 and 338+66 (Drawing 2) will require mining between twin-bore tunnels in siltstone, claystone and sandstone bedrock of the Puente formation (C). This is "soft-ground" tunnelling material, as described in Sections 5.3 and 5.5.5. Bedrock cover over the crown ranges from about 20 to 40 feet. Unit C should stand well with little, if any, caving or slaking that would require bracing, timbers, or rock bolts. Mechanical excavation equipment can excavate this material, possibly assisted by jack-hammers if very hard 1 inch to 3-foot thick cemented interbeds are encountered.

All other cross passages (Drawings 3, 4 and 5) will be excavated in inter-bedded cohesive and cohesionless-like, heterogeneous alluvium (A_3/A_4) below the water table and in ground considered potentially gassy. These cross passages should encounter similar stratigraphic, ground water and tunnelling conditions described in Section 7.1. 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. Based on Boring 17A, gas under pressure should be anticipated in the vicinity of cross passages at Stations 428+53, 436+26 and 443+15.

7.5 GAS, OIL AND FAULTING

The majority of the tunnel line segment in Design Unit A220 should be classified potentially gassy, and the area around Stations 430 to 450 possibly as gassy. These classifications are from the California Administrative Code, Title 8, page 684.18. Appropriate tunnelling equipment should conform with CALOSHA requirements and California Tunnel Safety Orders. For details on gas, refer to studies performed for SCRTD by Engineering Science, Arcadia, California.

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

There are no known faults crossing Design Unit A220 based on a review of published geologic maps and literature. However, because this is California earthquake country, the contractor should anticipate encountering small faults and shear zones. The small faults and shear zones should not impede tunnelling excavation progress to any great extent.

7.6 SHAFTS

A shaft, vent structure, is planned near Wilshire and Mullen Street between Stations 436+26 to 437+56. Criteria and guidelines for the design and construction of shafts are provided in Section 7.6.1.

7.6.1 Shaft Guidelines

The radial effective pressure on shafts, developed by Terzaghi (1943) and Szechy (1970) were used herein for the design of shafts in soft-ground geologic units. Another more recent approach for design of shafts is the method suggested by Prater (1977).

The radial pressure on shafts in soft-ground units will depend on, but is not necessarily limited to, the type of unit, geometry of shaft and method of construction. For current design purposes, the radial pressures acting on vertical shafts, and shafts inclined at less than 10° from the vertical, can be estimated as follows:

° Fine-Grained Alluvium (A_4) and Siltstone/Claystone (C)

Radial pressures can be assumed equal to the at-rest pressure based on effective stress plus the hydrostatic pressure. Thus,

$$\sigma_r = K_o \sigma_s' + \mu$$

where

σ_r = total radial pressure (psf)

K_o = at-rest lateral earth pressure coefficient

- ° A_4 $K_o = 0.5$
- ° Claystone $K_o = 0.4$

σ_s' = effective vertical earth pressure at designated location (psf)

μ = anticipated ground water pressure at designated location (psf)

° Granular Alluvium (A_4) and Siltstone/Sandstone (C)

Theoretical analyses based on methods developed by Terzaghi (1943) and Szechy (1970) indicate the radial effective pressure on shafts in granular soils is nearly equal to the active pressure at shallow depths but approaches a constant pressure at great depths. Radial pressure on shafts can be estimated as:

$$\sigma_r = RK_a \sigma_s' + \mu$$

where:

σ_r = estimated radial pressure

K_a = active lateral earth pressure coefficient

- ° A_1, A_3, A_4, SP $K_a = 0.3$
- ° Siltstone Sandstone $K_a = 0.2$

σ_s' = effective vertical earth pressure at designated location (psf)

μ = anticipated ground water pressure at designated location (psf)

R = reduction factor based on ratio of depth (z) to shaft diameter (D) where (after Mueser, and others, 1967):

z/D	0	1	2	4	6	10
R	1.0	0.9	0.8	0.7	0.6	0.5

Shafts, other than circular shafts, may also be utilized for vent structures. Design of non-circular structures may be based on normal earth pressure values such as recommended for the station structures.

7.7 SPECIAL TUNNELLING PROBLEM AREAS

Due to a high ground water table, relatively shallow cover over the tunnel crown and unknown conditions, research should be performed to establish underground conditions prior to start of construction at the following stations:

- Station 319 to 327 - An east-west trending depression about 50 feet deep by 200 feet wide is located about 300 feet north of the tunnel line (Drawing 2). This depression was located in the former" Bimini Bath" stream channel and has been filled in with Class III landfill. Since this is an old stream channel, it may well be filled with ground water also. The tunnel line should not encounter the landfill/water-filled depression, based on our interpretation of the old U.S. Geological Survey topographic contour map prepared by plane table in 1920 (scale 1"=2000', contour interval 5').

- Station 324 to 325± and Station 331 to 332± - Foundation conditions beneath the existing five-story mid-Wilshire apartment building at 624 Berendo Street and the six-story Evanston apartment building at 630 Kenmore Street should be researched prior to construction.

- Station 337± - The exact elevation of the bottom of the wall footings for the Equitable Life Assurance Company building parking structure needs to be established prior to the start of construction.

7.8 DESIGN FOR EARTHQUAKES

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the Southern California Rapid Transit District (SCRTD) report entitled "Guidelines for Design of Underground Structures", dated March, 1984. Evaluations of the 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.

Section 8.0

Supplementary Geotechnical Services

Section 8.0

Supplementary Geotechnical Services

8.0 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- Additional Field Exploration: Consideration should be given to drilling additional borings at the proposed station location sites where future at-grade structures. These additional borings are for the purpose of verifying the assumption that conditions encountered at the boring locations are applicable to the related station location.

  Due to the lack of data on subsurface materials, ground water conditions, gas and flowing San Pedro Sand along the tunnel alignment, we recommend drilling three borings to obtain samples for laboratory testing and evaluation of tunneling conditions. These borings should be located near Stations 355, 424 and 444.

  We also suggest drilling two additional man-sized auger borings. One at Station 378 (mixed-face invert) and the other at Station 437± (vent structure and cross passage) in order to assess the potential for flowing ground" condition and ground water parameters, in the San Pedro Sand.

- Pump Test: It is recommended that pumping tests be performed at the A220 Station sites to evaluate the pumping and dewatering characteristics. The test well(s) should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

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

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

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

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

- <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.

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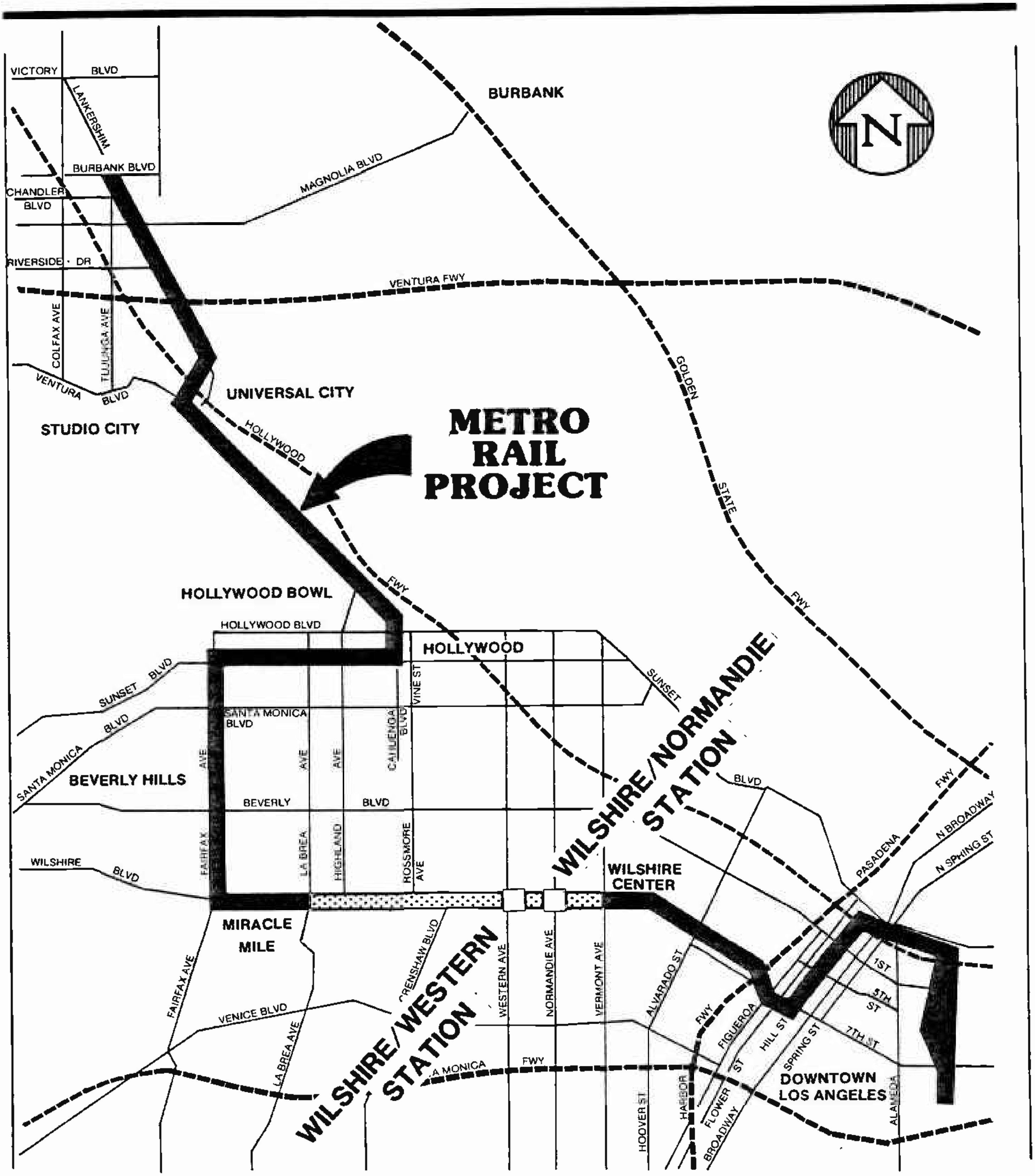


VICINITY MAP

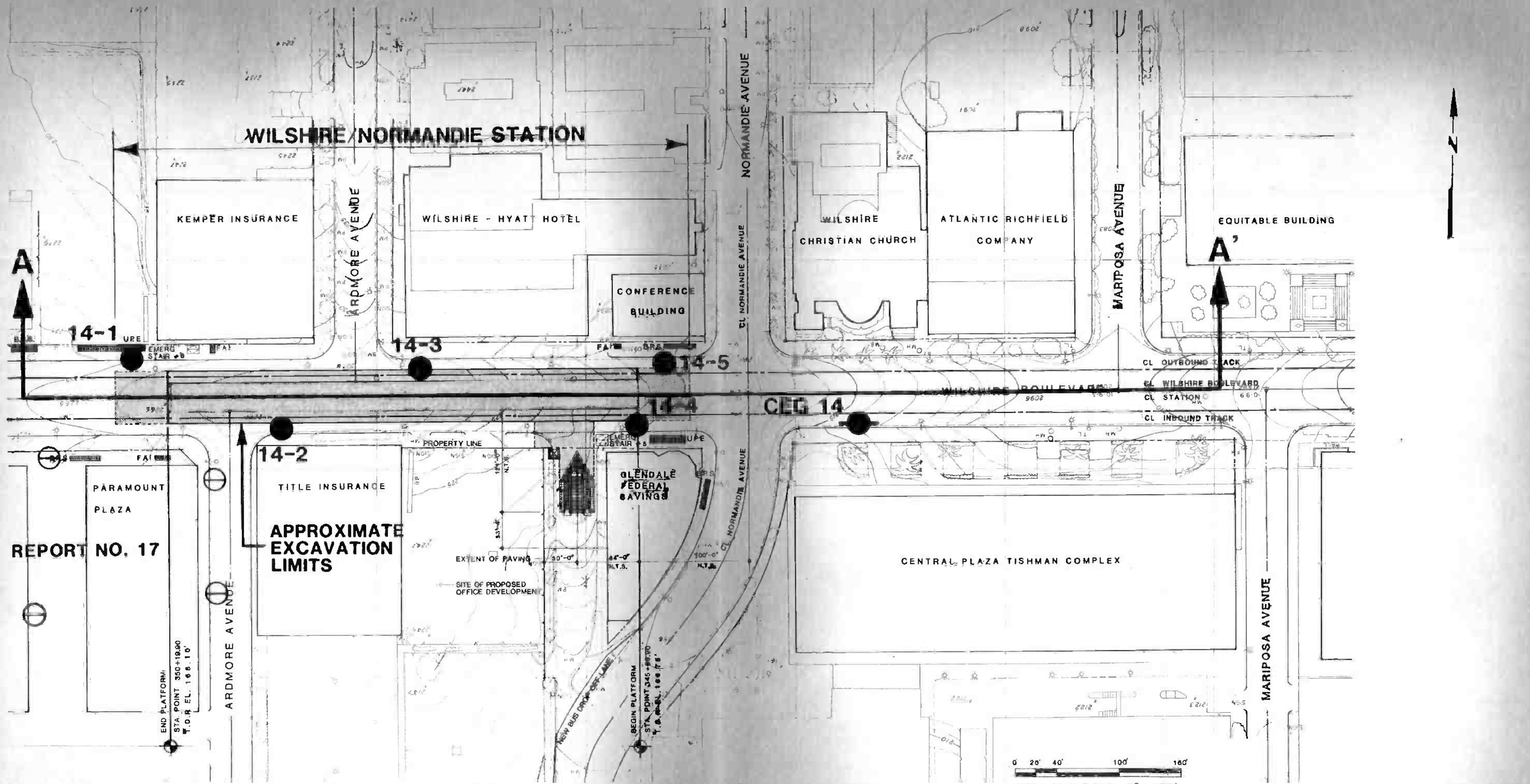
DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No **83-1140**

Drawing No **1**

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication



REF: "PRELIMINARY WILSHIRE/NORMANDIE STATION SITE PLAN", DRAWING #A-28, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40' REDUCED TO 1" = 100', DATED 3-15-83.

NOTES:
1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 7
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 10.

LOCATION OF BORINGS - WILSHIRE/NORMANDIE STATION

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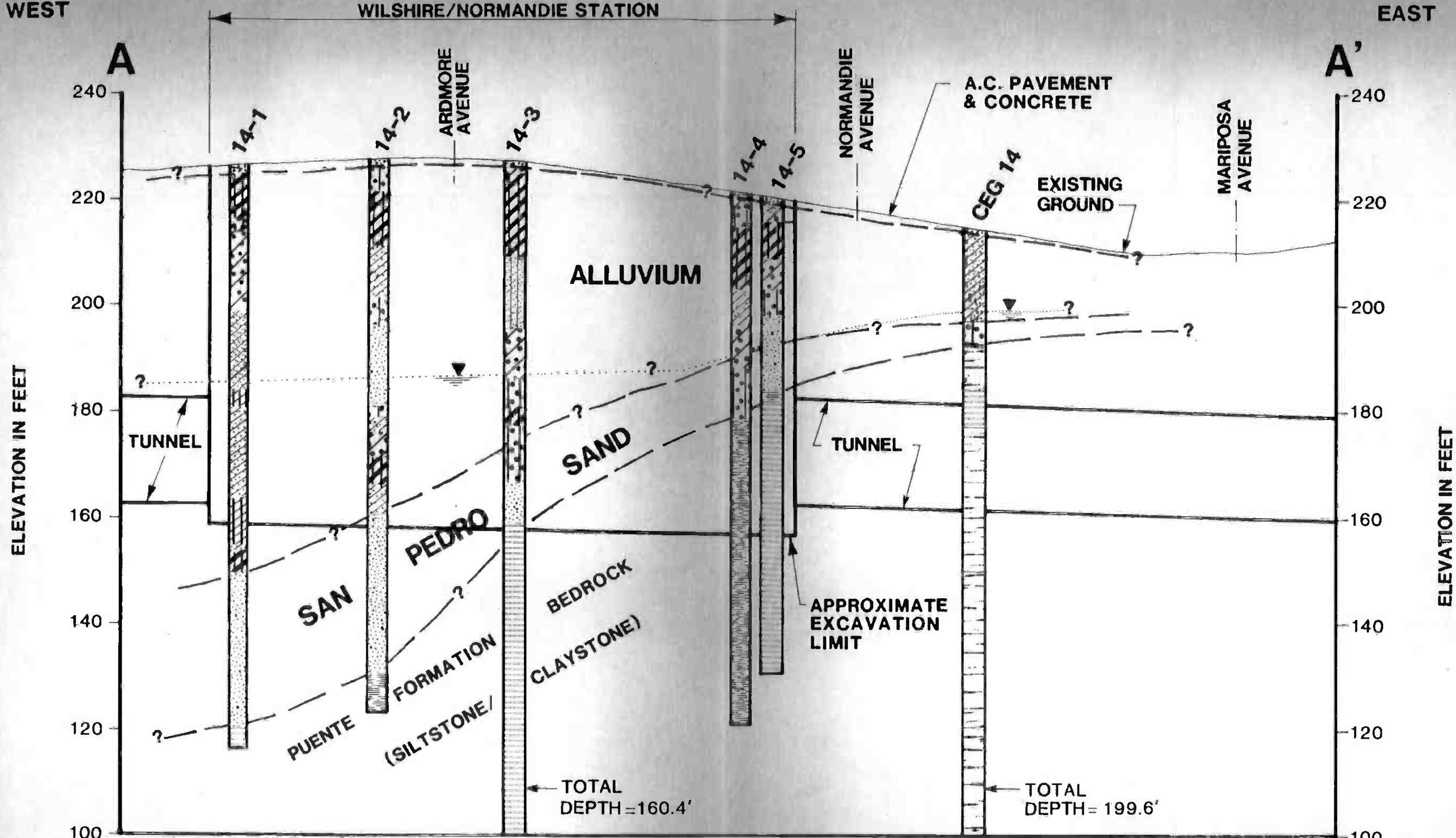
Scale: As Shown	Project No: 83-1140
Date: MAR., 1984	
Prepared by: RG	Drawing No: 6
Checked by: FYC	
Approved By: RMP	



SCALE: HOR. scale feet 100 0 100 200

VER. 20 0 20 40

NOTES:

1.) FOR LOCATION OF SUBSURFACE SECTION A-A' SEE DRAWING NO. 6

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 10

SUBSURFACE SECTION A-A' - WILSHIRE/NORMANDIE STATION

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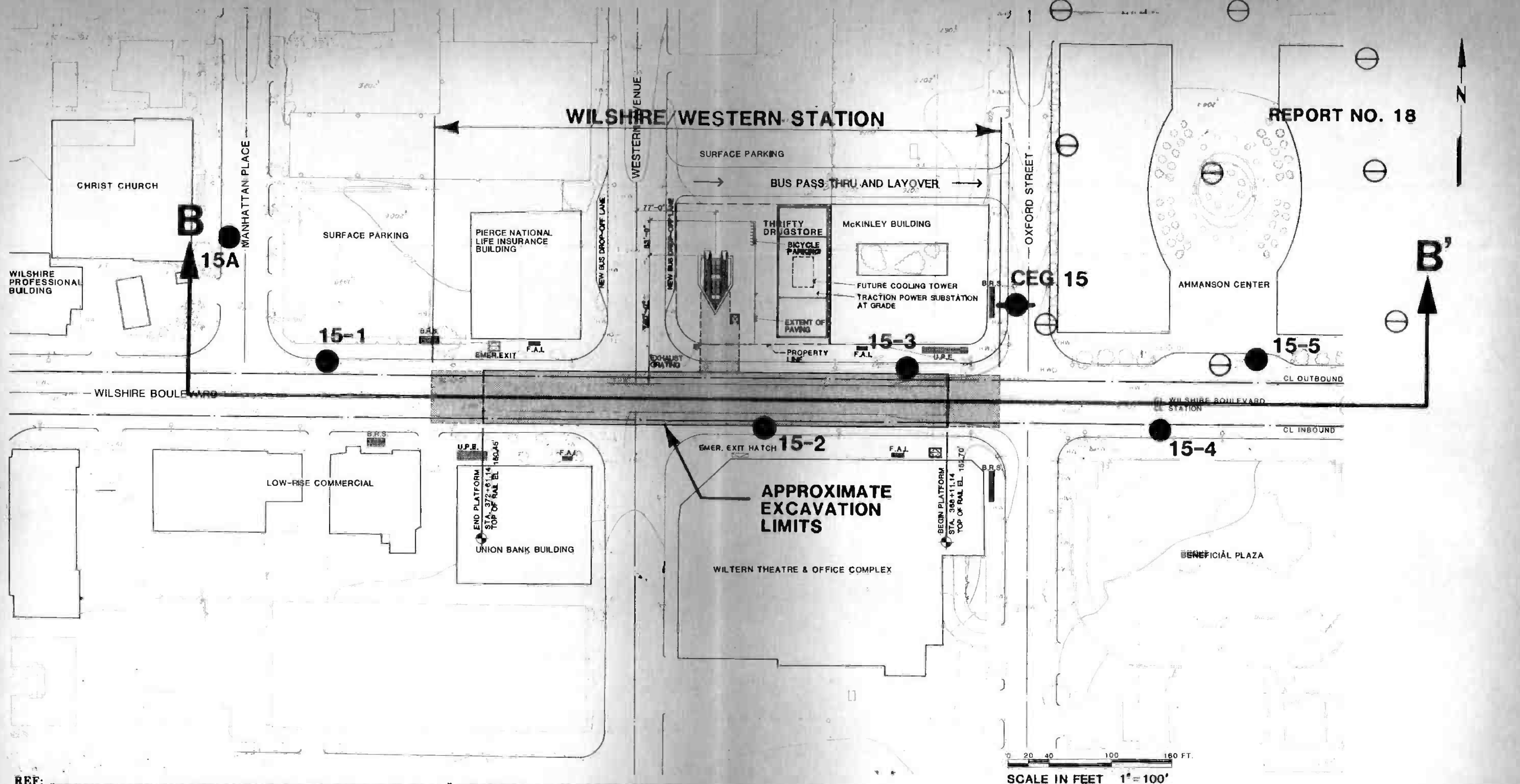
Scale: As Shown	Project No: 83-1140
Date: MAR., 1984	
Prepared by: RG	Drawing No: 7
Checked by: FYC	
Approved By: RMP	



REF: "PRELIMINARY WILSHIRE/WESTERN STATION SITE PLAN" DRAWING #A-31, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40' REDUCED TO 1" = 100' DATED 3-15-83.

NOTES:

1.) FOR SUBSURFACE SECTION B-B' SEE DRAWING NO. 9
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 10

LOCATION OF BORINGS - WILSHIRE/WESTERN STATION

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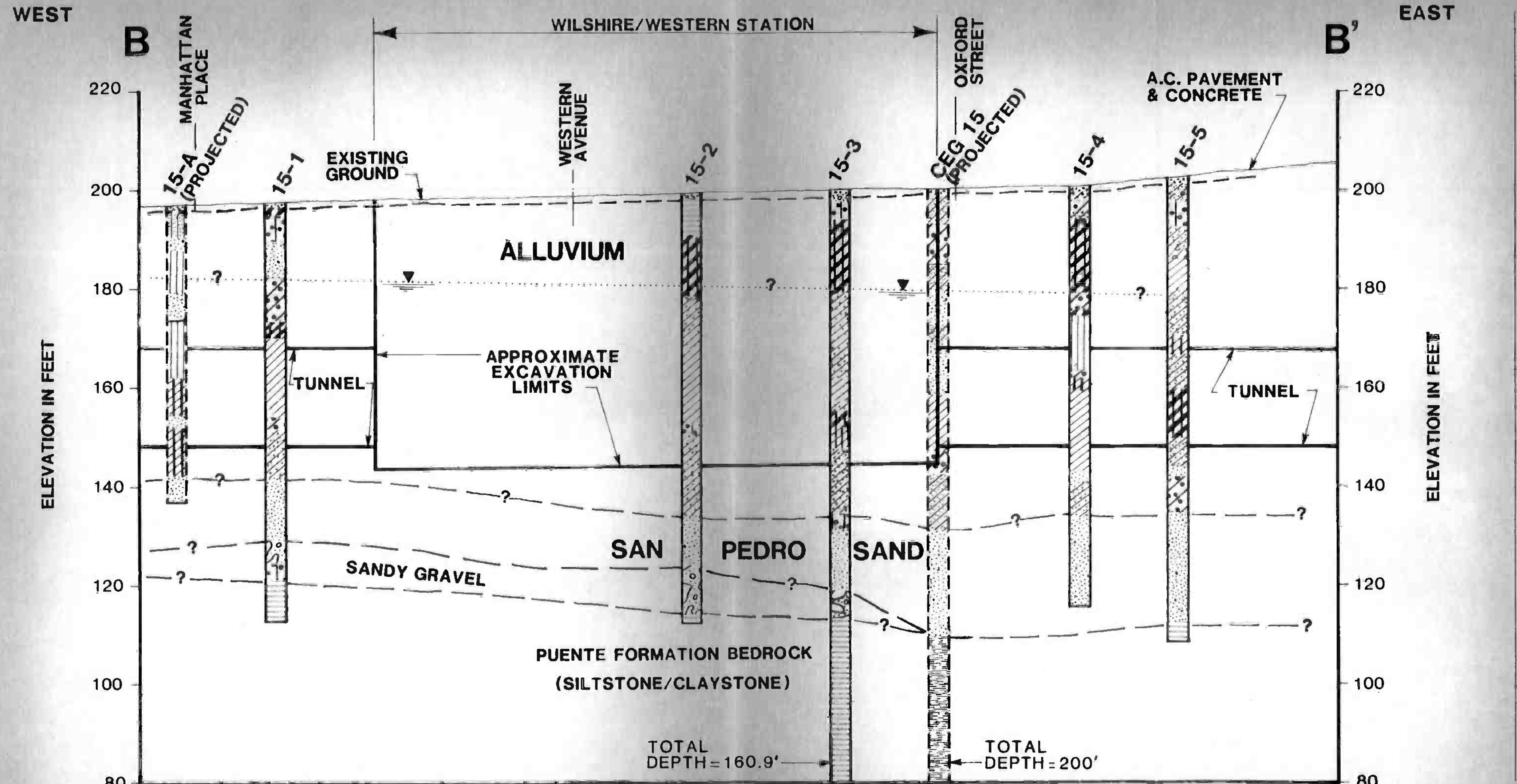
Scale	As Shown
Date	MAR., 1984
Prepared by	RG
Checked by	FYC
Approved By	RMP
Project No	83-1140
Drawing No	8



SCALE: HOR. scale feet 100 0 100 200
VER. 20 0 20 40

NOTES:

1.) FOR LOCATION OF SUBSURFACE SECTION B-B' SEE DRAWING NO. 8
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 10

SUBSURFACE SECTION B-B' - WILSHIRE/WESTERN STATION

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METRO RAIL PROJECT

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Scale	As Shown
Date	MAR., 1984
Prepared by	CSJ
Checked by	FYC
Approved By	RMP
Project No	83-1140
Drawing No	9

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.

QUATERNARY — PLEISTOCENE

- A_3 OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.
- A_4 OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from firm to hard.
- SP SAN PEDRO FORMATION: Predominantly clean, cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium dense to very dense. Locally impregnated with oil or tar.

TERTIARY — PLIOCENE

- C FERNANDO AND PUENTE FORMATIONS: Claystone, siltstone, and sandstone; thinly to thickly bedded. Primarily low hardness, weak to moderately strong. Locally contains very hard, thin cemented beds and cemented nodules.

ROCK TUNNELLING
(Terzaghi Rock Condition Numbers apply)*

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

TERTIARY — MIOCENE

- 2-5 TOPANGA FORMATION: Conglomerate, sandstone, and siltstone; thickly bedded; primarily hard and strong (Geologic symbol Tt).
- 1-5 TOPANGA FORMATION: Basalt; intrusive, primarily hard and strong (Geologic symbol Tb).

TERZACHI ROCK CONDITION NUMBERS:*

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

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

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

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

SYMBOLS

- - - - - ? Geologic contact: approximately located; queried where inferred

U ●●●●● ? D Fault (view in plan): dotted where concealed; queried where inferred; (U) upthrown side, (D) downthrown side

? Fault (view in geologic section): approximately located; queried where inferred; arrows indicate probable movement; attitude in profile is an apparent dip and is not corrected for scale distortion

40 Dip of bedding: from unoriented core samples; bedding attitudes may not be correctly oriented to the plane of the profile, but represent dips to illustrate regional geologic trends; number gives true dip in degrees, as encountered in boring

? Ground water level: approximately located; queried where inferred

- Boring — CEG (1981)
- Boring — CCI/ESA/GRC (1983)
- Boring — Nuclear Regulatory Commission (1980)
- Boring — Woodward-Clyde (1977)
- Boring — Kaiser Engineers (1962)
- Boring — Other (USGS 1977 and various foundation studies)

NOTES:

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

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

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

- SILT
- CLAY
- SANDY SILT
- SANDY CLAY
- CLAYEY SILT
- SILTY CLAY
- SILTY SAND
- CLAYEY SAND
- SAND
- GRAVELLY SAND
- SANDY GRAVEL
- GRAVEL
- GRAVELLY CLAY
- TAR SILT & CLAY
- TAR SAND
- FILL
- SILTSTONE
- CLAYSTONE
- INTERBEDDED SANDSTONE WITH SILTSTONE OR CLAYSTONE
- SANDSTONE
- SANDSTONE, CONGLOMERATE
- CEMENTED ZONE
- META—SANDSTONE
- BASALT
- BRECCIA
- SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	N/A	Project No	83-1140
Date	MAR.,1984		
Prepared by	RG	Drawing No	10
Checked by	JAD		
Approved By	HAS		

Appendix A
Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A220 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, 1983 and 1984 borings drilled for this A220 investigation, and 1983 borings drilled for Design Units A240 and A245. The specific boring logs included are summarized below:

° <u>1981</u>

  CEG-13, CEG-14, CEG-15, CEG-16 and CEG-17

° <u>1983 and 1984 - A220</u>

  13A, 13-7, 13-8
  14-1 through 14-5
  15-1 through 15-5, 15-A
  16B, 17A and 17B

° <u>1983 - A240</u>

  16A, 16-1 through 16-6

° <u>1983 - A245</u>

  18-2 through 18-7

Locations of the borings are shown on Drawings 2 through 5. Ground water observation wells (piezometers) were installed in borings listed in Section 5.4 (Table 5-1). Geophysical downhole and crosshole surveys were made for the 1981 investigation at Borings CEG-14 and CEG-15 (see Appendix B).

The borings were drilled to depths generally ranging from 60 to 200 feet, and penetrated through the alluvium into the underlying San Pedro sand or bedrock. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was generally good in both the siltstone and claystone bedrock and the alluvium.

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

A.2 FIELD STAFF AND EQUIPMENT

A.2.1 <u>Technical Staff</u>

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

A.2.2 Drilling Contractor and Equipment

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

A.3 SAMPLING AND LOGGING PROCEDURES

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

A.3.1 Sampling

In the overburden at about 10-foot intervals, the Converse ring sampler was driven using a down-hole 450-pound slip-jar hammer. The Converse sampler was followed with the standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the Puente Formation was encountered, the borings were sampled using a Pitcher Barrel and Converse ring sampler at 20-foot intervals.

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

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

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

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

A.3.2 Field Classification of Soils

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

Table A-1 shows the correlation of standard penetration information and the physical description of the consistency of clays (hand-specimen) and the compactness of sands used by the field geologists for describing the materials encountered.

TABLE A-1 Correlation of N-Values and Consistency/Compactness of Soil Obtained in the Field

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

A.3.3 Field Description of the Formations

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

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

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

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

Bedrock description terms used on the boring logs are given on Table A-2.

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

TABLE A-2 Bedrock Description Terms

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

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

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

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

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

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

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings 14-1, 14-3, 15-1 and 15-3 located at the Wilshire/Normandie and Wilshire/Western Station sites. Procedures for piezometer installation were as follows:

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

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

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

BORING LOG 13

Proj: DESIGN UNIT A 220 **Date Drilled** 1-30-81/2-1-81 **Ground Elev.** 249'

Drill Rig FAILING 1500 **Logged By** STEPHEN TESTA **Total Depth** 200.0'

Hole Diameter 5" **Hammer Weight & Fall** 140 lb - 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT:				started drilling at 9:30, augered to 6.5'
	CL	0.2-3.0 SANDY CLAY: light olive brown, mostly fines, with some fine sand moist; mottled; medium stiff				
2						
	SP	3.0-20.0 SAND: olive grey, mostly fine to medium sand, moist; trace of fines				
4						
		at 5.0: moderate yellowish brown fine to coarse sand, trace of fine gravel; dry				
6						
				RD		drove 8.5' ./5" casing
8						
10		10.0 moist; very dense		20	SS	Spt at 10.0', 1.5/1.5 recovery
			J-1	30		
				34		
12					RD	
		gravelly from 13.0 to 13.5'				moderate rod chatter from 13.0 to 13.5'
14						
		dusky yellow, mostly fine to coarse sand, with trace of fines and fine gravel; mottled; moist; very dense		22	SS	Spt at 15.0', 1.0/1.0 recovery
			J-2	50		
16	▽				RD	groundwater level ▽ at 16.0' (2-2-81)
18						
	SM					
20						

Project DESIGN UNIT A 220 Date Drilled 1-30-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	20.0-30.5 SILTY SAND: dusky yellow; mottled; mostly fine sand with some fines, trace of organic material; moist; very dense	J-3	21 24 32	SS	(continued) Spt at 20.0',1.5/1.5 recovery; CCI sample at 21.5'
				45 50	DR	
22					RD	minor rod chatter from 22.5-25.0'
24						
		dark yellowish brown and dark yellowish orange; mottled;moist very dense	J-4	19 33 31	SS	Spt at 25.0',1.0/1.5 recovery
26					RD	
28						
30		WEATHERED PUENTE FORMATION 30.5-33.0 CLAYSTONE: dark yellowish brown mostly fines; trace of fine sand; mottled;moist: firm; iron stained laminae		12 8 21	SS	Spt at 30.0',1.0/1.5 recovery
32	45				RD	
		PUENTE FORMATION 33.0-200.0 CLAYSTONE:Wavy, parallel, thin to medium laminae of olive grey; brownish black; greyish brown claystone; subordinate light olive grey; friable fine grained sands tone; and dark yellowish brown siltstone; occasional thin gypsum laminae micaceous; moist				slight change in drilling resistance at 33.0'
34	40			.PB		
36						
38	35-40	PHYSICAL CONDITION: massive; soft to low hardness, plastic to weak strength; fresh	Box 1			
40	20-30					
42	20-30					
44						

Sheet 2 of 9

Project DESIGN UNIT A220 Date Drilled 1-30-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		33.0-200-0 CLAYSTONE: (continued) wavy parallel, alternating very thin to medium laminae of claystone; siltstone and sandstone to 46.4'	Box 1		PB	(continued)
46						
48	30-40	47.5-50.0' olive grey micaceous claystone with wavy discontinuous very thin to thin fine sandstone laminae	Box 2			
50		PHYSICAL CONDITION: massive soft to low hardness; plastic to weak strength; fresh	S-1			
52	30-40					
54			Box 2			
56						
58	20-30					
60	15-20		Box 3			
62		wavy parallel, alternating very thin to medium laminae; mostly of claystone; fine sandstone and siltstone				
64						
66			S-2			
68			Box 3			

Sheet 3 of 9

Project DESIGN UNIT A220 Date Drilled 1-30-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		33.0-200-0 CLAYSTONE: (continued) wavy parallel, alternating very thin to medium laninae of micaceous claystone; fine sandstone and siltstone	Box 3		PB	(continued)
70	30-35	PHYSICAL CONDITION: massive soft to low hardness; plastic to weak strength; fresh	Box 4			
72	30-40	mostly olive gray; claystone with wavy discontinuous very thin to thin fine sandstone laminae				
74						
75	30-40					
76						
78	30-40		Box 5			
80			S-3			
82						
84			Box 5			
86						
88						
90	30-35		Box 6			
92						

Project DESIGN UNIT A220 Date Drilled 1-30-81/1-31-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		33.0 CLAYSTONE: (continued) from 72.5 mostly olive grey; micaceous claystone; with wavy, very thin to medium fine sandstone laminae	Box 6		PB	(continued)
94		PHYSICAL CONDITION: massive; soft to low hardness; plastic to weak strength; fresh				stopped drilling at a depth of 95.0' at 5:00
96			S-4			resumed drilling at 7:00 a.m. at a depth of 95.0', clear day
98	20-30	bivalves at 98.2	Box 6			
100	30-35					
102		alternating very thin to medium laminae of claystone; fine sandstone and siltstone	Box 7			
104						
106	10-20					
108			S-5			
110		well cemented fine grained sandstone at 110.5'	S-6	RD		225 psi; due to refusal at 111.o', put on tri-cone bit and rotary drilled to 112.5
112			Box 7	PB		
114	30					
116			Box 8			

Project DESIGN UNIT A220 Date Drilled 1-31-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		33.0-200-0 CLAYSTONE: (continued) wavy, parallel, alternating very thin to medium laminae of micaceous claystone with subordinate fine sandstone and siltstone			PB	(continued)
118	30					
			Box 8			
120		continued to 121.5; then primarily olive grey; micaceous claystone with very thin to medium laminae with fine sandstone				
122		PHYSICAL CONDITION: (continued) massive, soft to low hardness; plastic to weak strength; fresh				
124			S-7			
126	25-30		Box 8			
128						
130						
	40	alternating claystone; fine sandstone and siltstone	Box 9			
132						
134						
136	30					
138			S-8			
140						

Project DESIGN UNIT A220 Date Drilled 1-31-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		33.0-200-0 CLAYSTONE: (continued) mostly olive grey, micaceous claystone, with very thin to medium fine sandstone laminae			PB	(continued)
	30°					
142	30°	continued claystone				200 psi; gas check 0.0 % LEL (no gas encountered)
144			Box 10			
	30-40					
146						
148						
150	40	PHYSICAL CONDITION: massive, soft to low hardness; plastic to weak strength; fresh	Box 11			
152			S-9			
154						
156	40					
158		at 157.5' mostly olive grey micaceous claystone with wavy fine sandstone laminae				
160						
162	35		Box 12			
164						

Project DESIGN UNIT A 220 Date Drilled 1-31-81/2-1-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164		33.0-200.0 CLAYSTONE: (continued) 165 to 167-5			PB	(continued)
	30-40	alternating very thin to medium laminae of claystone, fine sandstone and siltstone PHYSICAL CONDITION: massive; soft to low hardness; plastic to weak strength; fresh	Box 12			
166						
168			S-10			
170	30-40					
			Box 12			
172		at 172.5 mostly micaceous claystone with wavy discontinous fine sandstone				
	20-35					
174						
176	20-25					
			Box 13			
178	20-25					
180	20-30	at 180.0 alternating very thin to medium laminae of claystone; fine sandstone and siltstone				Stopped drilling at 180.0' 2-1-81, resumed drilling at 7 a.m. from 180.0', clear day
182						
184			S-11			
186	45-50		Box 14			
188						

Project DESING UNIT A220 Date Drilled 2-1-81 Hole No. 13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		33.0-200.0 CLAYSTONE: (continued) wavy, parallel, alternating very thin to medium laminae with micaceous claystone; fine sandstone and siltstone PHYSICAL CONDITION: previously described			PB	(continued)
190	45 50		Box 14			
192	45					
194						
196			Box 15			
198			S-12			Terminated Hole at 200.0' at 11:30
200		B.H. Terminated at 200.0'				conducted water pressure test from 7-9:30 a.m., reamed hole out from 5.0" to 7.0"; installed 100.0' of 4' PVC and grouted.
202						
204						
206						
208						
210						
212						

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BORING LOG 13A

Proj: DESIGN UNIT A-220 Date Drilled 11-9-83 Ground Elev. 248'

Drill Rig MAN-SIZE AUGER Logged By J. Stellar Total Depth 60'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		A/C PAVEMENT 0.0-0.8				Hole stood well in general. 3' of belling @ 23'-27' due to seepage of perched water
	ML	ALLUVIUM 0.8-4.0 CLAYEY SILT: light brown, slightly moist, stiff, numerous roots				
4	ML	4.0-11.0 SILT: light brown, slightly moist, stiff, with layers of clayey silt				
11	SP	11.0-13.0 GRAVELLY SAND: light green, moist, medium dense, gravel to ½"				
13	SP	13.0-19.0 SAND: light brown to orange brown, very moist, medium dense				
19	SP/SM	19.0-21.4 SAND: light green, wet, medium dense w/ layers of silty sand				

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Project DESIGN UNIT A-220 Date Drilled 11-9-83 Hole No. 13A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP/SM	SAND: (continued)				
22	SM	21.4-29.0 SILTY SAND: light brown, wet, fine to medium sand				
24						
26		6" layers of sandy silt				5± g.p.m. seep at 26', 3' belling 23'-27', most H2O coming from southwest
28						
30		PUENTE FORMATION 29.0-60.0 SILTSTONE: mottled light brown to reddish brown, low hardness, closely laminated siltstone with sandy siltstone interbeds, weak strength strike dip bedding N 70°E 25°S				
32						
34						
36						
38		becomes blue gray with low hardness, weak strength				
40						
42						
44						

Project Design Unit A-220 **Date Drilled** 11-9-83 **Hole No.** 13A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44		PUENTE FORMATION 29.0-60.0 SILTSTONE: (continued)				
50						Bag Sample 50' - 53'
52		3"-4" sandy silt-stone interbeds becomes hard (silicious)				
60		B. H. 60.0' Terminated hole. Cased hole to 40'. Don Rose (Tudor) Downhole J. Stellar On Site Don Rose, Don Croft, Keith Bull — Tudor Frank McLean (MRTC)				

Sheet 3 of 3

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BORING LOG 13-7

Proj: DESIGN UNIT A-220 | Date Drilled 3-16-84 | Ground Elev. ____

Drill Rig Failing 1500 | Logged By M. Schluter | Total Depth 70.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.8 A.C. PAVEMENT			C	start drilling @ 8:50
	ML	0.8-2.5 CLAYEY SILT: olive black; some sandy silt; moist; soft to very soft			A	
2	CL	2.5-8.0 SANDY CLAY, SILTY CLAY: dark yellowish brown; moist; firm; slightly porous; increasing sand content	C-1	6 17	DR 325	
4					RD	rotary wash, drag bit
6			J-1	6 14 18	SS	1.5/1.5
8	SC/CL	8.0-12.5 CLAYFY SAND/ SANDY CLAY: Moderate yellowish brown and light olive grey; moist; loose to medium dense/soft to firm			RD	
			C-2	7 11	SS	
10	SC				RD	
12	SW	12.5-15.5 SAND/GRAVELLY SAND: dark yellowish brown; moist; medium dense to dense; gravel lenses	C-3	22 32	DR	top ring disturbed
14					RD	
16		PUENTE FORMATION 15.5-70.0 INTERBEDDED SILTSTONE/CLAYSTONE light olive grey & light brown & olive grey; moist; thinly to very thinly bedded; occasional cemented layers; micaceous	J-2	4 10 18	SS	1.3/1.5
18					RD	
20						

Project DESIGN UNIT A-220 Date Drilled 3-16-84 Hole No. 13-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20		Physical Condition: little fractured to massive; very soft to soft; plastic to friable; moderate weathering	C-4	18 29	DR	
					RD	
22						
			C-5	41 59	DR	
24					RD	
26		thinly to medium bedded; occasional very thinly bedded to laminated interbeds	J-3	25 26 50-5"	SS	1.5/1.5 SPT refusal @ 17"
28					RD	
			C-6	38 60	DR	
30	34	very thinly bedded; occasionally laminated			RD	
32						
	30	little weathering to fresh; soft to moderately hard; massive; moderately strong	C-7	36 50-4	DR	CCI refusal @ 10"
34	34				RD	
36		dark greenish grey; olive black; thinly to medium bedded; fresh; moderately hard; massive; moderately strong; slightly moist	J-4	12 28 50-3"	SS	1.2/1.3 SPT refusal @ 15"
38					RD	
			C-8	61 50-2"	DR	CCI refusal @ 8"
40					RD	
42						
44						

Project DESIGN UNIT A-220 Date Drilled 3-16-84 Hole No. 13-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		15.5-70.0 INTERBEDDED SILTSTONE/CLAYSTONE continued	C-9	53	DR	CCI refusal @ 9"
				50-3"		
					RD	
49.5			C-10	80	DR	CCI refusal @ 8"
				50-2"		
					RD	
54.5			C-11	77	DR	CCI refusal @ 8"
				50-2"		
					RD	
59.5	34	siltstone grades to a fine sandstone, micro fossil shells, diatoms, broken and fragmented to intact, thinly to very thinly bedded	C-12	41	DR	CCI refusal @ 9"
60	33			50-3"		
					RD	
64.5	30		C-13	80	DR	CCI refusal @ 7"
				50-1"		
					RD	
68						

Sheet 3 of 4

Project DESIGN UNIT A-220 Date Drilled 3-16-84 Hole No. 13-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		15.5-70.0 INTERBEDDED SILTSTONE/CLAYSTONE continued				
	33		C-14	64 50-3"	DR	CCI refusal @ 9"
70		END OF BORING 70.0' Filled hole with 3 sac/65 gallon cement slurry.				finished boring @ 2:55
72						
74						
76						
78						
80						
82						
84						
86						
88						
90						
92						

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BORING LOG 13-8

Proj: DESIGN UNIT A-220 **Date Drilled** 3-17-84 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** M. Schluter **Total Depth** 80.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.4 A.C. PAVEMENT			C	start drilling @ 8:00
	SM	0.4-3.0 SILTY SAND: yellowish grey, light olive grey; moist; loose to medium dense			A	
2						
	SP	3.0-13.0 SAND: yellowish grey, dusky yellow; medium dense; moist	C-1	17 30	DR	
4					RD	rotary wash, drag bit
6			J-1	12 22 40	SS	1.0/1.5
8					RD	
			C-2	10 19	DR	
10					RD	
12						
	31	PUENTE FORMATION 13.0-49.0 INTERBEDDED SILTSTONE AND CLAYSTONE: light olive grey; very thinly bedded; occasional cemented layers	C-3	18 21	DR	
14					RD	
16		Physical Condition: little fractured to massive; friable to low hardness; weak to mod. strong; moderate weathering	J-2	8 15 32	SS	
18					RD	
20	24		C-4	11 70	DR	

Sheet 1 of 4

Project DESIGN UNIT A-220 Date Drilled 3-17-84 Hole No. 13-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20		13.0-49.0 INTERBEDDED SILTSTONE AND CLAYSTONE: dark greenish grey & olive grey; very thinly to thinly bedded			RD	
22		dusky yellowish brown & olive grey; moderate to little weathering; moderately hard; massive; moderately strong				
			C-5	70 50-	DR 5"	
24					RD	
26			J-3	15 51	SS	1.0/1.0 SPT refusal @ 12"
					RD	
28	35	olive grey & light olive grey; thinly to medium bedded	C-6	35 50-	DR 5"	CCI refusal @ 11"
30					RD	
32						
34	32	dark greenish grey; fresh; massive; moderately hard; moderately strong	C-7	53 50-	DR 3"	CCI refusal @ 9" casing sinking fluid erosion reduced skin friction; installed additional casing to 7'
36					RD	
38						
	32		C-8	59 50-	DR 4"	CCI refusal @ 10"
40					RD	
42						
44						

Sheet 2 of 4

Project DESIGN UNIT A-220 Date Drilled 3-17-84 Hole No. 13-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		13.0-49.0 INTERBEDDED SILTSTONE AND CLAYSTONE: continued	C-9	56 50-	DR 4"	CCI refusal @ 10" added additional 8' of casing - 15' total
					RD	
46						
		thinly to medium bedded; fresh; massive; moderately hard; moderately strong				
48						
		49.0-54.0 SILTSTONE WITH INTERBEDDED SANDSTONE	C-10	45 50-	DR 4"	CCI refusal @ 10"
50					RD	
52						
54		54.0-80.0 SILTSTONE: thinly to medium bedded	C-11	67 50-	DR 2"	CCI refusal @ 8"
					RD	
56						
58						
	33	very thinly to thinly bedded	C-12	48 50-	DR 3"	CCI refusal @ 9"
60		60.5 cemented layer 4" thick			RD	slight drill rig chatter
62						
64			C-13	57 70-	DR 2"	CCI refusal @ 8"
					RD	
66						
68						

Project DESIGN UNIT A-220 Date Drilled 3-17-84 Hole No. 13-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		54.0-80.0 SILTSTONE: continued			RD	
			C-14	69 50-	DR 3"	CCI refusal @ 9"
70					RD	
72						
74		fresh; massive; thinly to medium bedded; moderately hard; moderately strong	C-15	61 75-	DR 3"	CCI refusal @ 9"
					RD	
76						
78						
			C-16	97 50-	DR 1.5"	CCI refusal @ 7.5"
80		END OF BORING 80.0' Filled hole with 3 sac/70 gallon cement slurry				Finished drilling @ 2:15
82						
84						
86						
88						
90						
92						

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BORING LOG 14

Proj: DESIGN UNIT A-220 Date Drilled 1/27-30/81 Ground Elev. 199.5'

Drill Rig Failing Logged By Gallinatti Total Depth 199.6

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-1.0 CONCRETE			AD	Begin drilling 2:00 1/27/81
		ALLUVIUM				
	CL	1.0-17.0 SANDY CLAY: dark yellowish brown; some fine to medium sand; damp; soft				Auger down to 8'
8		8.0 color change to olive grey			RD	
10			J-1	3 3 4	SS	1.3/1.5 recovery
12					RD	
15			J-2	7 12 15	SS	1.3/1.5 recovery
17					RD	
	SM	SAN PEDRO FORMATION 17.0-21.5 SILTY SAND: light brown; some fines; mostly fine sand; loose; moist to wet				
20						

Sheet 1 of 9

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SM	17.0-21.5 SILTY SAND: cont.			DR	No Recovery
			J-3	13	SS	0/1.5 recovery
22	ML	WEATHERED PUENTE FORMATION 21.5-30.0 CLAYEY SILTSTONE: light brown; moist		15		
				13		
					RD	
24						
			C-1		DR	1.0/1.0 recovery
26		sample: many oxide stained fracture surfaces	J-4	6	SS	1.5/1.5 recovery
				7		
				11		
28					RD	
30		PUENTE FORMATION 30.0-37.3 SILTSTONE with CLAYSTONE INTERBEDS: pale brown siltstone; dark mod. brown clay; damp Physical Condition: massive; low hardness; friable; fresh	J-5	40	SS	0.7/1.0 recovery
				55		
					RD	
32			Box #1		PB	1.6/2.8 recovery
34	50°					
						2.6/2,8 recovery
36	40°					
38		37.3-60.8 SILTSTONE: dark yellowish brown; damp Physical Conditions: massive; low hardness; friable; fresh				2.8/2.8 recovery pocket penetrometer >4.5 tsf
40						2.5/2.8 recovery
42						
44			S-1			

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44		PUENTE FORMATION 37.3-60.8 SILTSTONE: cont.	S-1		PB	2.8/2.8 recovery
46			Box #2			2.3/2.8 recovery
48						
		51-60' interbeds of claystone and silty sandstone				2.6/2.8 recovery pocket penetrometer >4.5 tsf
50						
52						
54						2.7/2.8 recovery
56			Box #3			
58			S-2			2.8/2.8 recovery
60			S-3			1.0/1.0 recovery 60' - hard cemented siltstone
		60.8-64.0 SILTSTONE: greyish brown; cemented; dry Physical Conditions: massive; hard; strong; little weathered			RD	
62						
64		64.0-166. SILTSTONE: dark yellowish brown; damp Physical Conditions: massive; low hardness; friable; fresh				
66			Box #3 cont.		PB	2.7/2.8 recovery
68						

Sheet 3 of 9

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68		PUENTE FORMATION 64.0-166.0 SILTSTONE: cont.	Box #3		PB	1.8/2.8 recovery
70						
	40°	71.0' - thin silty sandstone lens				2.8/2.8 recovery
72						
74						2.7/2.8 recovery
			Box #4			pocket penetrometer > 4.5 tsf
76		Physical Conditions: massive; low hardness; friable; fresh				2.8/2.8 recovery
78						
			S-4			2.8/2.8 recovery
80						
82		82.0-88.0' thinly bedded claystone	Box #4 cont.			2.7/2.8 recovery
	40°					
84						
	40°					2.7/2.8 recovery pocket penetrometer > 4.5 tsf
86						
			Box #5			
88						2.8/2.8 recovery
90						2.8/2.8 recovery
92						

Project DESIGN UNIT A-220 Date Drilled 1-28-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		PUENTE FORMATION 64.0-166.0 SILTSTONE: cont.	Box #5		PB	
						2.7/2.8 recovery
94		Physical Conditions: massive; low hardness; friable; fresh				
						pocket penetrometer > 4.5 tsf
96			S-5			2.8/2.8 recovery
98			Box5			
			Box #6			2.8/2.8 recovery
100						
102						2.8/2.8 recovery
		103.0-113.0' thinly bedded claystone				
104						pocket penetrometer > 4.5 tsf 2.8/2.8 recovery
106						
	40°					2.7/2.8 recovery
108			Box #7			
110						.2/.2 recovery (hard cemented zone) 2.2/2.8 recovery
	40°					
112						
			S-6			2.8/2.8 recovery
114						
116			Box 7			

Sheet 5 of 9

Project DESIGN UNIT A-220 Date Drilled 1-28-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		PUENTE FORMATION 64.0-166.0 <u>SILTSTONE</u>: cont.	Box #7		PB	pocket penetrometer > 4.5 tsf 2.8/2.8 recovery
118		<u>Physical Conditions</u>: massive; low hardness; friable; fresh				2.8/2.8 recovery
120						
122			Box #8			1.9/2.8 recovery
124						2.8/2.8 recovery
126						126.8' stop drilling 1/28/81;begin drilling 1/29/81-raining all day 2.8/2.8 recovery pocket penetrometer > 4.5 tsf
130			S-7			2.8/2.8 recovery
132						2.8/2.8 recovery
133	35°		Box #8 cont.			
134						
135			Box #9			
136						2.8/2.8 recovery
138						2.8/2.8 recovery
140						

Project DESIGN UNIT A-220 **Date Drilled** 1-29-81 **Hole No.** 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		PUENTE FORMATION 64.0-166.0 SILTSTONE: cont.	Box #9		PB	pocket penetrometer > 4.5 tsf 2.8/2.8 recovery
		Physical Conditions: massive; low hardness; friable; fresh				
142						
144		144.0-164.0 occasional claystone layers and thin silty sandstone layers	Box #10			2.8/2.8 recovery
146			S-8			2.8/2.8 recovery
148						
			Box #10 cont.			2.8/2.8 recovery
150	40°					
152		becoming more closely inter-bedded				2.8/2.8 recovery pocket penetrometer > 4.5 tsf
154		mostly thin siltstone layers, with some claystone layers and sandstone layers				2.6/2.8 recovery
156	40°		Box #11			
158						2.8/2.8 recovery
160	40°					2.8/2.8 recovery
162						
164			S-9			

Project DESIGN UNIT A-220 Date Drilled 1-29-83 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
164		PUENTE FORMATION 64.0-166.0 SILTSTONE: cont.	S-9		PB	1.8/2.8 recovery pocket penetrometer >4.5 tsf possible contact between Puente & the Fernando Formations
166		166.0-199.6 SILTSTONE: dark yellowish brown; damp; occasional thin claystone layers medium bedded Physical Conditions: massive; moderately hard; weak; fresh	Box #11 cont.			2.7/2.8 recovery
168						2.1/2.8 recovery
170			Box #12			
172						1.5/2.8 recovery
174	40°					2.8/2.8 recovery
176						pocket penetrometer >4.5 tsf
178						2.8/2.8 recovery
180			S-10			1.5/2.8 recovery
182			Box12			2.8/2.8 recovery
184		184.0-199.6' claystone thin beds	Box #13			
186	40°					2.6/2.8 recovery
188						

Project DESIGN UNIT A-220 Date Drilled 1-29-81 Hole No. 14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
188		166.0-199.6 <u>SILTSTONE</u>: cont.	Box #13		PB	2.4/2.8 recovery
190						
192	50°					pocket penetrometer >4.5 tsf 2.0/2.8 recovery
194	50°					2.1/2.8 recovery
			Box #14			
196						
			S-11			2.8/2.9 recovery
198						
200		B.H. 199.6' Terminated hole; gas test no combustibles, 20% Oxygen, water sampled within 2" piezometer 2/18/81				stop circulation 4:00 1/29/81 1/30/81 - run water pressure tests. Material was too soft for the packers to seat properly. The only successful test was from 100'-120' @ 20 psi. The formation took no water. Piezometers installed: 2" pvc from 0-200' with cloth covered perforations from 160-195'. 1" pvc from 0-30' with perforations from 15-25'. Gravel packed w/ Bentonite plug from 27-33'. Surface cap, clean-up site.
202						
204						
206						
208						
210						
212						

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

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BORING LOG 14-1

Proj: DESIGN UNIT A-220 Date Drilled 9-14-15-83 Ground Elev. 225'

Drill Rig Failing 1500 Logged By L. Schoeberlein Total Depth 109.9'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.2 APSHALT 0.2-1.0 CONCRETE			GB	start drilling 10:30
	CL	YOUNG ALLUVIUM 1.0-5.6 SILTY CLAY: dark yellowish brown; mottled with greenish grey; some sand; ; moist; stiff; becomes reddish brown, mottled and contains occasional sand lenses to 4" thick	C-1	4 9	DR GB	0.8/1.0
2–4			J-1	3 6 10	SS GB	1.2/1.5 11:15 setting tub and casing to 5'
6	CL	OLD ALLUVIUM 5.6-10.5 SILTY CLAY: dark yellowish brown; mottled with greenish grey; some sand; moist; stiff; becomes reddish brown, mottled and contains occasional sand lenses to 4" thick and roots	C-2	5 10	DR RD	1.0/1.0
8		increased to very stiff	J-2	5 12 21	SS RD	0.4/1.5 drove rock ahead
10	SM	10.5-11.5 SILTY SAND: dark yellowish brown; mostly fine sand; dense; wet; contains lenses of coarser material	C-3	11 20	DR RD	1.0/1.0
12	CL	11.5-13.0 SILTY CLAY: dark yellowish brown; some silt, trace sand; very stiff; moist				
14	SC	13.0-27.5 CLAYEY SAND: dark yellowish brown, mostly fine sand, some clay; wet; moderately plastic; occasional gravel; dense; moist		15 19 20	SS RD	0/1.5 lost drive head of SPT, fished w/Shelby, no luck, drilled out with drag bit
16				30 50-3"	DR RD	0/0.75
18				95	DR RD	attempted drive w/ CCI sampler, brought up SPT shoe
20				31	DR	

Sheet 1 of 5

Project DESIGN UNIT A-220 **Date Drilled** 9-14-15-83 **Hole No.** 14-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SC	13.0-27.5 CLAYEY SAND: continued	C-4	36	DR	1.0/1.0
					RD	
				33	DR	1.0/1.0
22		increased clay content	C-5	52		
					RD	
				30	DR	1.0/1.0
24		becoming very dense	C-6	47		
					RD	
26				31	DR	1.0/1.0
			C-7	59		
					RD	sand running in to hole
28	CL	27.5-42.5 SANDY CLAY: slightly darker, some fine to medium sand; occasional gravel; hard; moist; occasional silty or clayey sand lenses	J-3	10	SS	1.5/1.5
				18		
				24		
					RD	
30						
				20	DR	0.8/1.0
32			C-8	62		
					RD	sand in rods
			J-4	19	SS	0.8/1.3
34				30		
				50-	4"	
					RD	
36	SP	1' sand lens ; mostly fine to medium sand; some silt and gravel		61	DR	0.8/1.0
			C-9	40		
	CL				RD	mixed mud (1 sack)
38			J-5	29	SS	0.5/0.9
				50-	5"	
					RD	
40						
	SC	becoming clayey sand		40	DR	0.8/0.8
42			C-10	60-	3.5"	
					RD	
	ML	42.5-45.5 CLAYEY SILT: dark yellowish	J-6	6	SS	1.5/1.5
44				10		

Project DESIGN UNIT A-220 Date Drilled 9-14-15-83 Hole No. 14-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	ML	42.5-45.5 CLAYEY SILT: continued brown; mottled with greenish; very stiff; moist		14	SS	
					RD	
46	SC/ CL	45.5-62.5 SANDY CLAY/CLAYEY SAND: inter-bedded; stiff; moist	C-11	27 50-	DR 3"	0.8/0.8
48			J-7	9 20 26	SS	1.2/1.5
50					RD	
			C-12	53 33	DR	1.0/1.0
52	CH				RD	
			J-8	13 38 27	SS	1.2/1.5
54					RD	
56		decreased clay content for 1'	C-13	37 54	DR	1.0/1.0
					RD	
58			J-9	7 14 19	SS	1.5/1.5
60					RD	
			C-14	21 40	DR	1.0/1.0
62					RD	
	ML	62.5-71.6 CLAYEY SILT: dark yellowish brown w/ mottling; hard; moist	J-10	8 17 37	SS	0.2/1.5
64					RD	
66			C-15	14 49	DR	1.0/1.0
					RD	
68						

Project DESIGN UNIT A-220 Date Drilled 9-14-15-83 Hole No. 14-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	ML	62.5-71.6 CLAYEY SILT: continued	J-11	14 22 30	SS	1.5/1.5
70					RD	
			C-16	26 50-	DR 5"	1.0/1.0
72	CL	71.6-72.5 SANDY CLAY: dark yellowish brown; some fine sand; hard; moist			RD	
	CL	72.5-77.0 SILTY CLAY: greyish green; massive bedding; contains mica; hard; moist	J-12	20 32 40	SS	1.5/1.5 disturbed by rock
74					RD	
76						
	SP	SAN PEDRO FORMATION 77.0-106.0 SAND: light greyish green; mostly fine to very fine sand; very dense; saturated				
78			C-17	77 50-	DR 3"	0.5/0.8
					RD	
80						
82						
			C-18	86 70-	DR 3"	0.6/0.8
84						
86						
88			J-13	31 50	SS	0.2/1.0 truck fuel pump down. 7 pm 9/14/83 9:25 am 9/15/83
90						
92						

Project DESIGN UNIT A-220 Date Drilled 9-14-15-83 Hole No. 14-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92	SP	77.0-106 SAND: continued			RD	
			C-19	67 60-	DR 4"	0.8/0.8
94					RD	
96						
98			PB-1		PB	1.3/2.5
100					RD	
102		grading fine grained with increased silt content, occasional rounded gravels				
104			C-20	57 50-	DR 4"	0.6/0.9
					RD	
106		PUENTE FORMATION 106-109.9 SILTSTONE AND CLAYSTONE: thinly interbedded; greyish green to dark olive; moist; not cemented Physical Condition: little fractured to massive; friable hardness and strength; little weathered				
108			C-21	38 50-	DR 5"	0.9/0.9
110		B.H. 109.9. Terminated hole after extending it to fine siltstone, installed 2" piezometer to bottom. Lower 20' slotted.				completed drilling 11:15
112						
114						
116						

Sheet 5 of 5

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BORING LOG 14-2

Proj: DESIGN UNIT A-220 **Date Drilled** 9-18-19-83 **Ground Elev.** 223

Drill Rig Failing 1500 **Logged By** L. Schoeberlin **Total Depth** 104.7

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT 0.2-0.7 CONCRETE			GB	start drilling 4:15
	SM	YOUNG ALLUVIUM 0.7-6.0 SILTY SAND:med. yellowish brown; mostly very fine sand; some silt med. dense; moist; iron staining				
2			C-1	10 11	DR	1.0/1.0
		grain size increased to med. sand			AD	
4		clayey fine sand lens	J-1	6 12 15	SS	1.5/1.5
6	CL	OLD ALLUVIUM 6.0-16.5 SILTY CLAY: dark yellowish brown mottled w/lt. olive brown and black; hard; moist contains interbeds of sandy clay and clayey sand			AD	
			C-2	11 34	DR	1.0/1.0
8					RD	set tub and cased to 8.5
		8.5-11.00 sandy clay	J-2	15 16 20	SS	1.5/1.5
10					RD	1.5/1.5
12			C-3	10 20	DR	1.0/1.0
					RD	
14		becoming sandier	J-3	16 27 38	SS	0.7/1.5
					RD	
16		becoming 16.5-26.0 SANDY CLAY:light olive brown; some fine sand; v. stiff; moist				
	CL		C-4	19 23	DR	1.0/1.0
18	CH				RD	
			J-4	7 10	SS	1.5/1.5
20						

Project DESIGN UNIT A-220 Date Drilled 9-18-19-83 Hole No. 14-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	16.5-26.0 SANDY CLAY: cont.		14	SS	6 pm 9/18/83
					RD	7 am 9/19/83
22		becoming hard		14	DR	
			C-5	55		1.0/1.0
					RD	
24			J-5	14	SS	
				25		1.5/1.5
				38		
26					RD	
	SC	grades to CLAYEY SAND				
	SM	grades to 26.9-31.5 SILTY SAND: dark yellowish brown; mostly fine sand; very dense; moist; occasional clayey sand lenses		25	DR	
			C-6	50-	4"	0.8/0.8
28					RD	
			J-6	14	SS	
				25		1.4/1.4
30				50-	5"	
					RD	
32	SP	grading to 31.5-46.5 SAND: variable colors; mostly fine to coarse sand; occasional gravels; very dense moist; poorly graded		51	DR	
			C-7	50-	5"	0.9/0.9
					RD	
34			J-7	25	SS	
				37		1.1/1.3
				50-	4"	
36					RD	
				61	DR	
			C-8	50-	2"	0.7/0.7
38					RD	
				30	SS	
			J-8	50-	4.5	0.4/0.9
40					RD	
42	SW	becoming well graded increased silt		33	DP	
			C-9	50-	4"	
44					RD	

Project DESIGN UNIT A-220 Date Drilled 9-18-19-83 Hole No. 14-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SW	31.5-46.5 SAND: cont. well graded beds	J-9	31 52	SS	0.8/1.0
					RD	
46						
	SM	46.5-48.5 SILTY SAND: med. yellowish brown; mostly fine sand some silt and gravel; very dense moist	C-10	31 50-	DR 4.5	0.7/0.9
48					RD	
	CL	48.5-51.5 SANDY CLAY: light olive brown; some fine to medium sand; very stiff; moist	J-10	14 22 30	SS	1.5/1.5
50					RD	
52	SC	51.5-56.5 CLAYEY SAND: light olive brown; mostly fine sand; mod. plastic finee; very dense; moist; contains sand interbeds	C-11	32 50-	DR 5.5	0.8/1.0
					RD	
54			J-11	35 36 39	SS	1.5/1.5
56					RD	
	CL	56.5-61.5 SILTY CLAY: light olive brown; hard; moist; occasional sandy clay lenses	C-12	20 44	DR	1.0/1.0
58					RD	
			J-12	13 27 28	SS	1.5/1.5
60		iron staining			RD	
62			C-13	27 50-	DR 5"	0.9/0.9
					RD	
64	CL	61.5-67.0 SANDY CLAY: dark greenish gray; some fine sand; hard; moist	J-13	12 19 37	SS	1.5/1.5
66					RD	
	SP	SAN PEDRO FORMATION 67.0-97.0 SAND: yellowish grey;				
68						

Project DESIGN UNIT A-220 Date Drilled 9-18-19-83 Hole No. 14-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	67.0-97.0 SAND: cont. mostly very fine sand; very dense; saturated; massive			RD	
			C-14	76 50-	DR 2"	0.6/0.7 partial
70					RD	
			J-14	40 50-	SS 4"	0.8/0.8
72					RD	
74			C-15	93 30-	DR ½"	0.5/0.5
					RD	
76			J-15	33 50-	SS 4"	0.8/0.8
					RD	
78		color change to greenish grey				
		silty	C-16	50 50-	DR 3.5"	0.8/0.8
80						
82						
84			C-17	50 50-	DR 4.5"	0.9/0.9
					RD	
86						
88			C-18	49 50-	DR 2"	0.7/0.7 rig chatter
90					RD	
		occasional gravelly lenses				
92						

Project DESIGN UNIT A-220 Date Drilled 9-18-19-83 Hole No. 14-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92	SP	67.0-97.0 SAND: cont.			RD	
94		sulfur odor	C-19	100-5"	DR	0.4/0.4 partial can
		2' cemented zone and gravels			RD	hard drilling
96						
98		PUENTE FORMATION 97.0-104.7 SILTY CLAYSTONE: dark greenish grey; thinly bedded 1/4" to 3"				
100		Physical Condition: little fractured to massive; friable hardness and strength; little weathered	C-20	23 50-4"	DR RD	0.7/0.8
102						
104			C-21	36 50-3"	DR	0.7/0.7
106		B.H. 104.7 Terminated hole; tremied grout to ground surface				complete drilling 1:45
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.

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BORING LOG 14-3

Proj: DESIGN UNIT A-220 **Date Drilled** 9-16-16-83 **Ground Elev.** 226.5

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 160.4

Hole Diameter 4 1/8" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-1.0 CONCRETE			GB	start drilling 2:00 pm
	SC	FILL 1.0-2.5 CLAYEY SAND: dark yellowish brown; mostly fine to medium sand; mod. plastic fines; dense; moist		8	DR	
2			C-1	16		1.0/1.0
					RD	
	CL	YOUNG ALLUVIUM 2.5-12.8 SILTY CLAY: dark yellowish brown; mottled; stiff to very stiff; moist; occasional sand grains; color grading to medium olive brown		4	SS	
4			J-1	7		1.5/1.5
				12		
					GB	
6				5	DR	3.5 pocket pen (tsf)
			C-2	12		1.0/1.0
					GB	set up tub & cased to 6.5'
8				6	SS	
			J-2	9		1.5/1.5
				11		
10					RD	
		sandy clay lenses interbedded with silty clay		13	DR	1.0/1.0
12			C-3	13		pocket pen > 4.5 tsf
					RD	
	CL	OLD ALLUVIUM 12.8-18.5 SILTY CLAY: dk yellowish brown; mottled; stiff to very stiff; moist; occasional sand grains; color grading to medium olive brown; occasional iron straining		7	SS	
14			J-3	13		1.5/1.5
				14		
					RD	
16						
				8	DR	
			C-4	13		1.0/1.0
18					RD	
	ML	18.5-32.0 SANDY SILT: light olive brown, with fine sand		9	SS	1.2/1.5
20			J-4	18		

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	ML	18.5-32.0 SANDY SILT: cont. non-plastic fines; dense; moist		24	SS	
					RD	
22				10	DR	1.0/1.0
			C-5	18		pocket pen 4.25 tsf
					RD	
24				15	SS	
			J-5	17		1.0/1.5
				27		
					RD	
26		occasional sandy zones, clayey zones and iron staining				
				24	DR	1.0/1.0
28			C-6	28		
					RD	
				9	SS	
30			J-6	16		1.5/1.5
				20		
					RD	
		becoming				
32	SC	32.0-41.5 CLAYEY SAND: light olive brown; mostly fine sand; moderately plastic fines; dense; moist; silty sand interbeds		18	DR	1.0/1.0
			C-7	23		pocket pen 3.75 tsf
					RD	
34				7	SS	
			J-7	15		1.5/1.5
				21		
36					RD	
		occasional gravels to 2", sub angular to round		11	DR	
38			C-8	24		1.0/1.0
					RD	
				6	SS	
		medium dense	J-8	8		1.5/1.5
40				14		
					RD	
42	SM	41.6-46.5 SILTY SAND: light olive brown; mostly medium sand; low plastic fines; dense; wet		35	DR	0.8/1.0
			C-9	45		
44		less silty at 43'			RD	

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SM	41.5-46.5 SILTY SAND: cont.	J-9	17 / 32 / 32	SS	1.0/1.5
46					RD	
	CL	46.5-50.2 SILTY CLAY: light olive brown; occasional gravels, rounded; hard; moist	C-10	18 / 35	DR	pocket pen >4.5 tsf
48					RD	
			J-10	10 / 21 / 40	SS	1.5/1.5
50	SM	SAN PEDRO FORMATION 50.2-61.0 SILTY SAND: very dense			RD	
52		decreased silt	C-11	47 / 50-4"	DR	0.8/1.0
					RD	
54			J-11	33 / 50-4"	SS	0.8/0.8
56					RD	
			C-12	45 / 50-5"	DR	0.7/0.9
58					RD	
			J-12	25 / 50	SS	1.0/1.0
60					RD	
	SP	61.0-68.5 SAND: light greyish green; very dense; moist to wet				
62			C-13	28 / 50-4"	DR	0.7/0.8
					RD	
64			J-13	15 / 35 / 40	SS	1.5/1.5
66					RD	
68						

Sheet 3 of 7

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	61.0-68.5 SAND: cont.			RD	
		PUENTE FORMATION 68.5-160.4 INTERBEDDED SILTSTONE AND CLAYSTONE: light greyish green and dark olive; beds 1/4" - 3"; dipping 30°; occasionally cemented; sulfur odor	C-14	30 50-	DR 3"	pocket pen > 4.5 tsf 0.7/0.7 7:30 pm 9/15/83 water @ 41' in am start drilling 7:30 am 9/16/83
70					RD	
72		Physical condition: little factured to massive, friable hardness and strength; little weathered				
74			C-15	30 50-	DR 3.5"	0.7/0.7
					RD	
76						
78		occasional sand lenses to 1/4" thick				
80			C-16	26 50-	DR 5"	0.8/0.9
					RD	
82						
84					PB	2.5/2.5
86						
					RD	
88						
90			C-17	27 50-	DR 4"	0.8/0.8
					RD	
92						

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		68.5-160.4 <u>INTERBEDDED SILTSTONE AND CLAYSTONE</u>: cont.			RD	
						93.5 rig chatter
94				30	DR	
			C-17	47		0.8/1.0
					RD	
96						
98						
					PB	
100			PB-2			2.7/2.7
102					RD	
104						
106						
108						
110						
112						
114						
116						

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		68.5-160.4 INTERBEDDED SILTSTONE & CLAYSTONE: cont.			RD	
118						
120			C-18	49	DR	
				50-2"		0.7/0.7
					RD	
122						
		123.5-125.5 cemented zone; very hard				rig chatter, changed bits to tricone 28 min/ft
124						
126						
128						13 min/ft
130						
						changed back to drag bit
132						
134						
136						
138						
140						

Project DESIGN UNIT A-220 Date Drilled 9-15-16-83 Hole No. 14-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		68.5-160.4 INTERBEDDED SILTSTONE AND CLAYSTONE: cont.	C-19	90	DR	0.7/0.7
				50-	1" RD	
142						
144						
146						
148						
150						thinned fluid
152						
154						
156						
158						
160			C-20	50-4.5"	DR	
162		B.H. 160.4' Terminated hole at extended depth due to anticipated ground water level of 150'. Installed piezometer to total depth with 40' of perforated section at bottom, gravel pack to 89' and grout tremied to 27', backfill top.				completed drilling 3:30
164						

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 14-4

Proj: DESIGN UNIT A-220 **Date Drilled** 9-16-83 **Ground Elev.** 220.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 99.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.8 CONCRETE			GB	start drilling 7:45
	SC	YOUNG ALLUVIUM 0.8-3.0 CLAYEY SAND: mod. yellowish brown; mostly fine sand; mod. plastic fines; dense; moist				
2			C-1	7 19	DR	1.0/1.0 pocket pen > 4.5 tsf
	SM	3.0-6.5 SILTY SAND: light olive brown; mostly medium sand, some silt; interbeds of clayey sand, silty clay and sandy clay; dense; moist			GB	
4			J-1	10 11 14	SS	1.5/1.5
6					GB	
	CL CH	OLD ALLUVIUM 6.5-18.5 SILTY CLAY: dark yellowish brown to black; sandy clay lenses; occasional gravel; hard; moist iron staining	C-2	15 25	DR	set tub and 6.5' of casing, mixed mud 1.0/1.0
8					RD	
			J-2	10 15 24	SS	1.5/1.5
10					RD	
12			C-3	12 23	DR	1.0/1.0 pocket pen > 4.5 tsf
					RD	
14		6" silty sand lens , color change to light olive brown, very stiff	J-3	14 12 15	SS	1.5/1.5
16					RD	casing leaking
		iron staining	C-4	10 19	DR	1.0/1.0
18					RD	add casing to 9.5'
	CL	18.5-24.8 SANDY CLAY: some fine to med.	J-4	9 16	SS	1.5/1.5
20						

Sheet 1 of 5

Project DESIGN UNIT A-220 Date Drilled 9-18-83 Hole No. 14-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	18.5-24.8 SANDY CLAY: continued		16	SS	
		sand, interbedded clayey sand, and silty sand; hard; moist			RD	attempt to mud up leaking casing
22		grading sandier		21	DR	1.0/1.0
			C-5	50		
					RD	
24				10	SS	1.5/1.5
		becoming	J-5	19		
	SC/SM	24.8-32.8 CLAYEY SAND/SILTY SAND/SILTY CLAY: light olive brown; mostly fine sand; mod. plastic fines; interbedded; dense; moist		28		
26					RD	
	CL	26.0 silty clay lens				
		27.3 silty clay; green; hard		13	DR	1.0/1.0
28			C-6	46		
					RD	
	CL	28.5 silty clay lens ; brown; hard				
				10	SS	1.5/1.5
30			J-6	16		
				29		
					RD	
	CL	31.5 sandy clay lense; brown; hard				
32				41	DR	1.0/1.0
	SM	SAN PEDRO FORMATION	C-7	50-	5.5"	
		32.8-42.7 SILTY SAND: green; mostly sand; low plasticity fines			RD	
34	SM	33.5 silty sand; green; saturated; v. dense		23	SS	0.8/1.5
			J-7	26		
				46		
					RD	
36	SM	36.0 silty sand; light olive brown; saturated; v. dense				
				46	DR	0.8/0.8
			C-8	50-	3"	
38					RD	
				25	SS	1.4/1.4
			J-8	43		
40				50-	5"	
					RD	
42				34	DR	1.0/1.0
		PUENTE FORMATION	C-4	46		
		42.7-99.7 SILTY CLAYSTONE: light grayish green; thin interbeds of drk. olive			RD	
44						

Project DESGIN UNIT A-220 Date Drilled 9-18-83 Hole No. 14-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	30°±	42.7-99.7 SILTY CLAYSTONE: continued contains mica; sulfur odor	J-9	6 9 14	SS	1.5/1.5
46		Physical Condition: little fractured to massive; friable to soft hardness; friable to plastic strength; little weathered			RD	
48		color becomes dark olive green; friable hardness and strength	C-10	39 50-	DR 3" RD	0.8/0.8
50						
52						
54		occasional cobbles or coarse gravel	C-11	36 50-	DR 4" RD	0.5/0.8
56						
58		slight cementation & small sand inclusions	C-12	36 50-	DR 4.5" RD	0.9/0.9
60						
62						
64			PB-1		PB	2.5/2.5
66					RD	
68						

Project DESIGN UNIT A-220 Date Drilled 9-18-83 Hole No. 14-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68		42.7-99.7 SILTY CLAYSTONE: continued			RD	
		thicker bedding	C-13	49 50-	DR 2.5"	0.7/0.7
70					RD	
72						
74			C-14	46 50-	DR 2"	0.7/0.7
					RD	
76						
78						
			C-15	59 65-	DR 2"	0.7/0.7
80					RD	
82						
84			PB-2		PB	2.0/2.5
86						
					RD	
88						
	30°	thinner bedding ~ 1/2-1"	C-16	33 50-	DR 3"	0.7/0.7
90					RD	
92						

Sheet 4 of 5

Project DESIGN UNIT A-220 Date Drilled 9-16-83 Hole No. 14-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		42.7-99.7 SILTY CLAYSTONE: continued			RD	
94			C-17	38 50-3"	DR RD	0.7/0.7
96						
98			C-18	68 50-2"	DR	0.7/0.7
100		B.H. 99.7 Terminated hole, tremied grout to bottom of hole				completed drilling 2:15. continuous slight circulation loss throughout hole
102						
104						
106						
108						
110						
112						
114						
116						

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

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BORING LOG 14-5

Proj: DESIGN UNIT A-220 **Date Drilled** 9-17-83 **Ground Elev.** 220'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 89.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-1.0 CONCRETE			GB	
	CL	FILL 1.0-2.6 SANDY CLAY: various greys, blacks, greens and browns				
2				6	DR	1.0/1.0
	SC	YOUNG ALLUVIUM 2.6-3.5 CLAYEY SAND: dark yellowish brown; dense	C-1	19		pocket pen 3.0 tsf
4	CL	3.5-9.0 SILTY CLAY: light & medium olive brown; stiff to very stiff; moist	J-1	4 6 9	SS	1.5/1.5
6					RD	set tub & cased to 6.5
				7	DR	1.0/1.0
8			C-2	14		
					RD	
10	CL	OLD ALLUVIUM 9.0-11.5 SILTY CLAY: dark to yellowish brown; occasional sand interbeds (thin); stiff to v. stiff	J-2	5 14 13	SS	1.5/1.5
					RD	
12	SC	11.5-17.0 CLAYEY SAND: yellowish brown; mostly fine sand; moderately plastic fines; medium dense; moist		10	DR	1.0/1.0
			C-3	14		
					RD	
14			J-3	6 13 14	SS	1.3/1.5
16					RD	
				16	DR	1.0/1.0
18	SM	17.0-22.5 SILTY SAND: yellowish brown; mostly fine sand; low plastic fines; dense; moist; interbeds of clayey sand and sandy clay	C-4	31		
					RD	
20			J-4	14 20	SS	1.5/1.5

Sheet 1 of 4

Project DESIGN UNIT A-220 Date Drilled 9-17-83 Hole No. 14-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SM	17.0-22.5 SILTY SAND: continued		32	SS	
						leak around casing
22				36	DR	0.8/0.8
	SP	SAN PEDRO FORAMTION 22.5-36.5 SAND: light yellowish brown; mostly fine sand; very dense; moist to wet	C-5	50-	3"	
					RD	pushed additional casing to 9.5'
24				29	SS	0.0/1.0
				52		drove up to pull out, problem w/hammer
					RD	
26						
		silty clay lens		45	DR	
		coarse sand & silt lens	C-6	50-	5"	
28					RD	
		clayey sand lenses w/sand 2-3" thick	J-5	23	SS	1.0/1.5
				31		
30				47		
					RD	
32				28	DR	0.0/1.0
				32		
					RD	
34			J-6	20	SS	1.5/1.5
				30		
				47		
					RD	
36						
		WEATHERED PUENTE FORMATION 36.5-38.0 SILTY CLAYSTONE: light olive brown; mottled coloring		16	DR	1.0/1.0
			C-7	28		
38		PUENTE FORMATION 38.0-39.7 SILTY CLAYSTONE INTERBEDDED W/ CLAYEY SILTSTONE: greyish green dark olive; dip 45-50°; thinly to thickly bedded Physical Condition: little fractured to massive; friable hardness and strength; little weathered			RD	
			J-7	7	SS	1.5/1.5
40				11		
				14		
					RD	
42						
44						

Project DESIGN UNIT A-220 Date Drilled 9-17-83 Hole No. 14-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	50°	38.0-89.7 SILTY CLAYSTONE INTERBEDDED W/ CLAYEY SILTSTONE: continued	C-8	34 73-3"	DR RD	0.8/0.8
46						
48						
			C-9	40 70-3"	DR RD	0.8/0.8
50						
52						
54			C-10	34 50-4"	DR RD	0.8/0.8
56						
58						
60			PB-1		PB	2.5/2.5
62					RD	
64			C-11	33 58	DR RD	0.8/1.0
66						
68						

Sheet 3 of 4

Project DESIGN UNIT A-220 Date Drilled 9-17-83 Hole No. 14-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68		38.0-89.7 SILTY CLAYSTONE INTERBEDDED W/ CLAYEY SILTSTONE: continued			RD	
			C-12	41 68-	DR 3"	0.8/0.8
70					RD	
72						
		73.5-76.0 well cemented zone				rig chatter 2-1/2'/hr
74						
76						
78			PB-2		PB	trouble getting on bottom through tight hard zone 2.5/2.5 hard chips as slough at top of sample
80					RD	
82						
84			C-13	51 77-	DR 3"	0.7/0.7
					RD	
86						
88						
			C-14	44 50-	DR 3"	completed drilling 5:30
90		B.H. 89.7. Terminated hole, grouted to surface				continuous slight circulation loss throughout hole
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.

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BORING LOG 15

Proj: DESIGN UNIT A-220 **Date Drilled** 1-26-81/1-28-81 **Ground Elev.** 200'

Drill Rig FAILING 1500 **Logged By** S. Testa **Total Depth** 200.0'

Hole Diameter 5.0" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT			AD	started drilling at 11 a.m., augered down to 6.0'
	CL	ALLUVIUM 0.2-7.0 SANDY CLAY: dark yellowish brown; some medium to coarse sand; moist; stiff				
6					RD	
7	SC					
10		7.0-15.0 CLAYEY SAND: dark yellowish brown, fine grained sand, some fines, trace of tar, mottled, moist, dense	J-1	8 15 21	SS	SPT at 10.0'1.1/1.5 recovery, pocket penetrometer 3.5 tsf (broke apart)2-9-81
11.5					RD	
15	SP	15.0-15.5 TAR SAND: black, fine to medium subangular to subrounded sand	J-2	7 8 21	SS	SPT at 15.0' 1.5/1.5 recovery
15.5	CL	15.5-20.0 SANDY CLAY: dark yellowish brown mostly fines, with a little fine to medium sand, mottled, moist, stiff				
16.5					RD	
20						

Project DESIGN UNIT A-220 Date Drilled 1-26-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	20.0-21.3 SANDY CLAY: dark greenish gray, with some fine sand, mottled, moist, hard	C-1	25 26 29	DR	(continued) sample at 20.0', 1.5/1.5 recovery pocket penetrometer 3.75 (broke apart) 2-9-81
22	SP	21.3-28.0 SAND: dark greenish gray, trace of fines, fine to medium sand, trace of fine gravel, moist, medium dense, poorly graded	J-3	3 4 7	SS RD	SPT at 21.5' .4/1.5 recovery
24		25.0-26.0 continued, moist, dense	J-4	6 23 24	SS RD	SPT at 25.0',1.0/1.5 recovery
26						
28	CL					
30		28.0-30.8 SANDY CLAY: greenish black with some fine sand, moist	J-5	6 11 17	SS RD	SPT at 30.0', 1.5/1.5 recovery pocket penetrometer 3.5 tsf (broke apart) 2-9-81
	SP	30.8-42.0 SAND: greenish black, fine to coarse sand, trace of fine gravel, trace of fines, moist, medium dense, poorly graded				
32						
34		dark greenish gray, mostly fine to coarse sand, trace of fines, very dense	J-6	25 33 25	SS RD	SPT at 35.0', 1.5/1.5 recovery.
36						
38						
40		continued, moist, very dense	C-2	24 39 50	DR	sample from 40.0' to 41.5', 1.5/1.5 recovery
42	CL	42.0-53.0 CLAY: greenish black, fine sand, moist, very stiff	J-7	15 16 21	SS RD	SPT at 41.5, 1.5/1.5 recovery, pocket penetrometer > 4.5, 2-9-81
44						

Project DESIGN UNIT A-220 Date Drilled 1-26-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL	42.0-49.0 CLAY:(continued)			RD	(continued)
46		greenish black, trace of fine sand, moist, vert stiff	J-8	12 15 21		SPT at 45.0', 1.5/1.5 recovery
48					RD	
50	CL	49.0-53.0 SANDY CLAY: dark greenish gray with some trace of fine sand, gravel, moist, very stiff	J-9	16 23 26	SS	SPT at 50.0',1.5/1.5 recovery, pocket penetrometer > 4.5, 2-9-81
52					RD	
54 56	SC	53.0'-58.0' CLAYEY SAND: dark greenish gray, fine to coarse sand with some fines, moist, very dense grades coarser with depth	J-10	16 16 21	SS	SPT at 55.0', 1.5/1.5 recovery
					RD	
58 60	CL	58.0'-64.0' SANDY CLAY: dark greenish gray, some fine to coarse sand, moist, very stiff, thinly imbedded	C-3	30 62	DR	sample from 60.0' to 61.0', 1.0/1.0 recovery, pocket penetrometer 4.5 (broke apart) 2-9-81
62			J-11	11 13 22	SS	
					RD	
64 66	SC/CL	64.0'-69.5' SANDY CLAY AND CLAYEY SAND: thinly laminated dark greenish gray, moist	J-12	11 21 25	SS	SPT at 65.0', 1.0/1.5 recovery, pocket penetrometer > 4.5, 2-9-81
68		SANDY CLAY: fine to coarse sand CLAYEY SAND: fine to coarse sand, some fines			RD	

Project DESIGN UNIT A-220 Date Drilled 1-26-81/1-27-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SC / CL	64.0-69.5 SANDY CLAY AND CLAYEY SAND: (continued)			RD	(continued)
		SAN PEDRO FORMATION				
70	SP	69.5-90.0 SAND: dark greenish gray fine sand, little fines, very moist, very dense	J-13	22 30 32	SS	SPT at 70.0', 1.5/1.5 recovery
72					RD	
74		continued fine sand	J-14	15 10 21	SS	SPT at 75.00, 1.0/1.5 recovery
76					RD	
78						
80				50 50	DR SS RD	sample at 80.0', no recovery, SPT at 80.5', .1/.5 recovery, halted drilling at 81.0' at 5:00 PM. resumed drilling at 7:00 AM, 1-27-81
82						
84		continued		50/4"	SS	SPT at 85.0', .1/.4 recovery
86					RD	
88		shell fragments from cuttings				minor rod chatter below 80.6', considerable rod chatter from 88.0 to 88.5', minor from 88.5' to 90.0', continuous pitcher barrel sampling from 90.0'
		PUENTE FORMATION				
90	20°	90.0-200.0 CLAYSTONE: thinly laminated, primarily olive gray claystone with fine grained light gray sandstone laminae, micaceous, fossiliferous	Box # 1		PB	
92						

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		90.0-200.0 CLAYSTONE: 92.5-97.5 thinly laminated olive gray claystone, light gray sandstone,			PB	(continued) 200 psi
	5-30°	and dark yellowish brown silt-				
94		stone (occasionally well cement- ed).				
	0-15°					200 psi
96						
		97.5-100.0 primarily claystone with fine	Box # 1			200 psi
98	0-10°	sandstone				
		Physical Condition: massive soft to friable hardness, plastic				
100	0-10°	to weak strength, fresh. claystone and fine sandstone				200 psi
102		claystone and fine sandstone, no discernable bedding apparent				200 psi
104		Physical Condition: continued; tends to fracture along bedding planes, notably sandstone				200 psi pocket penetrometer >4.5 2-9-81
106			Box # 2			
	10-15°					200 psi
108						
110		alternating laminations of clay-stone, sandstone and siltstone				250 psi
112	45°					250 psi
	5-15°	continued, siltstone is fossili-ferous				
114						
			Box #3			
116						

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116	10-15°	90.0-200.0 CLAYSTONE: (continued) alternating thin to medium laminations, primarily claystone with subordinate fine sandstone and siltstone, cross-bedding evident in sandstone laminae, mottled from 116.8-117.0			PB	(continued) pocket penetrometer >4.5 2-9-81
118			Box # 3			200 psi, gas check 0.0% LEL, 11 a.m.
120		primarily claystone with fine sandstone laminae from 117.5				200 psi
122	0-10°					200 psi
124	20-30°	alternating claystone, sandstone and siltstone from 125.0', cross bedding apparent				250 psi
126						
128			Box # 4			250 psi pocket penetrometer >4.5 2-9-81
130		primarily claystone, fine sandstone				200 psi
132		continued Physical Condition: massive, soft to friable hardness, plastic to weak strength, fresh, tends to fracture along bedding planes notably sandstone				pocket penetrometer >4.5 2-9-81 200 psi
134		alternating thin to medium laminations of claystone, subordinate sandstone and siltstone from 135.7	Box # 5			
136						
138	15-20°	continued				
140						

Sheet 6 of 9

Project DESIGN UNIT A-220 Date Drilled 1-27-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		90.0-200.0 CLAYSTONE: (continued) Physical Condition: massive, soft to friable hardness, plastic to weak strength, fresh; tends to fracture along bedding planes, clay filled 60° fracture at 142.0, primarily claystone with thin to medium fine sandstone laminae	Box # 6		PB	(continued) 250 psi
142	60°					
	10-15°					200 psi
144						200 psi pocket penetrometer >4.5 2-9-81
	5-10°					
146						
	10-15°					200 psi
148						
150	0-15°	thin laminae of claystone, siltstone and fine sandstone				250 psi
						pocket penetrometer > 4.5 tsf 2-9-81
152			Box # 7			250 psi
154						
		continued				250 psi
156						250 psi
158						
160		primarily claystone, fine sandstone				200 psi
162			Box # 8			200 psi
164						

Project DESIGN UNIT A-220 Date Drilled 1-27-81/1-28-81 Hole No. 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
164		90.0-200.0 CLAYSTONE: (continued) Physical Condition: massive, soft to friable hardness, plastic to weak strength, fresh, tends to fracture along bedding planes, notably sandstone	Box # 8		PB	(continued) 200 psi
166						pocket penetrometer >4.5 2-9-81 250 psi
168	10-15°	primarily claystone to 169.0, alternating claystone with subordinate sandstone and siltstone	Box # 9			
170						200 psi
172	15-25°	alternating thin to medium laminae of claystone, fine sandstone and siltstone				pocket penetrometer >4.5 2-9-81 250 psi
174						stopped drilling at 5 at a depth of 175.0'
	15-25°					1-28-81 resumed drilling at 7 a.m., light rain
176						250 psi
178	25°	primarily claystone with sandstone laminae	Box #10			
180	20°					250 psi
182		fossiliferous				pocket penetrometer >4.5 2-9-81
184						
186						
188	15-20°		Box #11			

Project DESIGN UNIT A-220 **Date Drilled** 1-28-81 **Hole No.** 15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
188		90.0-200.0 CLAYSTONE: (continued) Physical Condition: as previously described				(continued)
190		primarily olive gray, claystone with thin to medium fine sandstone				200 psi
192			Box #11			pocket penetrometer >4.5 2-9-81 200 psi
194						200 psi
196						
198			Box #12			200 psi gas check 0.0% LEL, 11:50 pocket penetrometer >4.5 2-9-81 Terminated hole at
200		B.H. 200.0' Hole Terminated				11:45 at a depth of 200.0' 1-29-81 (heavy rain) water pressure test taken at the following intervals 100-120', 72-90'(10am to 11am, reamed hole to 7" then installed 100.0' of 4" PVC and grouted
202						
204						
206						
208						
210						
212						

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

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BORING LOG 15-A

Proj: DESIGN UNIT A-220 **Date Drilled** 11-8-83 **Ground Elev.** 199'

Drill Rig MAN-SIZE AUGER **Logged By** J. Stellar **Total Depth** 60'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		ASPHALTIC CONCRETE PAVEMENT 0.0-1.0				Hole stood well in general, 3'-4' of caving @ 17'-22' and probable caving 55'-60' Hole caved at 40 to 45" after drilling
	ML	FILL 1.0-2.0 CLAYEY SILT: dark brown, moist, stiff, minor coarse sand				
2	ML	ALLUVIUM 2.0-7.0 SANDY SILT: medium brown, moist, stiff				
4						
6						
	SP	7.0-9.0 SAND: medium brown, very moist, medium dense, medium to coarse grained				
8						
	ML	9.0-17.0 SILT: medium brown, moist, stiff, with layers of coarse sandy silt				
10						
12						
14						water level @ 15' after 2 hours water level @ 16' after 1½ hours
16						
	SP	17.0-23.0 SAND: light brown, wet, medium dense, coarse grained, saturated, very minor gravel to ½"				3 to 4' of caving at 17'-22', seepage at 10-15± g.p.m.
18						
20						

Project DESIGN UNIT A - 220 Date Drilled 11-8-83 Hole No. 15A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	17.0-23.0 SAND: (continued)				
22.5		gravel to 3' @ 22.5'				
23	ML	23.0-35.0 SILT: medium brown v. moist, stiff, with layers of sandy silt				
27		becomes reddish brown and v. stiff				
32.5		becomes medium brown				
33						bag sample @ 33'
35	ML	35.0-42.0 CLAYEY SILT: blue gray, very moist stiff, with layers of sandy silt				
42	SP/SM	42.0-44.6 SAND: blue, wet, dense with layers of silty sand and silt				

Depth scale: 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44

Project DESIGN UNIT A - 220 Date Drilled 11-8-83 Hole No. 15A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SM	42.0-44.6 SAND: (continued)				
	ML	44.6-55.0 CLAYEY SILT: blue, very moist, stiff with layers of silt and sandy silt				
46						
48		becomes v. stiff				bag sample @ 49'
50						
52						
54						Bag sample @ 55' unable to remove sand cuttings below 56' - saturated clean sand falling from bucket probable caving 55'-60' based on drilling behavior
	SP	SAND PEDRO FORMATION 55.0-60.0 SAND: blue, wet, dense, with minor layers of sandy silt and silt				
56		clean sand				
58						
60		B.H. 60.0'. Water at 27' during drilling operation. H20 @ 16' after 1½ hours. Hole caved back to 45' after 1½hrs. Water level at 15' after 2 hours. Hole caved back to 40' after 2 hours.				
62						
64						
66						
68						

Sheet 3 of 3

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BORING LOG 15-1

Proj: DESIGN UNIT A-220 | Date Drilled 9-22-23-83 | Ground Elev. 196'

Drill Rig Failing 1500 | Logged By L. Schoerlein | Total Depth 84.8

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT 0.2-1.0 CONCRETE			GB	start drilling 2:45
	CL	FILL 1.0-2.5 SILTY CLAY: dark greenish grey to dark yellowish brown; stiff; moist	C-1	4 9	DR	1.0/1.0
2					AD	
	SM ML	OLD ALLUVIUM 2.5-7.5 SILTY SAND/SANDY SILT: brown; dense; moist	J-1	11 14 26	SS	1.5/1.5
4					RD	set tub and cased to 6.5'
6			C-2	12 21	DR	1.0/1.0
					RD	
8	SP	7.5-15.5 SAND: yellowish grey; mostly fine sand, silt; dense to v. dense; dry to moist; occasional zones of silt	J-2	16 16 25	SS	1.0/1.5
10					RD	
			C-3	17 48	DR	1.0/1.0
12					RD	
			J-3	25 38 40	SS	1.0/1.5
14		increased silt				
					RD	
16	SC	15.5-24.0 CLAYEY SAND W/CLAYEY SILT: yellowish grey; mostly fine to medium sand; dense; moist	C-4	14 14	DR	1.0/1.0
					RD	
18			J-4	17 28 34	SS	
20					RD	

Project DESIGN UNIT A-220 Date Drilled 9-22-23-83 Hole No. 15-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SC	15.5-24.0 <u>CLAYEY SAND</u>: continued			RD	
		contains occasional silty zones and sand lenses, grades to sandy clay	C-5	24 22	DR	1.0/1.0
22					RD	
			J-5	8 11 16	SS	1.5/1.5
24	CL	24.0-27.5 <u>SILTY CLAY</u>: light olive grey; stiff; moist			RD	
26		thin sandy clay lens 3"	C-6	16 21	DR	1.0/1.0 pocket pen 3.5 tsf
					RD	
28	CL	27.5-45.8 <u>CLAY</u>: dark greenish grey; v. stiff to hard; moist; sample J-6 - 3" lenses of silty clay and sandy clay	J-6	14 30 39	SS	1.5/1.5
30					RD	
	CH	hard	C-7	25 58	DR	1.0/1.0 pocket pen > 4.5 tsf
32					RD	
			J-7	16 27 29	SS	
34					RD	
36	CL	grading to SILTY CLAY, v. stiff	PB-1		PB	2.5/2.5
38	ML	grading to SILT	J-8	12 25 39	SS	1.5/1.5
40					RD	
	SM	grading to SILTY SAND	C-8	19 61	DR	1.0/1.0 pocket pen > 4.5 tsf
42					RD	
44	CL	grading to SILTY CLAY	J-9	6 13	SS	1.5/1.5

Project DESIGN UNIT A-220 Date Drilled 9-22-23-83 Hole No. 15-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL	27.5-45.8 CLAY: continued		22		
					RD	
46	CL	45.8-55.6 SANDY CLAY: dark greenish grey; little fine sand; hard, moist; contains occasional lenses of sand clay	C-9	49 50-	DR 4" RD	0.8/0.8
48			J-10	17 30 30	SS	1.5/1.5
						6:30 9/22/83 6:30 9/23/83
50					RD	
			C-10	30 50-	DR 4.5" RD	0.9/0.9
52						
		clayey sand grading to sandy silt grading to silty sand grading to:	J-11	20 41 50-	SS 5" RD	1.4/1.4
54						
		SAN PEDRO FORMATION				
56	SP	55.6-67.5 SAND: dark greenish grey; fine sand; very dense; saturated	C-11	60 50-	DR 3" RD	0.8/0.8
58			J-12	25 50-	SS 5" RD	0.5/0.9
60						
			C-12	70 50-	DR 2.5" RD	0.6/0.7
62						
			J-13	36 56	SS RD	0.8/1.0
64						
66			C-13	100 30-	DR 1/2" RD	0.5/0.5
68	SW/GP	67.5-73.0 GRAVELLY SAND:				

Project DESIGN UNIT A-220 Date Drilled 9-22-23-83 Hole No. 15-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SW / GP	67.5-73.0 GRAVELLY SAND/SANDY GRAVEL: cont. greenish grey; mostly fine to coarse sand; fine to coarse gravel; v. dense; saturated	J-14	50-4"	SS RD	0.3/0.3
70						
				110-3"	DR RD	0.0/0.2
72						
	SM	73.0-76.5 SILTY SAND: dark greenish grey; mostly fine sand; some non-plastic silt; v. dense; saturated; sulfur odor				
74			J-15	68 50-	SS 3" RD	0.5/0.7
		75.5-76.5 gravelly				
76						
		PUENTE FORMATION 76.5-84.8 INTERBEDDED SILTSTONE, CLAYSTONE AND FINE SAND: olive grey and dark greenish; sand lenses; thinly to thickly bedded	C-14	30 50-	DR 5" RD	0.9/0.9
78						
		Physical Condition: fractured to massive; friable hardness and strength; little weathered to fresh	C-15	39 50-	DR 3" RD	0.7/0.7
80						
82						
84			C-16	34 50-	DR 3.5"	0.7/0.8
		B.H. 84.8 Terminated hole, installed piezometer bottom 20' slotted				complete drilling 10:15
86						
88						
90						
92						

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BORING LOG 15-2

Proj: DESIGN UNIT A-220 | Date Drilled 9-21-22-83 | Ground Elev. 199'

Drill Rig Failing 1500 | Logged By L. Schoeberlein | Total Depth 86.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.7 CONCRETE			GB	start drilling 5 pm
	SM / CL	FILL 0.7-8.5 INTERBEDDED SILTY SAND, SANDY CLAY, & SILTY CLAY: mottled yellowish grey to light brown; lenses 2" to 1' thick, stiff or dense; dry to moist				
2			C-1	12 18-	DR 6"	0.7/0.7
					AD	
4			J-1	11 20 25	SS	1.5/1.5
6					RD	set tub & cased to 6.5'
8			C-2	7 16	DR	1.0/1.0
					RD	
	CL	OLD ALLUVIUM 8.5-21.0 SILTY CLAY: medium brown; v. stiff; moist	J-2	9 15 27	SS	1.5/1.5
10					RD	
12	CH	mottled dark greenish grey and medium brown	C-3	18 29	DR	1.0/1.0
					RD	
14			J-3	9 15 20	SS	1.5/1.5
16					RD	
			C-4	13 24	DR	1.0/1.0 pocket pen > 4.5 tsf
18					RD	
20		contains 6" sandy clay lens	J-4	10 17	SS	1.0/1.5

Project DESIGN UNIT A-220 Date Drilled 9-21-22-83 Hole No. 15-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	8.5-21.0 SILTY CLAY: continued		24		
					RD	
	CL	21.0-31.0 SANDY CLAY: yellowish brown; trace fine sand; stiff; moist				
22				12	DR	1.0/1.0
			C-5	50-	5.5	" pocket pen 2.5 tsf
					RD	
24		sand content decreasing		9	SS	1.5/1.5
			J-5	12		
				20		
					RD	
26						
				14	DR	1.0/1.0
			C-6	26		pocket pen 4.25 tsf
28					RD	
				7	SS	1.5/1.5
			J-6	17		
30				23		
					RD	
	CL	31.0-46.5 CLAY: mottled greenish grey and yellowish brown, grading to dark greenish grey at 37.5'; v. stiff moist				
32				18	DR	1.0/1.0
			C-7	26		
					RD	
34				14	SS	1.5/1.5
		becomes silty and contains some sand lenses	J-7	20		
				25		
					RD	
36						
				27	DR	1.0/1.0
			C-8	45		7:30 9/21/83
38					RD	6:30 9/22/83
				15	SS	1.5/1.5
			J-8	27		
40				42		
					RD	
42					PB	
			PB-1			2.5/2.5
44						

Sheet 2 of 4

Project DESIGN UNIT A-220 Date Drilled 9-21-22-83 Hole No. 15-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL	31.0-46.5 CLAY: continued grading to silty			RD	
			J-9	11	SS	1.5/1.5
				23		
				44		
46					RD	
	SM	46.5-48.5 SILTY SAND: dark greenish grey; mostly fine sand, silt; v. stiff; moist	C-9	30	DR	0.8/0.8
				50-	4"	
48					RD	
	CL	48.5-65.0 SANDY CLAY: dark greenish grey; some fine sand; v. stiff; moist	J-10	9	SS	1.5/1.5
50				18		
				23		
					RD	
52			C-10	23	DR	1.0/1.0
				49		
					RD	
54		silt, clayey silt, clayey sand and silty sand interbeds to 8" thick	J-11	9	SS	1.5/1.5
				22		
				33		
56					RD	
			C-11	25	DR	1.0/1.0 pocket pen > 4.5
58				37		
					RD	
			J-12	9	SS	1.5/1.5
60				15		
				22		
					RD	
62			C-12	23	DR	1.0/1.0
				42		
		63.5-65.0 clayey sand lense			RD	
64			J-13	17	SS	1.0/1.5
				42		
	SP	SAN PEDRO FORMATION 65.0-75.0 SAND: dark greenish grey; mostly fine sand, silt; v. dense; saturated		56		
66					RD	
				67	DR	sample fell out
68				50-	1" DR	

Project DESIGN UNIT A-220 Date Drilled 9-21-22-83 Hole No. 15-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	65.0-75.0 SAND: continued			RD	
			J-14	17 24 36	SS	1.5/1.5
70					RD	
72			C-13	80 80-	DR 3"	0.7/0.7 best San Pedro sample
					RD	
74			J-15	22 51	SS	0.8/1.0
	SW	75.0-85.0 GRAVELLY SAND: dark greenish grey; mostly fine to coarse sand; fine to coarse gravel; v. dense; saturated			RD	rig chatter
76						
78			C-19	110 86-	DR 4.5"	
					RD	skip sample, too gravelly
80						
		very fine silty sand lens	J-16	32 50-	SS 2"	0.4/0.7
82					RD	
84				184	DR	0.0/0.5
					RD	
		PUENTE FORMATION 85.0-86.7 INTERBEDDED SILTSTONE & CLAYSTONE: olive grey and dark greenish grey; thinly to thickly bedded Physical Condition: little fractured to massive; friable hardness and strength; fresh to little weathered				
86			C-15	42 50-	DR 3"	gravels @ top fall in. 0.6/0.7, pocket pen ~~>4.5 tsf~~
						complete drilling 12:15
88						
		B.H. 86.7 Terminated hole, tremied grout to surface				
90						
92						

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BORING LOG 15-3

Proj: DESIGN UNIT A-220 **Date Drilled** 9-28-83 **Ground Elev.** 199'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 160.9

Hole Diameter 4 7/8" **Hammer Weight & Fall** NA

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.8 CONCRETE			GB	start day 7:00 am start drilling 7:30 am
	CL	FILL 0.8-1.8 GRAVELLY SANDY CLAY: mottled grey and browns; soft	C-1	4 11	DR	1.0/1.0
2	SM	OLD ALLUVIUM 1.8-6.0 SILTY SAND: yellowish brown; mostly fine sand; medium dense; moist			GB	
			J-1	3 5 8	SS	0.8/1.5
4					RD	set tub & cased to 4.5'
		grading to				
6	CL	6.0-20.5 SILTY CLAY: yellowish brown mottled with mod. brown; stiff; moist; interbedded sand lenses	C-2	7 10	DR	1.0/1.0 pocket pen> 4.5 tsf
					RD	
8			J-2	3 8 12	SS	1.3/1.5
10					RD	
			C-3	5 10	DR	1.0/1.0 pocket pen 3.5 tsf
12					RD	
			J-3	6 10 15	SS	1.3/1.5
14					RD	
16			C-4	12 18	DR	0.9/1.0 pocket pen > 4.5 tsf
					RD	
18			J-4	7 14 22	SS	1.3/1.5
20					RD	

Project DESIGN UNIT A-220 Date Drilled 9-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	6.0-20.5 SILTY CLAY: continued			RD	
	CL	20.5-38.0 SANDY CLAY: yellowish brown; some fine sand; dense; moist		13	DR	0.9/1.0
22			C-5	22		
					RD	
	SC	grading to CLAYEY SAND: moderately plastic fines; med. dense; moist;~ 2' thick	J-5	11 12 12	SS	1.0/1.5
24					RD	
26	CL	grading back to SANDY CLAY		10	DR	1.0/1.0
			C-6	23		
					RD	
28		interbedded sand and clay	J-6	10 16 23	SS	1.5/1.5
30		grading to dark greenish grey; interbedded silty clay with clayey sand~ 6" thick			RD	
			C-7	18 50-	DR 5"	1.0/1.0
32					RD	
			J-7	11 12 20	SS	1.3/1.5
34					RD	
36			C-8	25 50-	DR 4"	0.7/0.8
		increased silty clay			RD	
38	CL	38.0-42.5 CLAY: dark greenish grey; v. stiff to hard; moist	J-8	13 30 53	SS	1.5/1.5
40					RD	
			C-9	48 50-	DR 2"	0.6/0.6
42		grading to			RD	
	CL	42.5-44.5 SANDY CLAY: very stiff to hard	J-9	11 26	SS	1.5/1.5
44						

Project DESIGN UNIT A-220 Date Drilled 9-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL			30	SS	
	CL	44.5-47.5 SILTY CLAY: very stiff to hard			RD	
46				32	DR	1.0/1.0
			C-10	50		
		grading to			RD	
48	ML	47.5-52.0 CLAYEY SILT: dark greenish grey; stiff; wet	J-10	10	SS	1.5/1.5
				28		
				31		
50					RD	
					PB	
52	CL	grading to 52.0-65.5 SANDY CLAY: dark greenish grey; v. stiff; moist	PB-1			2.5/2.5
			J-11	9	SS	1.3/1.5
54				18		
				29		
					RD	
56		contains thin interbeds of sand, clayey sand, silty sand and silty clay, 2"-8"thick		51	DR	1.0/1.0
			C-11	47		
					RD	
58			J-12	17	SS	1.0/1.5
				40		
				43		
					RD	
60		increased sand content				
				30	DR	1.0/1.0
			C-12	50-	5.5"	
62					RD	
				7	SS	0.0/1.5
64				13		
				16		
					RD	
		grading to				
66	SM	65.5-67.5 SILTY SAND: dark greenish grey; plastic fines; dense; moist				
			C-13	31	DR	0.9/0.9
				50-	5"	
					RD	
68	SP	SAN PEDRO SAND				

Sheet 3 of 7

Project DESIGN UNIT A-220 Date Drilled 9-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	67.5-80.5 SAND: continued dark greenish grey; trace of silt; v. dense; saturated	J-13	23 51	SS RD	0.7/1.0
70						
			C-14	60 50-	DR 3" RD	0.5/0.7
72						
			J-14	50 50-	SS 5" RD	0.6/0.9
74						
76			C-15	57 50-	DR 2½" RD	0.6/0.7
78			J-15	32 50-	SS 5.5" RD	0.5/0.9
80						
	GP	80.5-86.5 SANDY GRAVEL: dark greenish grey; mostly fine to medium gravel; angular; occasional coarse gravel and cobbles; v. dense; saturated	C-16	125-	5.5" RD	rig chatter DR 0.4/0.4 disturbed sample
82						
84		fine sand lens, contains shells	J-16	32 50-	SS 4.5" RD	0.6/0.9
86						
		PUENTE FORMATION 86.5-160.9 INTERBEDDED SILTSTONE & CLAYSTONE: olive grey and greenish black; thinly interbedded 1/2" to 3"; occasional well cemented zones in primarily uncemented material.				
88			C-17	46 60-	DR 3" RD	0.7/0.8
90						
92						

Sheet 4 of 7

Project DESIGN UNIT A-220 Date Drilled 19-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		86.5-160.9 INTERBEDDED SILTSTONE & CLAYSTONE: continued Physical Condition: little fractured to massive; friable strength and hardness; little weathered to fresh			RD	
94						
96						
98						
100						
101						rig chatter probable cemented zone 1-2" thick
102						
104						
106						
108						
110		occasional 1/16" thick sand lenses	C-18	40 52-	DR 3"	0.7/0.7 rig chatter
112						
114						
116						

Project DESIGN UNIT A-220 Date Drilled 9-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		86.5-160.9 INTERBEDDED SILTSTONE & CLAYSTONE: continued			RD	
130			C-19	39 50-4"	DR RD	0.8/0.8

Depth scale: 116, 118, 120, 122, 124, 126, 128, 130, 132, 134, 136, 138, 140

Project DESIGN UNIT A-220 Date Drilled 9-28-83 Hole No. 15-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		86.5-160.9 INTERBEDDED SILTSTONE & CLAYSTONE: continued			RD	
145			C-20	65 50-3"	DR	0.7/0.7
146					RD	
159		35° dip fractures more readily along fine sand partings				
160			C-21	41 50-4.5"	DR	0.8/0.9
161-164		B.H. 160.9 Terminated hole at extended depth to install piezometer to read bedrock H20 levels. Bottom 20' slotted, seal from ~80 to 120'; pea gravel backfill from 120-160'				completed drilling 6:45

Sheet 7 of 7

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

Converse Consultants, Inc.
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BORING LOG 15-4

Proj: DESIGN UNIT A-220 | Date Drilled 9-19-20-83 | Ground Elev. 199'

Drill Rig Failing 1500 | Logged By L. Schoeberlein | Total Depth 84.9'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT 0.2-1.0 CONCRETE			GB	start drilling 6:00PM
	CL	FILL 1.0-2.5 SANDY CLAY: moderate brown and mottled; some fine sand; stiff moist				
2				3	DR	0.5/1.0
			C-1	3		
	SC	2.5-5.0 CLAYEY SAND: brownish black; moderately plastic fines; loose; moist			GB	
4						
			J-1	1	SS	
	CL	5.0-6.5 SANDY CLAY: mottled browns and blacks; very stiff; moist		5		
				14		1.5/1.5
6					GB	
	CL	OLD ALLUVIUM 6.5-18.5 SILTY CLAY: moderate yellowish brown mottled; very stiff; moist; interbeds of sandy clay to 8" to thickness		10		1.0/1.0
			C-2	31	DR	
8					RD	set tub and cased to 8.5'
			J-2	10		
				12	SS	1.5/1.5
10				21		
					RD	
12				6	DR	1.0/1.0
			C-3	13		
					RD	
14			J-3	7	SS	
				20		1.5/1.5
				27		
16					RD	casing leaking drove to 12.5'
				12	DR	
18			C-4	23		1.0/1.0
					RD	
	CL	18.5-21.5 SILTY CLAY: dk. greenish grey;		9	SS	1.5/1.5
20			J-4	15		

Sheet 1 of 4

Project DESIGN UNIT A-220 Date Drilled 9-19-20-83 Hole No. 15-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	18.5-21.5 SILTY CLAY: cont. fine sand; v. stiff; moist		20	RD	7:30 PM 9-19-83 7:00 AM 9-20-83
22	SC	21.5-24.8 CLAYEY SAND: dark greenish grey; plastic fines; dense; moist	C-5	20 35	DR	1.C/1.0
					RD	
24			J-5	25 35 37	SS	1.0/1.5
	CL	24.8-26.0 SILTY CLAY:				
26					RD	
	ML	26.0-37.4 SILT: dark greenish grey; non-plastics fines; stiff; moist	C-6	14 30	DR	1.0/1.0
28	CL	grading to silty clay				
					RD	
30		grading back to silt	J-6	15 23 35	SS	1.5/1.5
					RD	
		grading to clayey silt				
32		sulfur odor	C-7	14 32	DR	1.0/1.0
					RD	
34			J-7	12 17 31	SS	1.0/1.5
36					RD	
		grading to	C-8	20 39	DR	1.0/1.0
38	SC/SM	37.4-38.5 CLAYEY SAND/SILTY SAND INTER-BEDDED: dark greenish grey; mostly fine to med sand;very dense; moist				
					RD	
	ML	38.5-41.0 CLAYEY SILT: dark greenish grey; moderately plastic fines; very stiff; moist	J-8	13 27 42	SS	1.5/1.5
40						
					RD	
	CL	41.0-57.5 CLAY: dark greenish grey; very stiff; moist				
42			C-9	33 55	DR	1.0/1.0
44					RD	

Project DESIGN UNIT A-220 Date Drilled 9-19-20-83 Hole No. 15-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL	41.0-57.5 CLAY: cont.	J-9	11 / 20 / 35	SS	1.5/1.5
46					RD	
			C-10	27 / 48	DR	1.0/1.0
48		sandy silt interbeds to 8" thick			RD	
			J-10	13 / 18 / 25	SS	1.5/1.5
50		becoming less plastic			RD	
52			C-11	25 / 50-5.5"	DR	1.0/1.0
		silty sand interbed			RD	
54		sandy clay interbed	J-11	10 / 17 / 24	SS	1.5/1.5
56					RD	
			C-12	26 / 42	DR	1.0/1.0
58	SP	57.5-60.0 SAND: dark greenish grey; mostly medium sand; very dense; wet			RD	
			J-12	22 / 23 / 33	SS	1.5/1.5
60	CL	60.0-66.8 SANDY CLAY: dark greenish grey; fine sand; very stiff; moist			RD	
62			C-13	18 / 20	DR	1.0/1.0
					RD	
64		clayey sand lenses	J-13	15 / 17 / 30	SS	1.5/1.5
66					RD	
	SP	SAN PEDRO FORMATION 66.8-84.9 SAND: dark greenish grey;	C-14	50 / 50-5.5"	DR	0.9/1.0
68						

Sheet 3 of 4

Project DESIGN UNIT A-220 Date Drilled 9-19-20-83 Hole No. 15-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	66.8-84.9 SAND: cont. mostly fine sand; trace silt; very dense; saturated			RD	
70			J-14	23 39 52	SS	1.0/1.5
					RD	
72			C-15	63 50-	DR 4"	0.8/0.8
					RD	
74			J-15	8 23 50-	SS 4"	1.0/1.3
76					RD	
				77 72-	DR 2"	fell out had to drive out sampler 0.0/0.7
78					RD	
		contains some coarse sand and gravel	C-16	41 50-4	DR .5"	0.7/0.9
80					RD	
			J-16	24 50-4	SS .5"	0.3/0.9
82					RD	
84		gravel increase	C-17	38 50-4	DR .5"	
		B.H. 84.9' Terminated hole; tremied grout to surface				completed drilling 12:45
86						
88						
90						
92						

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BORING LOG 15-5

Proj: DESIGN UNIT A-220 **Date Drilled** 9-20-21-83 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 93.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	conc	0.0-0.6 CONCRETE			GB	start drilling 3:30
	baserock	0.6-1.0 BASEROCK				
	CL	FILL 1.0-2.6 SANDY CLAY: light brown				
2				8	DR	0.5/1.0
	SC	2.6-3.4 CLAYEY SAND: yellowish grey	C-1	16		
					AD	
	CL	3.4-4.8 SANDY CLAY:				
4			J-1	6	SS	1.5/1.5
				18		
	SM	4.8-10.0 SILTY SAND: yellowish grey; non-cohesive; medium dense		17		
6					RD	set tub and cased to 6.5'
		gravelly				
				23	DR	0.0/1.0
8				36		
					RD	
				13	SS	
10				15		0.0/1.5
	CL	10.0-14.2 SANDY CLAY: moderate brown; sand and gravel; soft; wet; contains other materials, brick fragments		10		
					RD	
				3	DR	1.0/1.0
12			C-2	3		
					RD	
			J-2	5	SS	0.4/1.5
14				10		
				18		
	GC	14.2-16.5 CLAYEY GRAVEL: variable color; mostly fine to medium gravel, with sand and clay			RD	
16				13	DR	1.0/1.0
	CL	16.5-22.5 CLAY: dark greenish grey; very stiff; moist	C-3	23		
					RD	
18			J-3	8	SS	1.3/1.5
				18		
				25		
20					RD	

Project DESIGN UNIT A-220 Date Drilled 9-20-21-83 Hole No. 15-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	16.5-22.5 CLAY: cont.			RD	
				20	DR	1.0/1.0
22			C-4	32		pocket pen > 4.5 tsf
					RD	
	CL	22.5-31.6 SANDY CLAY: dark greenish grey; fine sand; very stiff; moist	J-4	9	SS	1.4/1.5
24				18		
				34		
					RD	
26				15	DR	0.8/1.0
			C-5	26		
					RD	
28			J-5	10	SS	
				23		
				32		6:00 PM 9-20-83
30					RD	7:00 AM 9-21-83 water @ 11'
				40	DR	0.8/0.8
			C-6	50-4"		
32	ML	31.6-35.5 CLAYEY SILT: dark greenish grey; low plastic fines; sand; very stiff; moist			RD	gravel falling in from 14'
			J-6	14	SS	1.0/1.0
34				56		
					RD	
36	CL	35.5-42.5 SANDY CLAY: dark greenish grey; fine to medium sand; very stiff; moist		39	DR	0.8/0.8
			C-7	50-4"		pocket pen > 4.5 tsf
					RD	
38		silty sand lenses to 6" thick	J-7	14	SS	1.0/1.5
				44		
				47		
40					RD	
				40	DR	1.0/1.0
			C-8	57		
42					RD	
	CL	42.5-52.5 SILTY CLAY: dark greenish grey; very stiff; moist	J-8	11	SS	
44				19		

Project DESIGN UNIT A-220 Date Drilled 9-20-21-83 Hole No. 15-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	CL	42.5-52.5 SILTY CLAY: cont.		27	RD	
46			C-9	39 50-5"	DR	1.0/1.0 pocket pen > 4.5 tsf
					RD	
48			J-9	15 20 24	SS	1.5/1.5
50					RD	
			C-10	19 54	DR	1.0/1.0
52		grading to			RD	
	CL	52.5-57.8 SANDY CLAY: dark greenish grey; fine to medium sand; very stiff; moist; contains sand lenses to 3" thick	J-10	9 31 25	SS	1.5/1.5
54					RD	
56			C-11	15 34	DR	1.0/1.0
					RD	
58	SP	57.8-60.5 SAND: dark greenish grey; mostly fine to medium sand with silt or clay; dense; wet; contains 2-3" lenses of sandy clay	J-11	15 31 36	SS	1.3/1.5
60					RD	turned rope around
	SC	60.5-67.5 CLAYEY SAND: dark greenish grey; mostly fine sand; plastic fines; very dense; moist to wet; contains lenses of sand	C-12	32 50-4"	DR	0.8/0.8
62					RD	
64			J-12	10 13 21	SS	1.3/1.5
					RD	
66		6" sand lens, sulfur odor	C-13	31 45	DR	0.7/1.0
					RD	
68	SP	SAN PEDRO SAND				

Project DESIGN UNIT A-220 Date Drilled 9-20-21-83 Hole No. 15-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	67.5-90.0 SAND: dark greenish grey; mostly fine sand with silt; very dense; saturated; strong sulfur odor	J-13	25 / 43 / 47	SS	1.0/1.5
70					RD	
			C-14	36 / 50-4.5"	DR	0.8/0.8
72					RD	
			J-14	28 / 52	SS	1.0/1.0
74					RD	
76			C-15	54 / 50-2.5"	DR	0.6/0.7
					RD	
78			J-15	41 / 50-3.5"	SS	0.8/0.8
					RD	
80						
		grading coarser w/occasional gravels	C-16	104	DR	0.5/0.5
82					RD	
			J-16	60 / 50-3.5"	SS	0.7/0.7
84					RD	
86						
88						
		6" of coarse gravel				
90		PUENTE FORMATION 90.0-93.7 INTERBEDDED SILTSTONE, CLAYSTONE and SANDSTONE: dark greenish grey; olive grey, and pale green;				
92	30°					

Sheet 4 of 5

Project DESIGN UNIT A-220 Date Drilled 9-20-21-83 Hole No. 15-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		90.0-93.7 INTERBEDDED SILTSTONE, CLAYSTONE and SANDSTONE: cont. sand very fine grained; thinly bedded 1/8-3"; sulfur odor	C-17	57 60-3"	RD DR	0.7/0.7
94		B.H. 93.7' Terminated at extended depth, looking for bedrock surface. Tremied grout to ground surface				complete drilling 2:15
96						
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 16

Proj: DESIGN UNIT A-240 **Date Drilled** 1-20-27-81 **Ground Elev.** 211'

Drill Rig Failing 1500 **Logged By** Gallinatti **Total Depth** 199.2'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 CEMENT			AD	Begin drilling 11:30 1/20/81
	SM	OLD ALLUVIUM 0.5-9.0 SILTY SAND: light olive brown; poorly graded fine sand, moist; subangular; loose;				
4						install 5' of 5" surface casing
5					RD	
9	CL	9.0-12.0 SANDY CLAY: moderate yellowish brown; fine to medium sand; very stiff; moist				
10			J-1	22 50	SS	0.9/0.9 recovery
11					RD	install 5' more of surface casing
12	CH	12.0-20.0 CLAY: dusky yellow; firm; moist				
15			J-2	8 11 21	SS	1.5/1.5 recovery
16.5			S-1		PB	
18						2.8/2.8 recovery
20			Box 1			

Sheet 1 of 9

Project DESIGN UNIT A-240 Date Drilled 1-20-27-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	20.0-29.5 SILTY SAND: pale yellowish brown mostly medium sand; little fines loose to medium dense; moist	Box 4		PB	2.8/2.9 recovery
		21.0-22.0 fine sand				
22						2.7/2.8 recovery
		23.0-24.0 fine sand				
24						
						2.0/2.8 recovery
26						
28						2.1/2.8 recovery pocket penetrometer 1.5/3.25 2/9/81
	SM	29.5-32.0 SILTY SAND: medium blueish gray; fine sand with little fines; loose; moist to wet	Box 2			
30						2.7/2.8 recovery
32	CL	32.0-45.4 CLAY: dark greenish gray; stiff; moist				pocket penetrometer 1.0 2/9/81
						2.8/2,8 recovery
34			S-2			
36			Box 2 (cont)			2.7/2.8 recovery
		37.2-37.8 Clayey Sand: medium sand				
38						
		39.0-42.0 some inclusions of cemented material				2.8/2.8 recovery
40						
42						2.8/2.8 recovery pocket penetrometer >4.5 2/9/81
44			Box 3			

Project DESIGN UNIT A-240 Date Drilled 1-20-27-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	32,0-45.4 CLAY: (continued)	Box 3		PB	2.8/2.8 recovery
46	SM	45.4-51.0 SILTY SAND: dark greenish gray; poorly graded fine sand; little fines; medium dense to loose; moist				
48						2.4/2.8 recovery pocket penetrometer 2.25 2/9/81
50						2.8/2.8 recovery
52	CL/SC	51.0-56.2 CLAY AND CLAYEY SAND: dark greenish gray; CLAYEY SAND lenses are medium to coarse sand	S-3			
54			Box 3 (cont)			2.5/2.8 recovery
56	CL	56.2-60.0 CLAY: dark greenish gray; with trace of medium, angular sand; stiff; moist	Box 4			2.1/2.8 recovery
58						1.7/2.8 recovery
60	SM	SAN PEDRO FORMATION 60.0-85.0 SILTY SAND: dark greenish gray; fine sand with little fines; loose; wet				1.8/2.8 recovery
62						
64						pocket penetrometer 3.5 2/9/81
65						65' water table
66						0.6/2.8 recovery
68						

Sheet 3 of 9

Project DESIGN UNIT A-240 Date Drilled 1-20-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SM	60.0-85.0 SILTY SAND: (continued)			PB	No Recovery - saturated sands
70			S-4			1.5/2.8 recovery
72						
			S-5			.9/2.8 recovery
74						
		75'-77' medium to coarse sand				
76			Box 4 (cont)			1.8/2.8 recovery
78						
						No Recovery
80						
		81'-85' gravelly sand: probably basal conglomerate			RD	81.3'-stop drilling 1/20/81 begin rotary drilling 1/21/81
82						
84						
		FERNANDO FORMATION				
86		85.0-96.4 SILTSTONE: moderate brown; moist	Box 4 (cont)		PB	1.1/2.8 recovery
		Physical Condition: massive; low hardness; friable; fresh				
88						2.8/2.8 recovery
			Box 5			
90						
	↘ 40°	91'-96.4' some thin sandy siltstone lenses				
92						

Sheet 4 of 9

Project DESIGN UNIT A-240 Date Drilled 1-21-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		FERNANDO FORMATION 85.0-96.4 SILTSTONE: (continued)	Box 5		PB	2.8/2.8 recovery
94	40°					2.5/2.8 recovery pocket penetrometer >4.5 2/9/81
96		96.4-105.0 SILTSTONE with interbedded CLAYSTONE and SILTY SAND: dark moderate brown and greyish brown				2.8/2.8 recovery
98	30°		Box 6			
100		Physical Conditions: massive; low hardness; friable; fresh	S-6			2.8/2.8 recovery
102	30°		Box 6 (cont)			2.8/2.8 recovery
104		105.0-135.0 SILTSTONE: dark moderate brown; moist				
106		Physical Condition: massive; low hardness; friable; fresh				2.7/2.8 recovery
108						pocket penetrometer > 4.5 2/9/81 2.8/2.8 recovery
110		111'-114' some very thin silty sandstone layers and claystone layers (0.05' - 0.1 Thick)	Box 7			2.8/2.8 recovery
112	35°					
114						2.8/2.8 recovery
116						

Project DESIGN UNIT A-240 Date Drilled 1-21-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		FERNANDO FORMATION 105.0-135.0 SILTSTONE: (continued)	Box 7 S-7		PB	2.8/2.8 recovery
118						
			Box 7 (cont.)			pocket penetrometer >4.5 2/9/81 2.8/2.8 recovery
120		120.1-120.3: SILTY SANDSTONE: light brown; mostly fine sand with some fines				
122						2.8/2.8 recovery
		123.0-135: occasional very thin SILTY SANDSTONE layers	Box 8			
124						2.8/2.8 recovery
126	40°					
			S-8			2.0/2.8 - bent tube requires pipe cutter before extruding
128						
130	35°	becoming more competent with depth	Box 8 (cont.)			1.5/2.5 recovery (cemented material)
132				RD		RD to 133'
			S-9	PB		2.2/2.3 recovery
134						134' - Gas Test no gas
		135.0-169.5 SILTSTONE: as above, although gradually more competent Physical Condition: massive, low hardness, friable, fresh	Box 8 (cont.)			2.6/2.7 recovery pocket penetrometer >4.5 2/9/81
136						
138	35°					2.0/2.1 recovery
140			Box 9			

Project DESIGN UNIT A-240 Date Drilled 1-20-27-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		135.0-169.5 SILTSTONE: (cont.)	Box 9		PB	2.0/2.0 recovery
142		SILTY SANDSTONE layers				1.9/2.0 recovery
144						pocket penetrometer <4.5 2/9/81 2.0/2.0 recovery
146			S-10			2.5/2.5 recovery
		Physical Condition: massive, low hardness, friable to weak, fresh				
148			Box 9 (cont.)			2.3/2.8 recovery
		149.7 very thin volcanic ash layer				
150						1.9/2.0 recovery
152						
154			Box 10			2.8/2.8 recovery
156						2.7/2.8 recovery
	30°	157.2 very thin layer of SANDY SILTSTONE				
158						pocket penetrometer <4.5 2/9/81
						2.3/2.8 recovery
160						
162			S-11			2.2/2.2 recovery 164' - stop 1/21/81 - begin 1/26 - clean out hole to 164'
164						

Project DESIGN UNIT A-240 Date Drilled 1-20-27-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164		135.0-169.5 SILTSTONE: (cont.)	Box 10 / Box 11		PB	2.7/2.8 recovery pocket penetrometer >4.5 2/9/81
166						2.1/2.8 recovery
168		169.5-189.0 SILTSTONE with interbedded CLAYSTONE and SILTY SANDSTONE: very thinly bedded; claystone is greyish brown; silty sandstone is light blueish grey with mostly fine sand and little fines				2.8/2.8 recovery
170	40°					
172	45°	Physical Condition: massive, low hardness, friable, fresh				2.3/2.8 recovery
174			Box 12 / S-12			2.2/2.2 recovery
176						
178			S-13			2.8/2.8 recovery
180		180.0-181.5 SILTSTONE: (not interbedded)	Box 12 cont.			2.8/2.8 recovery pocket penetrometer >4.5 2/9/81
182						2.4/2.8 recovery
184	40					
186	30					2.3/2.8 recovery
188						

Sheet 8 of 9

Project DESIGN UNIT A-240 Date Drilled 1-20-27-81 Hole No. 16

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		169.5-189.0 SILTSTONE with interbedded CLAYSTONE and SILTY SANDSTONE:.	Box 12		PB	1.9/2.2 recovery
		189.0-199.2 SILTSTONE: dark moderate brown; moist Physical Condition: massive; low hardness, friable, fresh				
190			Box 13			2.8/2.8 recovery
192						
194			S-14			1.2/2.8 recovery
196		196.0-199.0 some SANDSTONE AND CLAYSTONE interbeds				
	30°		Box 13 cont.			2.8/2.8 recovery pocket penetrometer >4.5 2/9/81
198						
200		Bottom of hole				199.2' Terminate hole: 4:00 1/26/81 run electric logs 1/26/81. 1/27/81 - install piezometers; 2" PVC from 200' to surface w/cloth covered perforations from 160' to 195'. Gravel pack to 87'. Bentonite plug from ~75' to 87'. 1" PVC piezometer from 80' to surface w/perforations from 40' to 75'. Gravel pack to 5'. Bentonite surface plug & surface cap. Clean-up site and move off 12:00 1/27/81 water sampled 2" & 1" 2/18/81
202						
204						
206						
208						
210						
212						

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BORING LOG 16A

Proj: DESIGN UNIT A-240 **Date Drilled** 2-22-83 **Ground Elev.** 212.0'

Drill Rig B. Auger **Logged By** Dan Gillette **Total Depth** 72.0'

Hole Diameter 36" **Hammer Weight & Fall** NA

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	SC	YOUNG ALLUVIUM 0.0-10.0 CLAYEY SAND: grayish orange; medium sand; moist; medium dense	NO SAMPLES REQUIRED OBSERVATION HOLE			0.0-33.0 hole stands well
5.0		5.0-6.0 Sand lens				
10	CL	OLD ALLUVIUM 10.0-21.0 SANDY CLAY: pale yellowish brown; with fine sand; moist; stiff				
20		20.0 light brown streaks				

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Project DESIGN UNIT A-240 Date Drilled 2-22-83 Hole No. 16A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	10.0-21.0 SANDY CLAY: (cont.)				
	SW	21.0-31.0 SAND: moderate brown; contains mostly fine sand; moist to very moist; medium dense				
22						
24						Ground Water Data - recovery 10:15 - 51.5' 10:35 - 50.0' 11:45 - 46.6' 12:40 - 44.5' 145 min. 260 gal. $Q=\frac{260}{145}= 1.8$gpm
26						
28						
		29.0 medium gray streaks				
30						
	SM/ML	31.0-35.0 SANDY SILT: dark gray; silt and fine sand; firm; very moist to wet; has strong sulfur odor				30-33' hole ravels slightly due to seep at 32.0'
32						33-42' hole stands well
34						
	CH	35.0-42.0 SANDY CLAY: dark gray; some medium and coarse sand; firm; moist				
36						
38						
	WL	39.0			WL	39.0
40						
42						
	SP/SM	SAN PEDRO FORMATION 42.0-51.0 SILTY SAND: med. dark grey; fine sand; dense; moist				
44						

Project DESIGN UNIT A-240 Date Drilled 2-22-83 Hole No. 16A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SP/ SM	42.0-51.0 SILTY SAND: (cont.) 43.5-45.0 little fine gravel				
46						
48						
50						
	CL	51.0-53.0 SANDY CLAY: dusky blue green; little medium sand; firm; moist				
52						
	SC	53.0-72.0 CLAYEY SAND: medium dark grey; dense; medium sand; very moist;				
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 4

Project DESIGN UNIT A-240 Date Drilled 2-22-83 Hole No. 16A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SC	53.0-72.0 CLAYEY SAND: (cont.)				
72		End Boring 72.0'				SPECIAL HOLE CLOSURE Note: stopped drilling due to excessive belling backfill - placed pea gravel to 30' then slurry to 1' then concrete cap to side-walk grade

Sheet 4 of 4

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BORING LOG 16B

Proj: DESIGN UNIT A-220 | Date Drilled 11-7-83 | Ground Elev. 200'

Drill Rig MAN-SIZE AUGER | Logged By J. Stellar | Total Depth 60.0

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		A/C PAVEMENT 0.0-0.6				
	ML	ALLUVIUM 0.6-6.0 SANDY SILT: light yellowish brown, slightly moist, stiff				
6	ML	6.0-12.0 CLAYEY SILT: medium brown, moist stiff, streaks of sandy silt				
12	SM	12.0-13.0 SILTY SAND: light greenish gray, moist, medium dense				extensive caving 13'-22' (8'-10' belling) water level @ 13' after 2 hours
13	ML	12.0-15.0 SILT: light greenish gray, moist, stiff, with layers of silty sand and sandy silt				very rapid flow into hole 50± gpm
15	SP	15.0-22.0 SAND: mottled orange and gray, wet, clean, medium grained, medium dense				
18						bag sample @ 18' H20 @ 20' after 1 hour
19		1' layers of gravelly sand w/ gravel to 1"				
20						

Sheet 1 of 3

Project DESIGN UNIT A-220 Date Drilled 11-7-83 Hole No. 16B

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	15.0-22.0	SAND: (continued)				
22	ML	22.0-25.0	SILT: light yellowish brown, moist firm with layers of sandy silt				
24							
	SP	25.0-29.0	SAND: light yellowish brown, wet, medium dense, alternating with layers of gravelly sand and silty sand				
26							
28							
	ML	29.0-36.0	SANDY SILT: greenish blue, very moist, stiff with layers of sandy silt and sand				
30							
32							bag sample @ 32'
34			6" layers of clayey silt clay, and clayer sand				
36	SM	36.0-38.0	SILTY SAND: greenish blue, wet, dense, with layers of sandy silt				
38	ML	38.0-42.0	SANDY SILT: greenish blue, very moist, firm to stiff, w/ layers of sand, silty sand, and gravelly sand				
40							
42	SM	42.0-47.0	SILTY SAND: greenish blue, wet, dense, with layers of sand, and sandy silt				
44							

Project DESIGN UNIT A - 220 Date Drilled 11-7-83 Hole No. 16-B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SM	42.0-47.0 SILTY SAND: (continued)				
47	SP	47.0-51.0 SAND: greenish blue, wet, dense with layers of silty sand				bag sample @ 48'
51	ML	51.0-55.0 SANDY SILT: greenish blue, wet, stiff, with layers of silty sand and sand				
55	SP	SAN PEDRO FORMATION 55.0-60.0 SAND: greenish blue, wet, dense with layers of silty sand and sandy silt, coarse grained				flow into holes from saturated sands. (quantity undetermined)
60		B.H. 60.0' Terminated hole. Hole caved back to 49.0' after 2 hours and to 20' after 4 hours. No Gas No downhole observations due to water and caving.				

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

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BORING LOG 16-1

Proj: DESIGN UNIT A-240 **Date Drilled** 10-1-83 **Ground Elev.** 199.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 95.5

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.5 ASHALT			GB	start drilling 10:30
	GP	0.5-0.7 ROAD BASE				
	CL	FILL 0.7-5.0 SANDY CLAY: moderate to dark yellowish brown; some fine sand, occasional gravel and cobbles; contains asphalt pieces; soft to firm; moist to wet				
2			C-1	5 9	DR	0.9/1.0
					AD	
4						
	CL	OLD ALLUVIUM 5.0-15.2 SANDYCLAY: moderate brown to moderate yellowish brown; trace fine sand; firm; wet				
6			SH-1		SH	2.3/2.5 pushed Shelby
8		interbedded clayey sand lenses, moderate yellowish brown	J-1	5 5 6	SS	1.5/1.5
					AD	
10						
12			C-2	6 10	DR	1.0/1.0
					RD	set tub & cased to 13', mixed 1 sack mud
14						
	SM/SP	15.2-27.0 SILTY SAND/SAND: dusky yellow fine sand; trace silt; very dense; moist; occasional gravel lenses and silty clay lenses				rig chatter
16						
	(CL)	15.2-15.7 gravel lens, silty clay lens	PB-1		PB	2.5/2.5
18						
			J-2	21 36 47	SS	
20						

Project DESIGN UNIT A-240 Date Drilled 10-1-83 Hole No. 16-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM/SP	15.2-27.0 SILTY SAND/SAND: (Cont.)			RD	
22		little silt; some mottling with moderate brown-ferrous staining	C-3	16 28	DR	1.0/1.0
23					RD	
24						
25			PB-2		PB	2.5/2.5
26						
27	CL	27.0-32.8 SILTY CLAY: dark greenish grey, stiff; moist; occasional sand				
28			J-3	7 14 18	SS	1.3/1.5
29.5					RD	
30						
32		some lighter grey cemented nodules	C-4	38 50	DR	1.0/1.0 pocket pen >4.5
33	CL	32.8-61.0 SANDY CLAY: dark greenish grey little fine sand, very stiff; moist.			RD	
34						
35					PB	2.5/2.5
36		thin cemented zone	PB-3			
38		1' well cemented zone	J-4	50-	4"SS	0.3/0.3 intense chatter
					RD	
40	(SM)	thinly interbedded silty sand with sandy clay, numerous cemented nodules of both sand and clay				
42			C-5	21 53	DR	1.0/1.0
43					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A-240 Date Drilled 10-1-83 Hole No. 16-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	32.8-61.0 SANDY CLAY: (cont.)			RD	
46			PB-4		PB	slight chatter 0.9/2.5 bent tube
48			J-5	15 31 50	SS	1.5/1.5
50					RD	
		51.0 sand				
52	(SP) (SM)	interbedded sand, gravelly sand, silty sand, silty clay, sandy clay with well cemented zones	C-6	39 49	DR	1.0/1.0
54					RD	
56			PB-5		PB	1.6/2.5
58			J-6	8 20 26	SS	1.5/1.5
60					RD	
	SP	SAN PEDRO FORMATION 61.0-79.5 SAND: dark greenish gray; mostly fine sand; trace silt; occasional gravels; very dense; wet				
62			C-7	75 50-3"	DR	0.7/0.7
64					RD	
66			PB-6		PB	0.6/2.5 tube bent
68						

Sheet 3 of 5

Project DESIGN UNIT A-240 Date Drilled 10-1-83 Hole No. 16-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	61.0-79.5 SAND: (cont.)	J-7	55	SS	0.5/0.5
					RD	
72			C-8	49 60-3"	DR	0.7/0.7
					RD	
75		grading coarser w/some fine gravel	PB-7		PB	2.3/2.5
78				37 50	SS	0.0/1.0 intense chatter
79		6" gravelly lens			RD	
80		PUENTE FORMATION 79.5-95.5 INTERBEDDED SILTSTONE AND CLAYSTONE: olive grey and dark greenish grey; very thinly to thickly interbedded; weakly cemented in places Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh				
82			C-9	24 50-5"	DR	0.9/0.9
					RD	
86			PB-8		PB	2.5/2.5
88					RD	
90		weakly cemented	C-10	39 50-3.5"	DR	0.8/0.8
					RD	
92						

Project DESIGN UNIT A-240 Date Drilled 10-1-83 Hole No. 16-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		79.5-95.5 INTERBEDDED SILTSTONE and CLAYSTONE: (cont.)			RD	
94			PB-9		PB	2.3/2.5
96		B.H. 95.5' Terminated hole, grouted to surface				complete drilling 5:30
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 16-2

Proj: DESIGN UNIT A-240 Date Drilled 10-2-3-83 Ground Elev. 203'
Drill Rig Failing 1500 Logged By L. Schoeberlein Total Depth 96.5'
Hole Diameter 4 4/8" Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	conc	0.0-1.0 CONCRETE			GB	start drilling 3:30
	CL	OLD ALLUVIUM 1.0-6.5 SANDY CLAY: dusky yellow brown; little fine sand; firm; moist				
2			C-1	3 9	DR	1.0/1.0
					AD	
4						
6			SH-1		SH	2.4/2.5
	CL	6.5-9.0 SANDY CLAY: moderate brown; some fine sand; very stiff; moist				
8			J-1	5 14 17	SS	1.5/1.5
	SC	9.0-11.0 CLAYEY SAND: moderate brown; mostly well graded sand; some fines; occasional fine gravel; dense; moist			AD	
10						
	CL	11.0-12.5 SILTY CLAY: dusky yellow; stiff moist				
12			C-2	11 39	DR	
	CL	12.5-15.0 SANDY CLAY: dusky yellow; mostly fines with little fine sand; very stiff; moist			RD	set up tub & cased to 13.5' mixed mud
14						
	SM/SP	15.0-21.5 SILTY SAND: dusky yellow; mostly fine sand; trace fines; dense; moist	PB-1		PB	2.7/2.7
16						
18			J-2	10 13 25	SS	1.5/1.5
20					RD	

Sheet 1 of 5

Project DESIGN UNIT A-240 Date Drilled 10-2-3-83 Hole No. 16-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	15.0-21.5 SILTY SAND: (cont.)			RD	
22	SP	21.5-26.5 SAND: dusky yellow; fine sand; trace of silt; dense; moist	C-3	22 39	DR	1.0/1.0
24					RD	
26			PB-2		PB	2.5/2.5
	CL	26.5-30.0 SILTY CLAY: dark greenish grey; very stiff; moist; contains cemented nodules				
28			J-3	8 12 17	SS	1.5/1.5
30	SP	30.0-32.8 SAND: dark greenish grey; fine to medium sand; trace of silt; dense; wet			RD	
	(CL)					
32	(SM)	sandy clay lens, silty sand lens	C-4	34 42	DR	1.0/1.0 5:30 10/2/83 7:00 10/3/83
	CL	32.8-37.0 SILTY CLAY: dark greenish grey; very stiff; moist; contains cemented nodules			RD	
34						
36		sandy clay	PB-3		PB	2.7/2.7
	ML/SM	37.0-41.5 SILT/SILTY SAND: dark geeenish grey; very stiff; moist; contains cemented nodules				
38			J-4	15 32 46	SS	1.5/1.5
40					RD	
42	SM	41.5-44.0 SILTY SAND: dark greenish grey fine sand; moist; very dense; contains cemented nodules up to 2"	C-5	32 52	DR	1.0/1.0
44					RD	

Project DESIGN UNIT A-240 **Date Drilled** 10-2-3-83 **Hole No.** 16-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC/CL	44.0-57.0 SANDY CLAY/CLAYEY SAND: dark greenish grey; thin layers; hard to dense			RD	
					PB	2.3/2.5
46			PB-4			
48			J-5	12 23 32	SS	1.5/1.5
50					RD	
52			C-6	22 35	DR	1.0/1.0
					RD	
54						
56			PB-5		PB	2.7/2.7
	CL	57.0-58.8 CLAY: dark greenish grey; hard; moist				
58			J-6	8 46 50-4"	SS	1.3/1.3
	SP	SAN PEDRO FORMATION 58.8-76-5 SAND: dark greenish grey; mostly fine sand; trace of silt; very dense; wet; occasional coarse sand and gravelly inclusions			RD	
60						
62			C-7	75 76-½	DR "	0.5/0.5 disturbed
					RD	
64					PB	
66					PB	2.5/2.5
68		some coarse sand	PB-6			

Project DESIGN UNIT A-240 Date Drilled 10-2-3-83 Hole No. 16-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	58.8-76.5 SAND: (cont.)	J-7	57	SS	0.4/0.5
					RD	
70		gravelly lens				
72			C-8	125	DR	0.3/0.5
					RD	sample disturbed
74						
76			PB-7		PB	2.5/2.5
		PUENTE FORMATION 76.5-96.5 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; thinly to thickly bedded; occasional fine sandstone lenses Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh becoming more massive; olive grey clayey siltstone				
78					RD	
80			C-9	40	DR	0.7/0.7
				60-3	"	
82					RD	
84						
86			PB-8		PB	2.5/2.5
88					RD	
90	30°	increased bedding	C-10	36	DR	0.7/0.7
				50-3	"	
92					RD	

Sheet 4 of 5

Project DESIGN UNIT A-240 Date Drilled 10-2-3-83 Hole No. 16-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		76.5-96.5 INTERBEDDED SILTSTONE and CLAYSTONE: (cont.)			RD	
94			PB-9		PB	
96						
		B.H. 96.5' Terminated hole, installed piezometer to bottom, 76-96' slotted.				complete drilling 12:00
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 16-3

Proj: DESIGN UNIT A-240 Date Drilled 10-2-83 Ground Elev. 207'

Drill Rig Failing 1500 Logged By L. Schoeberlein Total Depth 96.5

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.0-0.2 ASPHALT			GB	start drilling 8:00
	GP	0.2-1.0 BASE ROCK				
	CL	OLD ALLUVIUM				
2		1.0-12.0 SANDY CLAY: moderate brown; little to trace of fine sand very stiff; moist;		5	DR	0.8/1.0
			C-1	11		
					AD	
4						
					PB	pushed Shelby 2.5/2.5
6			S-1			
8		color change to light olive grey, becomes hard		11	SS	1.5/1.5
			J-1	23		
				26		
					AD	
10						
12				8	DR	0.8/1.0
	CL	12.0-15.0 SILTY CLAY: greyish green mottled with light brown; ferrous staining; hard; moist; occasional cemented nodules and sand	C-2	25		
					RD	set tub and drove casing to 13' mixed mud
14						
	SC	15.0-19.0 CLAYEY SAND: dark greenish grey; mostly fine sand, little fines; interbeds of silty sand; dense; moist				
16	(SM)		PB-1		PB	2.5/2.5
18				7	SS	1.5/1.5
			J-2	14		
				19		
	CL	19.0-22.4 SANDY CLAY: dk. greenish grey			RD	
20						

Project DESIGN UNIT A-240 Date Drilled 10-2-83 Hole No. 16-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.0-22.4 SANDY CLAY: cont. fines, trace of fine sand; hard; moist			RD	
22				16	DR	0.9/1.0
	SM/SP	22.4-27.5 SILTY SAND: dark greenish grey; mostly fine sand, trace of fines; dense; moist	C-3	24		
					RD	
24		color change to dusky yellow	PB-2		PB	2.1/2.5
26						
28	ML	27.5-32.4 CLAYEY SILT: interbedded w/silt and silty sand; hard	J-3	9	SS	1.5/1.5
				17		
				20		
30					RD	
		color change to dark greenish grey; medium sand lens				
32				17	DR	pocket pen 4.0
	CL	32.4-41.0 SILTY CLAY: dark greenish grey; hard; moist	C-4	26		1.0/1.0
					RD	
34						
36	(SP)	fine sand lens	PB-3		PB	2.5/2.5
		weakly cemented				
38		moderately cemented nodules and occasional sand	J-4	13	SS	1.5/1.5
				30		
				54		
40					RD	
	ML	41.0-51.0 SILT: dark greenish grey; occasional silty sand lenses; hard; moist to wet; contains occasional cemented nodules				
42				33	DR	1.0/1.0
			C-5	50		
44					RD	

Project DESIGN UNIT A-240 Date Drilled 10-2-83 Hole No. 16-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	41.0-51.0 SILT: cont.			RD	
		45' sand lens ~ 1.5' thick				
46			PB-4		PB	1.9/2.3
						rig chatter
		47.8' sand lens ~ 1.3' thick				
48			J-5	30	SS	1.5/1.5
				46		
		cemented zone		48		
					RD	
50						
	CL/SC	51.0-62.8 SANDY CLAY/CLAYEY SAND: dark greenish grey; interbeds grade from mostly fine sand to mostly fines; very dense to hard; moist				
52			C-6	23	DR	1.0/1.0
				42		
					RD	
54						
56			PB-5		PB	2.1/2.5
58			J-6	16	SS	1.5/1.5
				29		
				41		
					RD	
60						
62			C-7	15	DR	0.9/1.0
				37		
	SP	SAN PEDRO FORMATION 62.8-80.5 SAND: dark greenish grey; fine sand, trace of silt; very dense; wet			RD	
64						
66			PB-6		PB	1.3/2.0
68				41	SS	

Project DESIGN UNIT A-240 Date Drilled 10-2-83 Hole No. 16-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	62.8-80.5 SAND: cont.	J-7	50-45	SS	0.5/0.8
					RD	
70						
72				97 60-3"	DR	no recovery
					RD	
74		coarse sand and fine gravel lens				occasional chatter
			PB-7		PB	1.2/1.5
76						
78			J-8	41 50-5"	SS	0.8/0.9
					RD	
80						
		PUENTE FORMATION 80.5-96.5 INTERBEDDED SILTSTONE and CLAY-STONE: olive grey and dark greenish grey; occasional fine sandstone lenses; thinly to thickly interbedded				
82			C-8	29 50-4"	DR	0.7/0.7
					RD	
84		Physical Condition: little fractured to massive, most occurring along the fine sand partings; friable hardness and strength; little weathered to fresh				
86			PB-8		PB	2.4/2.5
88					RD	
90		becoming more massive w/irregular fine sand inclusions				
			C-9	29 50-5	DR "	
92						

Project DESIGN UNIT A-240 Date Drilled 10-2-83 Hole No. 16-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		80.5-96.5 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94			PB-9		PB	2.5/2.5
96						
		B.H. 96.5 Terminated hole, tremied grout to surface				Completed drilling 2:15
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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

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BORING LOG 16-4

Proj: DESIGN UNIT A-240 **Date Drilled** 9-29-30-83 **Ground Elev.** 205.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 94.5'

Hole Diameter 4 4/8" **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 3 pm
	SM	OLD ALLUVIUM 0.8-6.0 SILTY SAND: moderate brown; fine to medium sand, little fines; loose to medium dense; moist; with thin gravelly lenses				
2			C-1	32 26	DR	0.7/1.0
					AD	
4			J-1	5 5 4	SS	1.3/1.5
					AD	
6	CL	6.0-6.8 SILTY CLAY: greyish brown; firm; moist to wet				set tub and cased to 7'
	SC	6.8-16.5 CLAYEY SAND: moderate brown; fine to medium sand, little fines; medium dense; moist		4 6	DR	0 recovery
8					RD	
			J-2	4 7 9	SS	0.8/1.5
10					RD	
12	(CL)	interbedded with sandy clay	C-2	3 5	DR	0.8/1.0
					RD	
		13.5' - sandy lens				
14		color change to yellowish grey becomes dense	J-3	7 13 19	SS	1.0/1.5
					RD	
16						
	SM/SP	increased sand content and decreased plasticity	C-3	9 21	DR	0.8/1.0
18		16.5-26.5 SILTY SAND: yellowish grey; fine sand, trace fines; dense; moist; occasional clayey sand lenses			RD	
			J-4	4 19	SS	1.0/1.5
20						

Project DESIGN UNIT A-240 Date Drilled 9-29-30-83 Hole No. 16-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	16.5-26.5 SILTY SAND: cont.		20	SS	
					RD	
22				5	DR	0.9/1.0
			C-4	15		
					RD	
24			J-5	4	SS	1.0/1.5
		becomes medium dense		7		
				12		
26		silt content increases			RD	mixed 1 sack mud losing circulation
	ML	26.5-28.5 SANDY SILT: yellowish grey; little fine sand; stiff; moist		11	DR	1.0/1.0
			C-5	10		
28					RD	
	CL	28.5-33.8 SILTY CLAY: dark greenish grey; occasional sand; very stiff; moist	J-6	3	SS	
				7		
30				12		5:00 9/29/83
					RD	8:00 9/30/83 H20 @ 24'
32			PB-1		PB	
34	CL	33.8-35.2 SANDY CLAY: moderate olive grey; mostly fine sand; hard; moist	J-7	7	SS	1.3/1.5
	CL	35.2-40.5 SILTY CLAY: dark greenish grey; occasional sand; hard; moist; occasional cemented nodules		12		
				22		
36					RD	
38				48	DR	lost all circulation add mud, drove casing to 36'
			C-6	61		1.0/1.0
40					RD	dewatering wells of adjacent building dewatering our hole,
	ML (SM)	40.5-46.5 CLAYEY SILT: dark greenish grey; hard; moist; interbeds of silty fine sand	PB-2		PB	cased to 39' 2.4/2.5
42						
44				16	SS	

Project DESIGN UNIT A-240 Date Drilled 9-29-30-83 Hole No. 16-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	40.5-46.5 CLAYEY SILT: cont.	J-8	34 50-4"	SS	1.2/1.3
					RD	
46						
	CL	46.5-49.5 SILTY CLAY: dark greenish grey; occasional sand; hard; moist; occasional cemented nodules				
			C-7	21 50-5.5"	DR	1.0/1.0
48					RD	
50	CL	49.5-52.0 SANDY CLAY: dark greenish grey; mostly fines, with some fine sand				
			PB-3		PB	2.4/2.5
52	SM (SC) (CL)	52.0-56.5 SILTY SAND: dark greenish grey; mostly fine sand, little silt; very dense; moist to wet; interbedded sandy clay and clayey sand				
54			J-9	20 50	SS	1.0/1.0
					RD	
56						
	ML	56.5-62.0 CLAYEY SILT: dark greenish grey; very stiff; moist				
58			C-8	17 34	DR	1.0/1.0
					RD	
60						
			PB-4		PB	2.5/2.5
62	SP	SAN PEDRO FORMATION 62.0-73.0 SAND: dark greenish grey; fine to medium sand, with trace of silt; very dense; wet				
64			J-10	46 53	SS	0.5/1.0
					RD	
66						
68			C-9	150	DR	0.4/1.0

Project DESIGN UNIT A-240 Date Drilled 9-29-30-83 Hole No. 16-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	62.0-73.0 SAND: cont.			RD	
70						
			PB-5		PB	1.8/2.5
72						
	GP	73.0-80.7 SANDY GRAVEL: dark greenish grey; mostly subrounded to subangular, fine to medium gravel, granitic origin with some fine to coarse sand; interbedded with graded sand lenses; very dense; wet				rig chatter
74			J-11	47 50-2	SS "	
	(SP)				RD	
76						
78		sand lens	C-10	78 50-2	DR "	0.4/0.7
					RD	
80						
		PUENTE FORMATION				
		80.7-94.5 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; thinly interbedded	PB-6		PB	2.4/2.5
82	~30°					
84		Physical Condition: little fractured to massive, fractures occur primarily along sandstone lenses; friable hardness and strength; little weathered to fresh	J-12	19 31 50-4.5"	SS	1.4/1.4
					RD	
86						
88		88.0' - 3 " well cemented zone	C-11	43 66-½"	DR	0.5/0.5
					RD	
90						
92			PB-6		PB	

Project DESIGN UNIT A-240 Date Drilled 9-29-30-83 Hole No. 16-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		80.7-94.5 INTERBEDDED SILTSTONE and CLAYSTONE: cont.	PB-6		PB	2.5/2.5
94			J-13	22 51	SS	1.0/1.0
		B.H. 94.5 Terminated hole, grouted bottom to 40', backfilled with pea gravel to surface				complete drilling 4:45
96						
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 16-5

Proj: DESIGN UNIT A-240 **Date Drilled** 10-3-4-83 **Ground Elev.** 211.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 99.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AC	0.9-0.2 ASPHALT			GB	start drilling 2:15
	CL	FILL 0.2-1.5 SANDY CLAY: brown, stiff; moist				
2	CL	1.5-10.0 SANDY CLAY: greyish green; some fine sand; stiff to hard; moist		7	DR	1.0/1.0
			C-1	11		
4					AD	
6		thinly bedded clayey sand	SH-1		SH	2.5/2.5
8			J-1	16 17 20	SS	1.5/1.5
10					AD	
	CL	10.0-11.5 SILTY CLAY: light olive brown; fines, trace fine sand; hard; moist				
12	CL	11.5-21.0 SANDY CLAY: mottled light olive brown with greyish green; little fine sand; hard; moist		13	DR	1.0/1.0
			C-2	37		set tube & cased to 13', mixed mud
14					RD	
16	SP	sand lens			PB	2.1/2.5
	CL		PB-1			
18			J-2	7 17 23	SS	1.5/1.5
20					RD	

Sheet 1 of 5

Project DESIGN UNIT A-240 Date Drilled 10-3-4-83 Hole No. 16-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	11.5-21.0 SANDY CLAY: cont.			RD	
	SC	21.0-25.0 CLAYEY SAND: light olive grey; dense; moist				
22			C-3	22 39	DR	1.0/1.0
					RD	
24						
	SP	25.0-27.0 SAND: light olive grey; fine to medium sand, trace of silt; medium dense; wet				
26			PB-2		PB	2.2/2.5
	ML	27.0-28.0 CLAYEY SILT: dusky yellow; very stiff; moist				
28			J-3	4 9 14	SS	1.5/1.5
	CL	29.0-31.5 SILTY CLAY: dusky yellow; very stiff; moist			RD	
30						
	ML	31.5-35.0 SANDY SILT: dusky yellow; occasional silty sand lenses; stiff; moist				
32			C-4	18 19	DR	1.0/1.0
					RD	
34						
	CL/SM/ML	35.0-52.0 SANDY CLAY/SILTY SAND/SILT: dark greenish grey; thin to medium layers; dense; moist; contains cemented nodules				
36			PB-3		PB	2.7/2.7
38			J-4	17 26 50	SS	1.5/1.5
40					RD	
42			C-5	36 50-5	DR "	0.9/0.9
					RD	
44						

Project DESIGN UNIT A-240 Date Drilled 10-3-4-83 Hole No. 16-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL/ SM/ ML	35.0-52.0 SANDY CLAY/SILTY SAND/SILT: cont.			RD	
46					PB	bent tube
48			J-5	12 27 39	SS	1.5/1.5 5:30 10/3/83
					RD	7:00 10/4/83
50						
					PB	2.1/2.5
52	SM	52.0-58.5 SILTY SAND: dark greenish grey; fine sand; some fines; dense; moist	PB-5			
54					RD	
56			PB-5		PB	2.5/2.5
58			J-6	12 20 25	SS	1.5/1.5
	CL	58.5-60.5 SANDY CLAY: dark greenish grey; little fine sand; hard; moist; contains cemented nodules			RD	
60	SM	60.5-64.5 SILTY SAND: dark greenish grey; fine sand, some fines; dense; moist				
62			C-6	40 56	DR	1.0/1.0
					RD	
64						
	SP	SAN PEDRO FORMATION 64.5-86.5 SAND: dark greenish grey; fine sand, trace of silt; very dense; wet				
66			PB-6		PB	1.6/2.5
68						

Project DESIGN UNIT A-240 Date Drilled 10-3-4-83 Hole No. 16-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	64.5-86.5 SAND: cont.	J-7	29 50-4.5"	SS RD	0.7/0.9
70						
72			C-7	48 50-4.5"	DR RD	0.7/0.9
74						
76			PB-7		PB	1.9/2.0
78			J-8	50-4"	SS RD	0.3/0.3
80	GP	gravelly sand grading to sandy gravel with some fine sand lenses				
82			C-8	106	DR RD	0.3/0.5
84						
86						
88		PUENTE FORMATION 86.5-99.7 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; thinly to thickly interbedded; mostly fines, trace fine sand partings Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh	PB-8		PB RD	2.0/2.5
90						
92						

Project DESIGN UNIT A-240 Date Drilled 10-3-4-03 Hole No. 16-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		86.5-99.7 INTERBEDDED SILTSTONE and CLAYSTONE: cont.	C-9	41 50-3"	DR	0.7/0.7
					RD	
97			PB-9		PB	2.5/2.5
100		B.H. 99.7 Terminated hole; flushed hole and installed piezometer to bottom, ~ 80-100' slotted; pea gravel backfill to surface				Completed drilling 12:15

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

Sheet 5 of 5

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BORING LOG 16-6

Proj: DESIGN UNIT A-240 **Date Drilled** 10-23-24-83 **Ground Elev.** 204'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 95.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.8 CONCRETE			GB	started drilling 9:30
	GP	0.8-1.5 BASE ROCK				
2	CL	OLD ALLUVIUM 1.5-3.5 SANDY CLAY: dark greenish grey; trace of fine sand; very stiff; moist	C-1	9 16	DR	0.8/1.0 pocket pen 4.25 tsf
					AD	
4	CL	3.5-11.0 CLAY: dark greenish grey; hard; moist	J-1	7 13 20	SS	1.5/1.5
6					RD	set tub & cased to 6.5'
		contains white cemented nodules	C-2	8 9	DR	1.0/1.0 pocket pen 4.25 tsf
8					RD	
				6 11 15	SS	0.1/1.5
10					RD	
	CL	11.0-15.0 SILTY CLAY: dusky yellow; stiff; moist; mottled with ferrous staining				
12			C-3	11 8	DR	1.0/1.0 pocket pen 3.5 tsf
					RD	
14			J-2	3 5 15	SS	1.4/1.5
	SP	15.0-23.5 SAND: dusky yellow; trace of fines; very dense; moist; occasional lenses of medium sand			RD	
16						
			C-4	23 36	DR	1.0/1.0
18					RD	
		intense ferrous mottling	J-3	17 41	SS	1.5/1.5
20						

Project DESIGN UNIT A-240 Date Drilled 10-23-24-83 Hole No. 16-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	15.0-23.5 SILTY SAND: cont.		42	SS	
					RD	
22		thin clayey silt lens		15	DR	0.9/1.0
			C-5	36		
					RD	
24	CL	23.5-43.5 SILTY CLAY: dark greenish grey; hard; moist	J-4	8 18 30	SS	1.5/1.5
					RD	
26						
			C-6	21 50-5.5"	DR	1.0/1.0 pocket pen 2.75 tsf
28		clayey sand lens			RD	
			J-5	11 20 31	SS	1.5/1.5
30					RD	
32		becoming weakly cemented throughout with cemented nodules	C-7	34 50-5"	DR	0.8/0.9 pocket pen > 4.5 tsf
					RD	
34		no cementation, no nodules; contains some thin clayey sand lenses	J-6	16 34 50-5"	SS	1.5/1.5
36					RD	
		occasional cemented nodules, increased silt, sandy clay lens	C-8	36 43	DR	1.0/1.0 pocket pen > 4.5 tsf
38					RD	
		interbedded silty sand, silty clay and silt	J-7	9 24 28	SS	1.1/1.5
40					RD	
42		weakly cemented in places	C-9	24 59	DR	0.9/1.0
					RD	
44	CL	43.5-46.5 SANDY CLAY:				

Project DESIGN UNIT A-240 Date Drilled 10-23-24-83 Hole No. 16-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	CL	43.5-46.5 SANDY CLAY: cont. dark greenish grey; little fine sand: hard; moist; occasional clayey sand lenses	J-8	19 24 30	SS RD	1.5/1.5
46	SM/SP	46.5-49.6 SILTY SAND: dark greenish grey; little silt; very dense; moist; sulfur odor	C-10	58 50-3"	DR RD	0.9/1.0
48			J-9	21 29 38	SS RD	1.5/1.5
50	CL	49.6-54.0 SILTY CLAY: dark greenish grey; trace of sand; hard; moist; contains occasional small cemented nodules				
52			C-11	31 46	DR RD	0.9/1.0 pocket pen > 4.5 tsf
54	ML	54.0-56.0 SANDY SILT: dark greenish grey; increasing sand content with depth; hard; moist	J-10	5 13 24	SS RD	1.5/1.5
56	SM	56.0-57.4 SILTY SAND: dark greenish grey; increasing sand content with depth; very dense; wet	C-12	42 50-3	DR " RD	0.7/0.7
58	SP	SAN PEDRO FORMATION 57.4-67.0 SAND: dark greenish grey; fine sand, trace silt; very dense; wet		22 50	SS RD	0.1/1.0
60						
62			C-13	71 50-2	DR 5" RD	0.4/0.7 disturbed
64				31 53	SS RD	0.1/1.0
66						
68	SW	67.0-71.0 GRAVELLY SAND: dark greenish grey				

Sheet 3 of 5

Project DESIGN UNIT A-240 Date Drilled 10-23-24-83 Hole No. 16-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	67.0-71.0 GRAVELLY SAND: cont. mostly fine to coarse sand with little fine gravel; very dense; wet			RD	
			J-11	91-5.5"	SS	0.4/0.5
70						
	SP	71.0-76.0 SAND: dark greenish grey; fine sand, trace of silt; very dense; wet				
72			C-14	86	DR	0.3/0.5
					RD	disturbed
74		becoming fine grained	J-12	25 50-4.5"	SS	0.9/0.9
					RD	
		gravelly sand lens				rig chatter
76		PUENTE FORMATION 76.0-95.7 INTERBEDDED SILTSTONE and CLAYSTONE: dark greenish grey and olive grey; thinly to thickly bedded; uncemented	C-15	30 50-4.5"	DR	0.6/0.9
78					RD	
		Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh				
80						
82		some sandstone beds	C-16	46 50-4	DR "	4:30 10/23/83
					RD	7:00 10/24/83
84						
86		contains some irregular inclusions of variable color	C-17	39 50-2.5"	DR	0.6/0.7
88					RD	
90						
			C-18	53 50-4	DR " RD	0.7/0.8
92						

Project DESIGN UNIT A-240 Date Drilled 10-23-24-83 Hole No. 16-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		76.0-95.7 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94						
			C-19	80 50-2.5"	DR	
96		B.H. 95.7 Terminated hole; installed piezometer to bottom, ~ 75-95' slotted.				complete drilling and flushing 8:45
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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BORING LOG 17

Proj: DESIGN UNIT A-220 Date Drilled 1-17-20-81 Ground Elev. 196'

Drill Rig Failing Logged By Gallinatti Total Depth 200.9'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	ALLUVIUM 0.0-4.5 SILTY SAND: dusky yellow; mostly fine sand; with some fines; loose; dry			AD	begin drilling 1:00 1/17/81; auger to 3', begin rotary drilling
3					RD	
4.5	CL	4.5-15.0 SANDY CLAY: moderate blue-green; some fine sand; stiff; moist				
10			J-1	8 12 17	SS	1.3/1.5 recovery
11.5					RD	
15	SP	15.0-20.0 SAND: moderate yellowish brown; mostly fine sand with some medium to coarse sand; trace of gravel; medium dense to loose				
20						

Project DESIGN UNIT A-220 Date Drilled 1-17-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	CL	20.0-30.0 CLAY: moderate yellowish brown; stiff; slightly moist to moist	C-1		DR	1.0/1.0 recovery
			J-2	4 4 6	SS	1.5/1.5 recovery
22					RD	
24						
26						
28						
30	SM/SP	30.0-34.5 SILTY SAND - SAND: pale yellowish brown; fine sand; trace of fines; poorly graded; loose; moist	J-3	8 18 34	SS	1.5/1.5 recovery
					RD	contact at very top of sample
32						
34	CL	34.5-38.0 CLAY: moderate yellowish brown; stiff; slightly moist to moist				
36						
38	ML	38.0-49.5 CLAYEY SILT: moderate blue-green; stiff; slightly moist; horizontal thin laminations				
40			C-2		DR	1.0/1.0 recovery
			J-4	8 18 32	SS	1.5/1.5 recovery
42			Box #1		PB	42.5' begin continuous pitcher barreling; pocket pen>4.5
44						

Sheet 2 of 9

Project DESIGN UNIT A-220 Date Drilled 1-17-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	ML	38.0-49.5 <u>CLAYEY SILT</u>: cont.	Box #1		PB	1.9/2.5 recovery
						1.9/2.5 recovery
46						
48						2.5/2.5 recovery
		SAN PEDRO FORMATION				
50	SP	49.5-64.0 <u>SAND</u>: grayish blue-green; poorly graded fine sand; trace of fines; dense; moist to wet				0.0/2.5 recovery
						No Recovery
52						
						1.7/2.5 recovery
54						
		55.1' medium to coarse sand				
56						1.5/2.5 recovery
		56.9' medium to coarse sand	Box #2			
58						2.5/2.5 recovery
60						
						1/17/81 60' stop drill 1/18/81 begin drilling 60' Gas Test - no gas
62						1.1/2.8 recovery
			S-1			1.9/2.8 recovery
64		FERNANDO FORMATION 64.0-74.0 <u>CLAYSTONE</u>: dark yellowish brown; slightly moist to moist				
66		<u>Physical Conditions</u>: massive; low hardness; friable; fresh	Box #2 cont.			1.9/2.8 recovery
68						

Project DESIGN UNIT A-220 Date Drilled 1-17-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68		FERNANDO FORMATION 64.0-74.0 CLAYSTONE: cont.	Box #2		PB	2.1/2.8 recovery
70						2.2/2.8 recovery
72			Box #3			
74	40°	74.0-79.1 CLAYSTONE with interbedded SANDSTONE: thinly bedded; claystone is dark yellowish brown; sandstone is light blueish grey				1.9/2.8 recovery
76	40°	Physical Conditions: massive; low hardness; friable; fresh				1.9/2.4 recovery
78						
80		79.1-80.5 CLAYSTONE: olive gray; cemented claystone Physical Conditions: massive; hard; moderately strong; fresh			RD	@ 79.2' no recovery, bent tube, cemented zone is too hard to cut; rotary drill through it
		80.5-107.2 CLAYSTONE: mod. brown; slightly moist	Box #3 cont.		PB	2.2/2.8 recovery
82		Physical Conditions: massive; low hardness; friable; fresh				2.3/2.8 recovery
84			S-2			
86						pocket penetrometer > 4.5 tsf 2.0/2.9 recovery
88			Box #3 cont.			
90			Box #4			1.7/2.8 recovery
92						

Project DESIGN UNIT A-220 Date Drilled 1-17-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		80.5-107.2 CLAYSTONE: cont.	Box #4		PB	2.0/2.8 recovery
		93.0' thin beds of sandy claystone; some fine sandstone				
94	30°					
						2.3/2.8 recovery
96	30°					pocket penetrometer > 4.5 tsf
						2.8/2.8 recovery
98	35°					
100			S-3			2.8/2.8 recovery
102						
			Box4 cont.			1.7/2.8 recovery
104			Box #5			
106						2.3/2.8 recovery
		107.2-111.0 CLAYSTONE with interbedded SANDY CLAYSTONE: thinly bedded claystone with occasional beds of sandy claystone(little fine sandstone); color varies: mod. brn, dk yellowish brn & lt. brn Physical Conditions: massive; low hardness; friable; fresh				
108	25°					2.7/2.8 recovery
110						pocket penetrometer > 4.5 tsf
		111.0-120.0 CLAYSTONE: dark, mod. brown; slightly moist				
112						2.8/2.8 recovery
114						2.8/2.8 recovery
116			Box #6			

Sheet 5 of 9

Project DESIGN UNIT A-220 Date Drilled 1-18-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		111.0-120.0 CLAYSTONE: cont.	Box #6		PB	
		Physical Conditions: massive; low hardness; friable; fresh				2.8/2.8 recovery
118		grading coarser				pocket penetrometer > 4.5
120	30°					2.5/2.8 recovery
		120.0-124.0 SILTSTONE: mod. yellowish brown; thinly bedded; slightly moist				
122		Physical Conditions: massive; low hardness; friable; fresh				
			S-4			2.8/2.8 recovery
124		124.0-131.0 CLAYSTONE: dark mod. brown; slightly moist				2.8/2.8 recovery
126		Physical Conditions: massive; low to moderate hardness; friable to weak; fresh	Box #6 cont.			
		127.8-129.3 layer of hard claystone				
128			Box #7			2.8/2.8 recovery
		gradational contact				
130						
		131.0-138.0 SILTSTONE: greyish olive; slightly moist				2.8/2.8 recovery
132						
		Physical Conditions: massive; low hardness; friable; fresh				
134						2.8/2.8 recovery
136						
			S-5			2.8/2.8 recovery
138		138.0-158.1 CLAYSTONE: dark mod. brown; moist				
		Physical Conditions: massive;	Box 7			
140		low hardness; friable; fresh	Box 8			

Project DESIGN UNIT A-220 Date Drilled 1-18-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		138.0-158.1 CLAYSTONE: cont.	Box #8		PB	2.8/2.8 recovery
142						2.8/2.8 recovery
144						pocket penetrometer > 4.5 tsf 2.8/2.8 recovery
146						
148						2.8/2.8 recovery
150			Box #9			2.8/2.8 recovery
152						pocket penetrometer > 4.5 tsf
154			S-6			2.8/2.8 recovery
156			Box #9 cont.			2.6/2.8 recovery pocket penetrometer > 4.5 tsf
158		158.1-164.8 CLAYSTONE and SILTSTONE: thinly interbedded layers of claystone and siltstone; claystone is dk. mod. brown; siltstone is grayish olive; slightly moist Physical Conditions: massive; low hardness; friable; fresh				158.9' stop drilling 1/18/81; 1/19/81 10:00 begin drilling
160	30°					2.8/2.8 recovery
162			Box #10			2.8/2.8 recovery
164						

Project DESIGN UNIT A-220 Date Drilled 1-18-81 Hole No. 17

<table>
<tr><th>DEPTH</th><th>USCS</th><th>MATERIAL CLASSIFICATION</th><th>SAMPLE</th><th>SPT (6")</th><th>DRILL MODE</th><th>REMARKS</th></tr>
<tr><td>164</td><td></td><td>158.1-164.8 <u>CLAYSTONE and SILTSTONE</u>: cont.

164.8-200.9 <u>CLAYSTONE</u>: dark mod. brown; slightly moist

<u>Physical Conditions</u>: massive; low hardness; friable; fresh</td><td>Box #10</td><td></td><td>PB</td><td>2.8/2.8 recovery

pocket penetrometer > 4.5 tsf
2.8/2.8 recovery</td></tr>
<tr><td>166</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>168</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>170</td><td></td><td></td><td>S-7</td><td></td><td></td><td>2.8/2.8 recovery</td></tr>
<tr><td>172</td><td></td><td></td><td>Box #10 cont</td><td></td><td></td><td>2.8/2.8 recovery</td></tr>
<tr><td>174</td><td></td><td></td><td>Box #11</td><td></td><td></td><td></td></tr>
<tr><td>176</td><td></td><td></td><td></td><td></td><td></td><td>2.8/2.8 recovery</td></tr>
<tr><td>178</td><td></td><td></td><td></td><td></td><td></td><td>2.8/2.8 recovery</td></tr>
<tr><td>180</td><td></td><td></td><td></td><td></td><td></td><td>pocket penetrometer > 4.5 tsf
2.8/2.8 recovery</td></tr>
<tr><td>182</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>184</td><td>30°</td><td>184.9-185.1' sandstone layer; light blueish gray; fine sandstone</td><td>Box #12</td><td></td><td></td><td>2.8/2.8 recovery</td></tr>
<tr><td>186</td><td></td><td></td><td>S-8</td><td></td><td></td><td></td></tr>
<tr><td>188</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

Project DESIGN UNIT A-220 Date Drilled 1-19-81 Hole No. 17

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
188		164.8-200.9 CLAYSTONE: cont. Physical Conditions: massive; low hardness; friable; fresh	S-8		PB	2.8/2.8 recovery
190			Box #12 cont.			2.8/2.8 recovery; pocket penetrometer > 4.5 tsf
192						2.8/2.8 recovery
194						
196	30°	196-198' some thin layers of sandy claystone	Box #13			2.8/2.8 recovery
198			S-9			2.8/2.8 recovery
200						
202		B.H. 200.9' Terminated hole				stop circulation 2:30 1/19/81. Run electric logs. 1/20/81 install piezometer (2" pvc) down to 200'; with cloth covered perforations from 50-70' and from 180-195'. Gravel pack to surface, surface Bentonite plug. Move off site 11:00 1/20/81. water sampled 2/18/81
204						
206						
208						
210						
212						

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

Converse Consultants, Inc.
Earth Sciences Associates
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BORING LOG 17A

Proj: DESIGN UNIT A-220 Date Drilled 10-27-83 Ground Elev. 200'

Drill Rig MAN-SIZED AUGER Logged By J. Stellar Total Depth 42'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		A/C PAVMENT 0.0-0.6				Hole stands well 0'-42', very minor belling at 13', 18' & 26'
	ML	ALLUVIUM 0.6-4.0 CLAYEY SILT: black to dark gray, moist, firm, with organics, trace of sand and gravel to ¼"				
4	ML	4.0-10.0 SANDY SILT: gray, moist, stiff				
10	SM	10.0-13.0 SILTY SAND: light green, very moist, medium dense, numerous calcareous streaks				
13	SP	13.0-19.0 SAND: light brown, wet, medium				seepage at 1± gpm
18						water level @ 18' after 1½ hours
19	ML	19.0-26.0 SANDY SILT:				

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Sheet 1 of 2

Project DESIGN UNIT A-220 Date Drilled 10-27-83 Hole No. 17A

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	ML	19.0-26.0	SANDY SILT: (continued) light green, stiff, moist with lenses of sand				
26	SP	26.0-30.0	SAND: light green, wet, dense with layers of silty sand				water seepage at 2-3 gpm
30	ML	30.0-33.0	SANDY SILT: light green, very moist, stiff to hard, with layers of sand				
33	ML	33.0-38.0	SILT: bluegray, very moist, stiff to hard, with layers of silty clay and clayey silt				
38	SP	38.0-42.0	SAND: light green, slightly moist, dense, strong sulfur odor				100% LEL Gas reading. gas vapors visable at surface. 20% LEL after caving sand sealed off most of the gas. Hole caved back to 35.5 after 1 hour
42			B.H. 42'. Terminated hole due to high amounts of combustibles. Gas churning water @ 38'. Took water sample @ 25' after 1 hr. Took gas sample @ 20' after 1 hr.				Sheet 2 of 2

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

Converse Consultants, Inc.
Earth Sciences Associates
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BORING LOG 17B

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

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

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

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

Sheet 1 of 3

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

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	ML	16.0- 34.0	SILT: (continued)				
22							water level at 23' after 1 hour
24							
26							
28			moderate H20 seepage gravelly layer, gravel to 1"				seepage 1± gpm
30							
32							water level at 32' during drilling operation
34	ML/ GM	34.0- 37.0	GRAVELLY SILT: orange brown, wet, stiff, gravel to 1"				slow drilling @ 34'
36							
	ML	37.0- 39.0	SILT: blue, wet, dense, layers of sandy silt				
38							
	SM	41.0- 56.0	SILTY SAND: orange brown, wet dense, with layers of sandy silt and sand				
40							
	ML		SILT: blue, wet, stiff, with layers of sandy silt and sand				
42							
44							

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	ML	41.0-56.0 SILT: (continued)				
46						bag sample at 46'
48						
50						
52		52'-54': sand layer, wet				
54						
56	SP	SAN PEDRO FORMATION 56.0-64.0 SAND: blue, wet, dense, medium grained, slight sulfur odor				sand continously caving,only small amounts of material remain in bucket bag sample @ 58' hole caved back to 48' after 2 hours
58						
60						
62						
64		B.H. 64.0' Hole terminated due to running ground below 56'. No gas detected by meter. Downhole Observers: J. Stellar				
66						
68						

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BORING LOG 18-2

Proj: DESIGN UNIT A-245 **Date Drilled** 10-4-5-83 **Ground Elev.** 195.5'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 94.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CONC	0.0-0.7 CONCRETE			GB	start drilling 3:30
	CL	FILL 0.7-2.8 SANDY CLAY: olive black; mostly clay with a trace of fine sand; very stiff; moist				
2				6	DR	1.0/1.0
			C-1	8		
	CL	OLD ALLUVIUM 2.8-5.0 SILTY CLAY: moderate brown; stiff; moist			AD	
4						
	ML	5.0-8.0 SILT: dusky yellow; hard; moist		14	SS	0.8/1.5
6			J-1	26		
				21		
					AD	
8	SM/ML	8.0-10.0 SILTY SAND: dusky yellow; mostly fine sand with some fines; dense; moist		17	DR	1.0/1.0
			C-2	27		
					AD	
10	ML	10.0-12.5 CLAYEY SILT: dusky yellow; hard; moist		12	SS	1.5/1.5 set tub & case to 13'
			J-2	19		
				32		4:30 10/4/83
					RD	7:00 10/3/83
12						
	ML	12.5-14.5 SILT: dusky yellow; hard; moist; with cemented nodules		7	DR	1.0/1.0
			C-3	22		
14					RD	
	SP	14.5-16.0 SAND: brown; mostly fine sand, trace of silt; dense; moist		1	SS	1.5/1.5
			J-3	23		
16				25		
	CL	16.0-17.5 SANDY CLAY: dusky yellow; mostly fines with a trace of sand and gravel; hard; moist			RD	
	CL	17.5-22.5 CLAY: dusky yellow; hard; moist; with cemented zones & nodules				
18				12	DR	1.0/1.0
			C-4	22		pocket pen> 4.5
					RD	
20						

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

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

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

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

Sheet 3 of 5

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

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

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

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

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BORING LOG 18-3

Proj: DESIGN UNIT A-245 **Date Drilled** 10-5-6-83 **Ground Elev.** 195.5'
Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 160.8'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

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

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

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

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

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

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	61.0-86.0 SAND: cont.			RD	
		occasional gravel	J-12	46 50-5	SS "	0.5/0.9
70					RD	
72			C-15	106	DR	0.3/0.5
					RD	partial
74		several very thin clayey lenses	J-13	41 50-	SS 5"	0.6/0.9
					RD	
76						
			C-16	51 60-4	DR 5"	0.7/0.9
78					RD	
			J-14	33 53	SS	0.5/1.0
80					RD	
82		with little silt	C-17	59 60-	DR 3"	0.7/0.7
					RD	
84			J-15	44 62	SS	0.5/1.0
					RD	rig chatter
		basal gravel				
86		FERNANDO FORMATION 86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: olive grey and dark greenish grey; with fine sand partings; contains mica thinly bedded to massive bedding; sulfur odor				
			C-18	52 52-	DR 3"	0.7/0.7
88					RD	
		Physical Condition: little fractured to massive; friable hardness and strength; little weathered to fresh				
90						
			C-19	50 50-	DR 3.5"	0.5-0.8
92						

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94						
		contains irregular angular inclusions of different color siltstone	C-20	63	DR	0.7/0.7
				63-3	"	
96					RD	
98						
100						
102						
104						
106						
108						
110						
112						
114						
			C-21	48	DR	0.7/0.7
116				65-	3" RD	

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		86.0-160.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
118						
120						
122						
124						
126						
128						
130						
132						
134						
				56	DR	0.7/0.7
			C-22	50-3	"	
136					RD	
138						
140						

Sheet 6 of 7

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

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

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 18-4

Proj: DESIGN UNIT A-245 **Date Drilled** 10-10-83 **Ground Elev.** 196.5'
Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 94.8'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

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

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

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

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

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

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

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

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		86.0-94.8 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94				41	DR	0.8/0.8
			C-17	50-3.5"		
		B.H. 94.8 Terminated hole; tremied grout to surface				Drilling complete 5:15
96						
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 18-5

Proj: DESIGN UNIT A-245 **Date Drilled** 10-7-8-83 **Ground Elev.** 197'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 95.7

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

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

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

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

Sheet 2 of 5

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

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

Sheet 3 of 5

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

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

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		87.0-95.7 INTERBEDDED SILTSTONE and CLAYSTONE: cont.			RD	
94						
			C-20	52	DR	0.7/0.7
				75-3	"	
96		B.H. 95.7 Terminated hole, tremied grout to surface				Complete drilling 7:45
98						
100						
102						
104						
106						
108						
110						
112						
114						
116						

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BORING LOG 18-6

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

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

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

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

Sheet 1 of 4

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

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

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

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

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

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

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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Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 18-7

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

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

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

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

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

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

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

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

Appendix B

Geophysical Exploration

APPENDIX B GEOPHYSICAL EXPLORATION

B.1 DOWNHOLE SURVEY

B.1.1 Summary

Downhole shear wave velocity surveys were performed in Borings CEG-14 and CEG-15 for Design Unit A220. Measurements were made at 5-foot intervals from the ground surface to depths of 130 feet. A description of the technique and a summary of the results are attached.

B.1.2 Field Procedure

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

B.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure B-1. The timing line shows a 20 millisecond (MS) break at the end of the record, indicating that each vertical line is 10 MS. The time of the first arrivals of compressional shear energy is indicated by P and S, respectively. Wave arrival records similar to Figure B-1 were analyzed to estimate wave travel times and velocities for CEG-14 and CEG-15.

B.1.4 Discussion of Results

Estimated velocity structures are summarized in Table B-1. Velocity estimates are based on selection of linear portions of the downhole arrival time curves (see Figures B-2 and B-3).

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^*). V_p^* are the values to be used for studies of the response of these sites. N is the number of data points used for the straight line fit for each velocity estimate.

In general, the near-surface shear wave velocity was found to be approximately 1200 feet per second. To depths of about 200 feet, shear wave velocity estimates generally increased to 1700 to 2000± feet per second. One exception to this trend occurred at Boring CEG-14 where the shear wave velocity decreased from 2710± feet per second between depths of 55 and 75 feet to 860 feet per second between depths of 75 and 95 feet. Another exception occurred at Boring CEG-15 where the shear wave velocity decreased from 1180± feet per second between depths of 75 to 145 feet to 960± feet per second between depths of 145 and 175 feet.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

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

B.2.2 Field Procedure

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

B.2.3 Data Analysis

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

B.2.4 Discussion of Results

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

TABLE B-1
DOWNHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
14	15- 55						1194	61	60	9	1190±120
	55- 75						2711	348	136	5	2710±480
	75- 95	6492	562	325	38	6490±890	856	32	43	5	860±70
	95-125						1429	394	71	7	1430±470
	125-198						1676	100	84	16	1680±180
15	10- 75	3935	544	197	14	3940±740	1277	48	64	14	1280±110
	75-145						1180	100	59	15	1180±160
	145-175	5267	629	263	23	5270±890	963	49	48	7	960±100
	175-200						2054	616	100	5	2054±720

$\bar{V}p$ = mean estimate of compressional wave velocity.
$\bar{V}s$ = mean estimate of shear wave velocity.
σp = standard deviation of estimated compressional wave velocity.
σs = standard deviation of estimated shear wave velocity.
Ep = estimated accuracy of compressional survey.
Es = estimated accuracy of shear survey.
Np = number of points used for straight line fit of comrpessional wave.
Vp* = overall accuracy of compressional wave velocity estimate.
Vs* = overall accuracy of shear wave velocity estimate.
Ns = number of points used for straight line fit of shear wave velocity data.

TABLE B-2

CROSSHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		V̄p	σp	Ep	Np	Vp*	V̄s	σs	Es	Ns	Vs*
15	10	2043	36	137	2	2040±170	684	2	46	6	680±50
	15	3240	590	227	2	3240±820	855	19	43	8	860±50
	20	2752	830	138	2	2750±970	970	36	49	12	970±85
	25	3020	300	150	2	3020±450	985	65	50	12	985±115
	30	4150	105	200	4	4150±300	847	9	42	8	850±50
	35	4380	574	219	3	4380±790	858	7	43	8	860±50
	40	4621	41	231	5	4620±270	941	2	47	4	940±50
	45	6066	8	303	2	6060±310	1049	6	51	4	1050±60
	50	4410		440	1	4440±440	1155	30	58	8	1155±90
	55	4460	67	220	5	4460±290	1093	10	52	12	1090±60
	60	4390	6	220	2	4390±320	911	16	46	11	910±60
	65	4120	114	206	3	4120±320	921	21	46	12	920±70
	70	3740	640	187	7	3740±830	919	11	46	15	920±60
	75	3940		400	1	3940±400	952	15	48	15	950±60
	80	4260		430	1	4260±430	972	6	48	12	970±70
	85	3950		400	1	3950±500	975	17	48	12	975±70
	90	4505	336	225	2	4500±560	881	1	44	5	880±50
	95	4475	225	224	3	4480±450	973	22	48	6	970±70
	97	4085	794	200	3	4085±990	1243	24	62	12	1240±90

V̄p = mean estimate of compressional wave velocity.

V̄s = mean estimate of shear wave velocity.

σp = standard deviation of estimated compressional wave velocity.

σs = standard deviation of estimated shear wave velocity.

Ep = estimated accuracy of compressional survey.

Es = estimated accuracy of shear survey.

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

Vp* = overall accuracy of compressional wave velocity estimate.

Vs* = overall accuracy of shear wave velocity estimate.

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

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DOWNHOLE SAMPLE RECORD

TRACE IDENTIFICATION

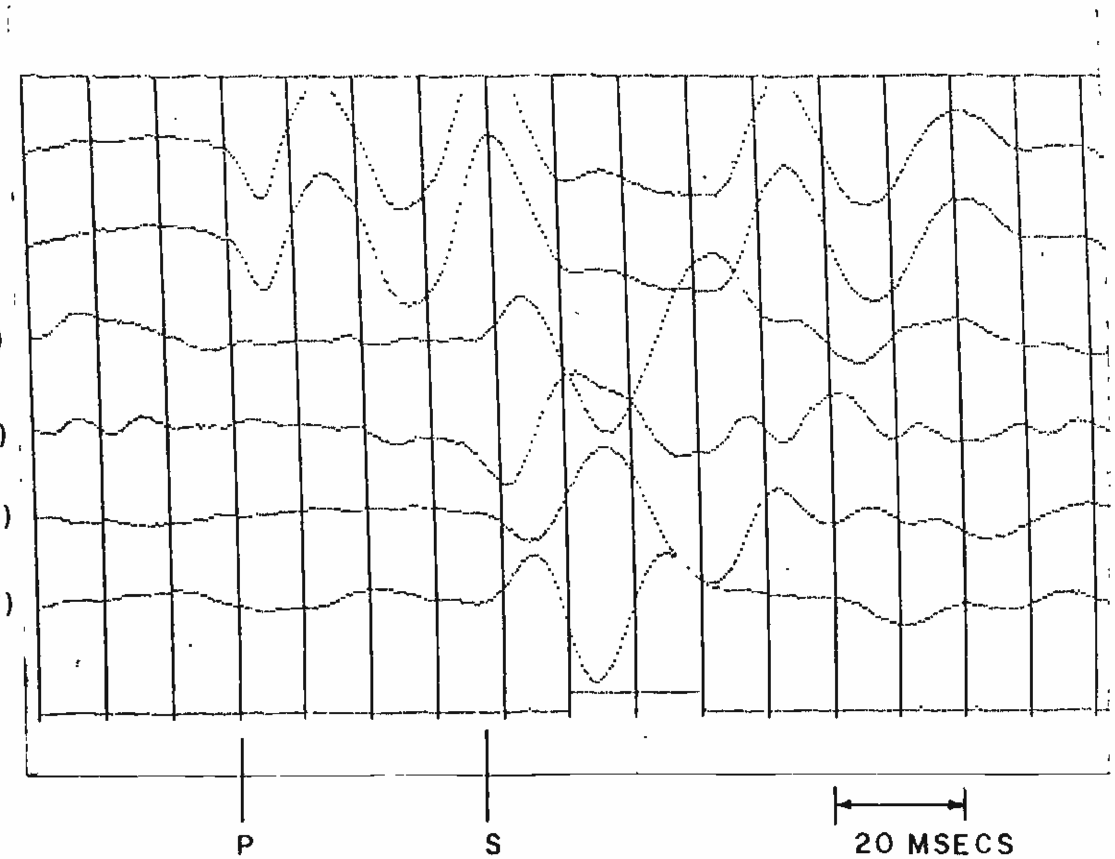
VERTICAL (DOWN)

HORIZONTAL 1 (WEST)

HORIZONTAL 1 (EAST)

HORIZONTAL 2 (WEST)

HORIZONTAL 2 (EAST)



BOREHOLE: 13

DEPTH: 70 FT

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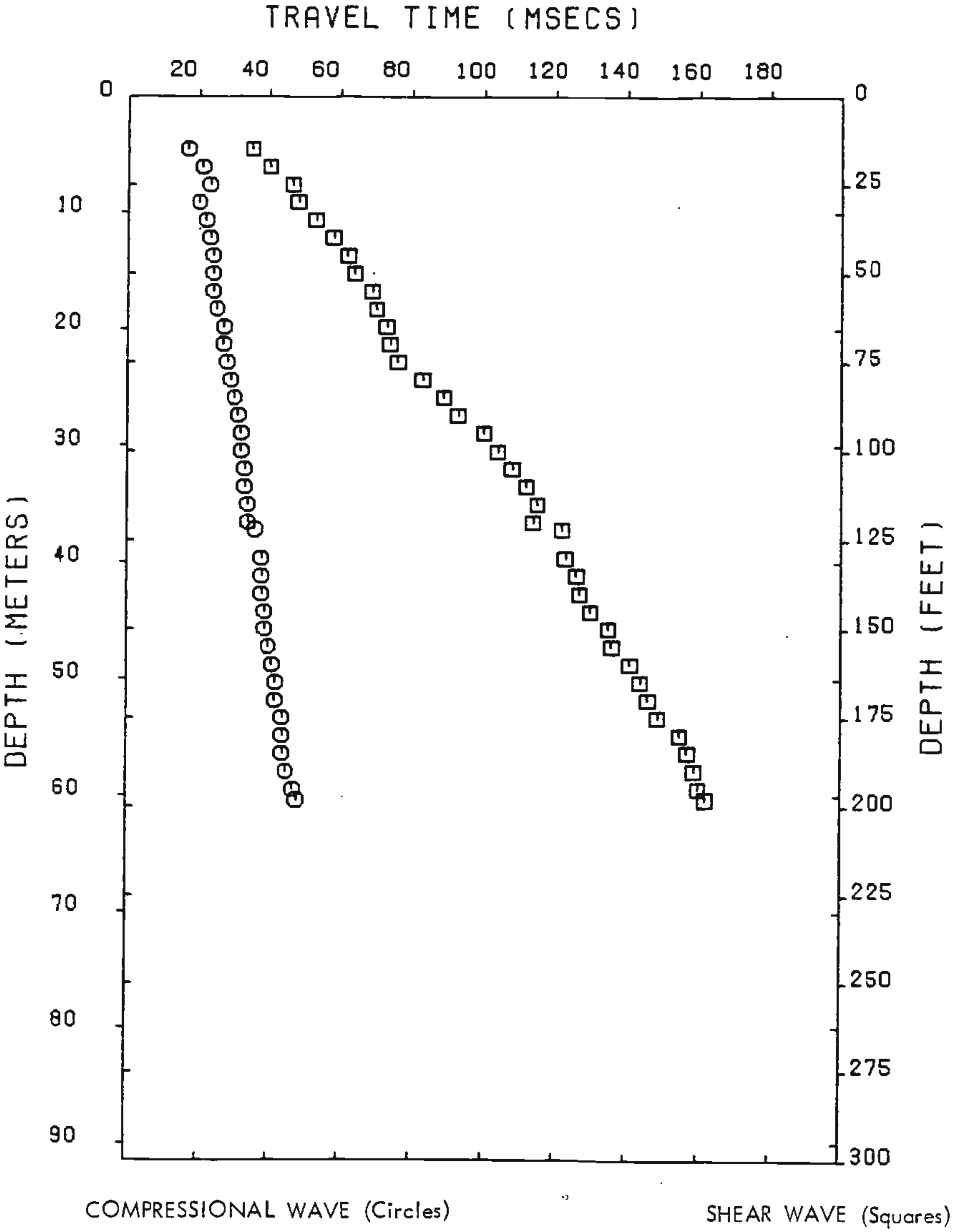
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Figure No. B-1



DOWNHOLE TRAVEL TIME PROFILE - BORING 14

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Figure No B-2

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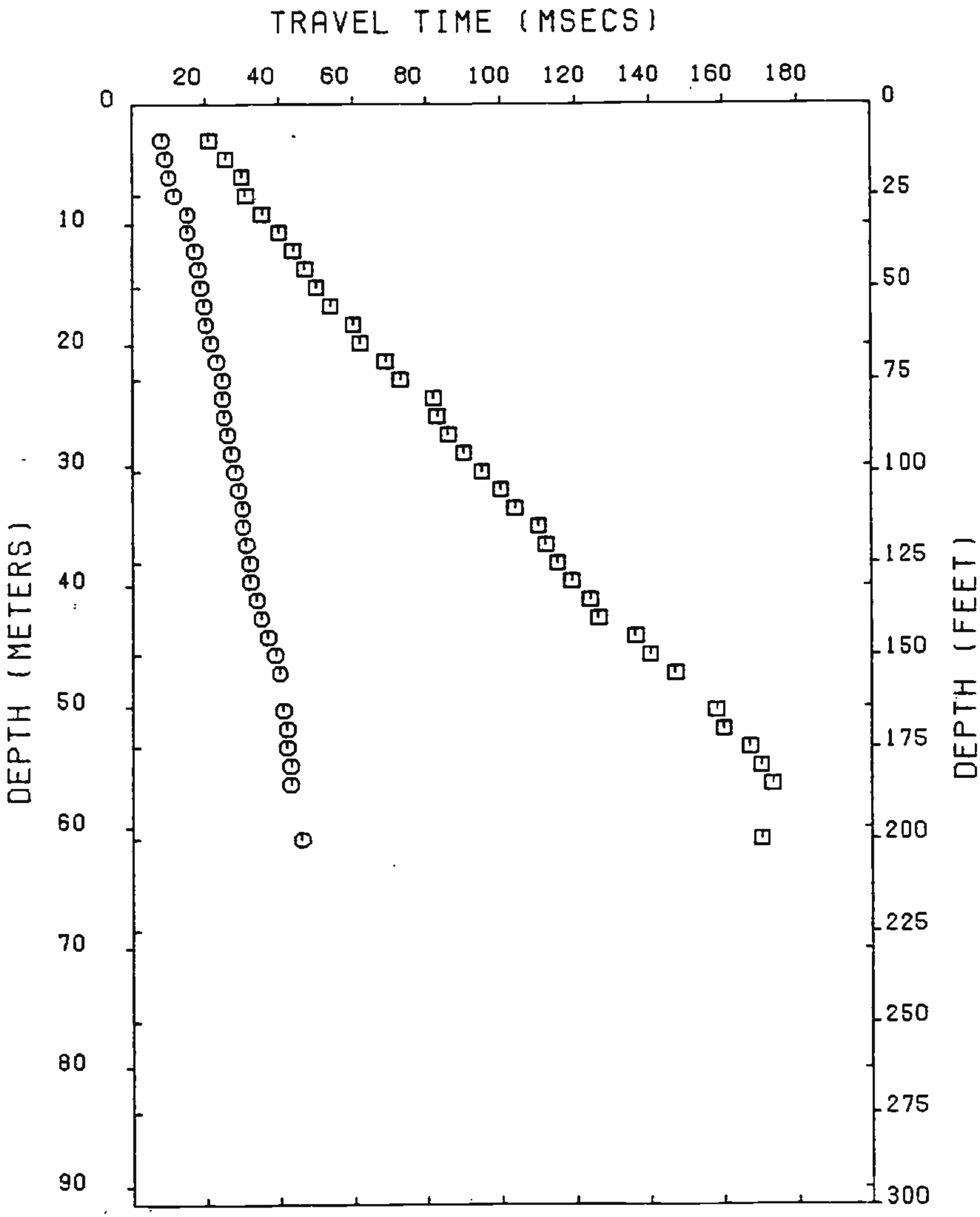

COMPRESSIONAL WAVE (Circles) SHEAR WAVE (Squares)

DOWNHOLE TRAVEL TIME PROFILE - BORING 15

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Figure No B-3

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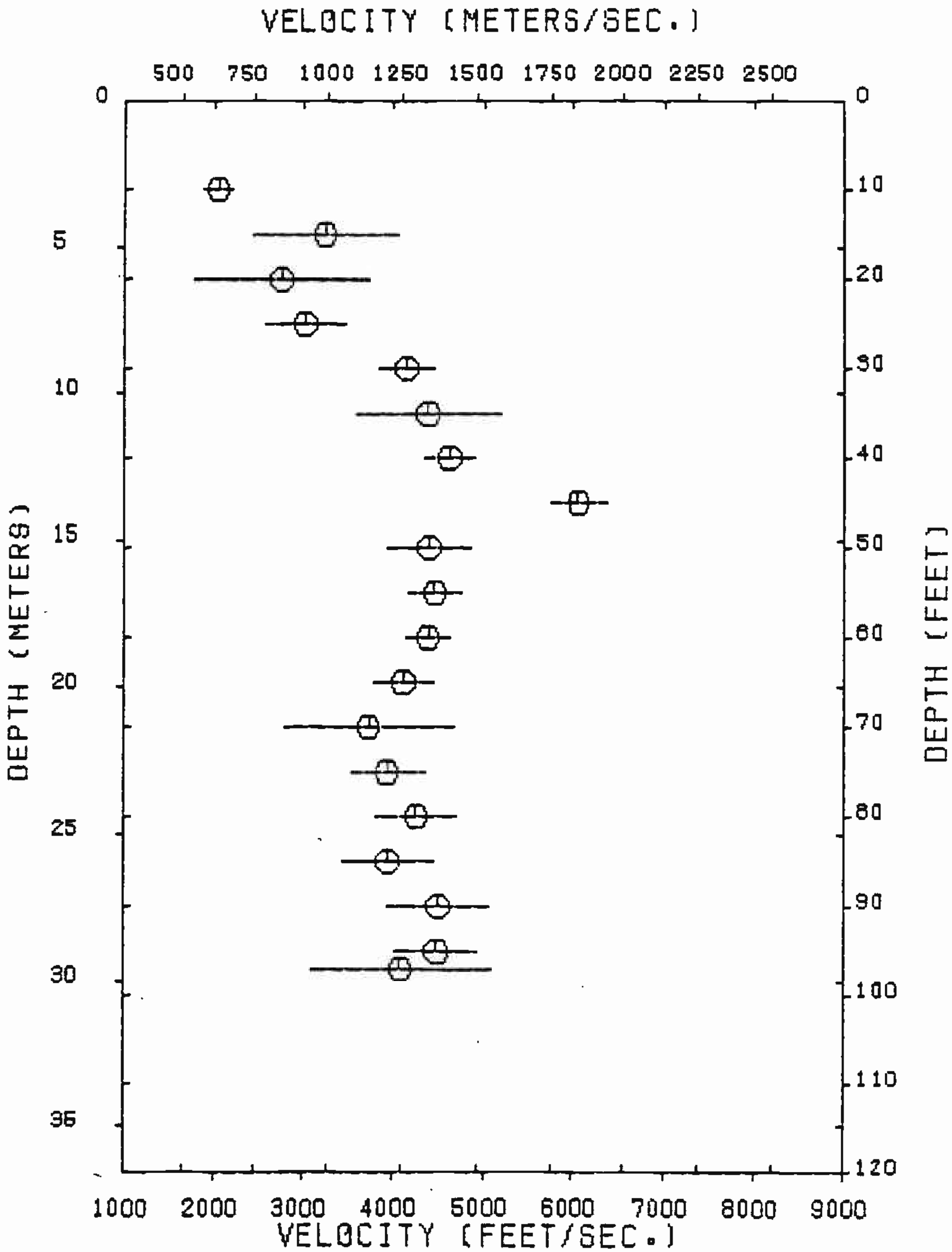

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 15

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Figure No.
B-4

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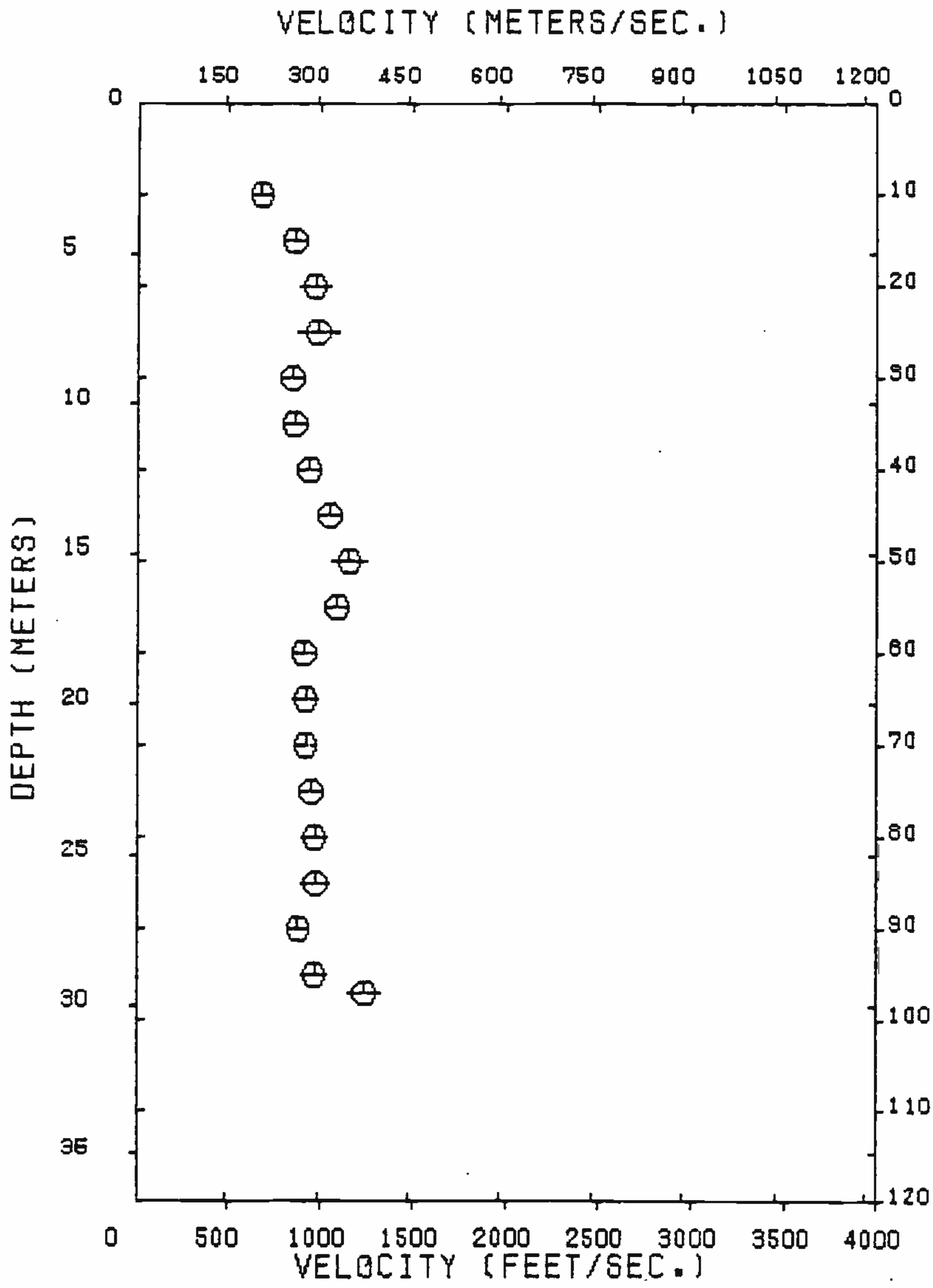

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 15

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Figure No.
B-5

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Appendix C

Geotechnical Laboratory Testing

APPENDIX C GEOTECHNICAL LABORATORY TESTING

C.1 INTRODUCTION

This appendix presents laboratory geotechnical tests performed on selected soil and bedrock samples obtained from the borings drilled at the Wilshire/ Normandie and Wilshire/Western Station sites. Laboratory testing of the remaining borings is presented in the geotechnical reports for Design Units A240 and A245

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-13 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-14 through C-18.

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-19 and Tables C-2 and C-3.

C.2.4 Moisture Content

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

C.2.5 Unit Weight

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

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

C.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-20 through C-26.

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

A free swell test was performed on a selected undisturbed sample of cohesive, potentially expansive soil. The test procedure entailed placing the undisturbed soil sample in a consolidometer, applying a vertical confining load, and inundating the sample with tap water. The resulting one-dimensional swell of the sample was measured and recorded. Results of the test are presented on Table C-1.

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 Figures C-27 through C-37.

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.3.7 Porosity

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

$$e = \frac{1-Vs}{Vs} \text{ where } Vs = \frac{\gamma_d}{G \times \gamma_w} \text{ and } n = \frac{e}{1+e}$$

Where:

γ_w = unit weight of water

γ_d = unit dry weight of soil

G = specific gravity of soil solids.

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

C.4 ENGINEERING PROPERTIES: DYNAMIC

C.4.1 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.4.1.1 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.4.1.2 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.4.1.3 Apparatus: The apparatus described below 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.

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.4.1.4 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.

The Dynamic Triaxial test results are shown on Figures C-38 through C-45.

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
13-7	1	4	Silty Clay	107	19												
	2	10	Sandy Clay	101	21												
	3	15	Sand	109	16												
	4	21	Siltstone	87	32												
	6	30	Claystone	78	43												
	8	40	Siltstone	80	43												
	10	50	Sandy Siltstone	89	30												
	12	60	Siltstone	82	40												
	14	70	Siltstone	78	40												
13-8	1	4	Sand	93	12												
	2	10	Sand	96	16												
	3	14	Silty Clay	86	36												
	5	24	Claystone	88	36												
	7	35	Siltstone	84	36												
	9	45	Siltstone	77	47												
	11	55	Siltstone	81	41												
	13	65	Siltstone	88	36												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-1	1	2	Silty Clay	99	24												
	2	7	Clayey Silt	98	24				2.6								
	3	12	Silty Sand	105	19												
	4	21	Silty Clayey Sand	105	11												
	5	22	Clayey Sand	125	13				4.9								
	6	24	Clayey Sand	121	11												
	7	27	Clayey Sand	118	13												
	8	32	Sandy Clay	120	12				3.6								
	9	37	Sand, Silt and clay	114	11					28	1.50						
	10	42	Silty Sand	112	9												
	11	46	Clayey Sand	118	15									X			X(1)
	12	52	Sandy Silty Clay	96	31	51	27							X			X(1)
	13	57	Silty Sand	113	17												
	14	62	Sandy Silty Clay	113	17									X			X(1)
	15	67	Sandy Clay	96	30											X	
	16	72	Sandy Clayey Silt	109	20											X	
	17	77	Sand	111	17												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-1	18	84	Silty Sand	107	19												
	19	94	Silty Sand	99	23												
	20	104	Silty Sand	99	25												
	21	105	Siltstone	83	35												
14-2	1	2	Silty Sand	106	12												
	2	7	Clayey Sand	118	14				6.1								
	3	12	Sandy Clay	111	19												
	4	17	Sandy Clay	102	24	54	37		7.6								
	5	22	Sandy Clay	117	16				10.3								
	6	27	Silty Sand	113	11												
	7	32	Silty Sand	112	10												
	8	37	Silty Sand	112	13					37	0.80						
	9	42	Gravelly Sand	115	12			2.1×10^{-3}						X			
	10	47	Silty Sand	110	18												
	11	52	Silty Sand	109	18												
	12	57	Sandy Clay	110	19				6.4								
	13	62	Sandy Clay	106	22				10.9								

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-2	14	69	Silty Sand	108	18												
	15	74	Silty Sand	103	21					30	0.87					X	
	16	79	Silty Sand	109	20			3.0×10^{-4}(10)						X			X(3)
	17	84	Silty Sand	96	24											X	
	18	89	Silty Sand	98	25												
	19	94	Silty Sand	99	25												
	20	99	Silty Clay	84	35												
	21	104	Silty Clay	75	41												
14-3	1	2	Clayey Sand	110	12												
	2	7	Silty Clay	101	23				1.7								
	3	12	Sandy Clay	110	18	36	20										
	4	17	Clayey Silt	95	25				4.4								
	5	22	Sandy Clayey Silt	99	21					34	0						
	6	27	Sandy Silt	100	23												
	7	33	Sandy Silt	106	20												
	8	38	Clayey Sand	105	20					26	0.65						
	9	43	Sand	14	19			4.7×10^{-3}		36	0.50			X			

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-3	10	48	Sandy Silt	103	22				3.9								
	11	52	Silty Sand	105	19					28	1.14						
	12	58	Silty Sand	99	24			6.5×10^{-4}		27	0.50			X			
	13	62	Sand	96	25			3.6×10^{-4}		30	0.65						
	14	69	Clayey Silt	71	47											X	
	15	74	Clayey Silt	83	34				8.4								
	16	79	Clayey Silt	81	37				3.3								
	17	89	Clayey Silt	83	37				11.4								
	18	94	Clayey Siltstone	84	34												
	19	120	Clayey Siltstone	76	47												
	20	140	Clayey Siltstone	86	35												
	21	160	Clayey Siltstone	91	32												
14-4	1	2	Clayey Sand	108	17												
	2	7	Silty Clay	101	26	54	38		6.9								
	3	12	Clay	94	28				6.3								
	4	17	Clay	91	32												
	5	22	Clayey Sand	112	17				4.7								

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-4	6	28	Silty Clay	91	30					34	2.02						
	7	32	Sandy Clay	111	18			5.7×10^{-6}		22	2.10						
	8	37	Silty Sand	90	25			2.6×10^{-3}		30	0.60			X			
	9	42	Sandy Silty Clay	80	46				1.4								
	10	49	Siltstone	91	31												
	11	54	Siltstone	87	37							0.05(1)					
	12	59	Claystone	90	31											X	
	13	69	Clayey Siltstone	84	34				7.6								
	14	74	Clayey Siltstone	91	30											X	
	15	79	Siltstone	92	30												
	16	89	Clayey Siltstone	85	35				12.6								
	17	94	Clayey Siltstone	d i s t u r b e d													
	18	99	Siltstone	96	28												
14-5	1	3	Silty Clay	95	30												
	2	8	Silty Clay	97	26				5.7								
	3	13	Sandy Clay	110	18												
	4	18	Sandy Clay	106	20					18	1.26						

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
14-5	5	22	Sandy Clay	111	11					30	0.58						
	6	28	Sand	114	10					46	0.41						
	7	38	Silty Clay	88	32				6.2								
	8	45	Clayey Silt	86	33									X	X		X(1)
	9	50	Clayey Siltstone	83	37							0.21(1)					
	10	54	Siltstone	92	31											X	
	11	65	Clayey Siltstone	90	31	45	10							X	X		X(1)
	12	70	Clayey Siltstone	87	31				4.8								
	13	84	Clayey Siltstone	96	28									X	X		X(1)
	14	90	Clayey Siltstone	91	31												
15-1	1	2	Silty Clay	110	20												
	2	7	Silty Sand	112	16					30	0.90						
	3	12	Sand	107	14					34	0.60						
	4	17	Sandy Clayey Silt	109	20												
	5	21	Sandy Silty Clay	110	19												
	6	27	Silty Clay	100	23					20	1.30						
	7	31	Silty Clay	103	23	56	38										

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
15-1	8	41	Sandy Silt	94	30				1.8								
	9	46	Clayey Silt	107	22												
	10	51	Sandy Clay	112	17												
	11	56	Sand	114	15												
	12	61	Sand	108	19			4.1×10^{-4}								X	
	13	66	Silty Sand	108	19					27	0.95						
	14	76	Siltstone	90	33				7.4								
	15	79	Siltstone	92	29												
	16	84	Siltstone	91	27											X	
15-2	1	2	Silty Clay	107	21												
	2	7	Clayey Silt	105	20												
	3	13	Clay	103	23	51	34			36	1.55						
	4	17	Silty Clay	102	25												
	5	23	Silty Sand	116	18					37	0.60						
	6	27	Silty Clay	107	23												
	7	32	Silt	98	26				6.6								
	8	37	Silty Clay	105	22												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
15-2	9	47	Silty Sand	95	30					32	0.45						
	10	52	Sandy Silt	116	17												
	11	57	Sandy Clay	112	16												
	12	63	Silty Sand	98	28											X	
	13	72	Sand	106	19												
	14	77	Silty Sand	121	14												
	15	86	Siltstone	93	29				4.1								
15-3	1	2	Sandy Clay	107	17												
	2	6	Silty Sand	105	18												
	3	11	Sandy Silty Clay	93	30				4.2								
	4	16	Silty Clay	105	21				7.7								
	5	21	Sandy Clay	113	17												
	6	26	Silty Sand	105	19				3.7								
	7	31	Silty Clay and Sand	119	14				5.2								
	8	36	Sandy Silty Clay	106	22												
	9	41	Silty Clay	106	22				10.3								
	10	46	Sandy Silt	96	28												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
15-3	11	56	Sandy Silt	107	19												
	12	62	Sandy Silt	114	16			$1.0x10^{-6}$								X	
	13	66	Silty Sand	118	16												
	14	71	Silty Sand	107	17												
	15	76	Sand	122										X			X(3)
	16	81	Disturbed														
	17	89	Siltstone	91	26											X	
	18	110	Clayey Siltstone	82	34											X	
	19	130	Clayey Siltstone	86	34											X	
	20	145	Siltstone	81	40												
	21	160	Siltstone	84	35											X	
15-4	1	2	Sandy Clay	107	17												
	2	7	Clayey Silt	98	26				5.0								
	3	12	Sandy Clayey Silt	99	23				2.9								
	4	17	Silty Clay	101	24												
	5	22	Sandy Silty Clay	109	15												
	6	27	Silty Clay	100	25												X(2)

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
15-4	7	32	Silty Clay	96	27				4.2								
	8	37	Sandy Clayey Silt	106	20				1.7								
	9	42	Silty Clay	101	24				7.2								
	10	47	Clayey Silt	93	28				3.1								
	11	52	Silty Clay	109	21												
	12	58	Sand	111	15											X	
	13	62	Sandy Clay	109	19												
	14	67	Silty Sand	110	15												
	15	72	Sand	108	19			6.5×10^{-4}								X	
	16	79	Silty Sand	119	10					38	1.0						
	17	84	Disturbed														
15-5	1	2	Clayey Sand	113	14					22	1.0						
	2	11	Silty Sand	102	20												
	3	16	Sandy Clayey Silt	100	24				7.9								
	4	21	Silty Sand	98	25												
	5	26	Silty Sand	105	18				3.6								
	6	31	Silty Sandy Clay	102	23												

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
15-5	7	36	Silty Clay	115	17												
	8	41	Sandy Clayey Silt	108	18				8.0								
	9	46	Clayey Silt	104	22												
	10	51	Silty Clay	108	22												
	11	56	Sandy Clay	110	18												
	12	61	Clayey Sand	112	16											X	
	13	66	Sandy Silty Clay	114	16												
	14	71	Sand	107	19					25	0.85						
	15	76	Sand	105	19												
	16	81	Disturbed														
	17	93	Siltstone	87	30				5.1								

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

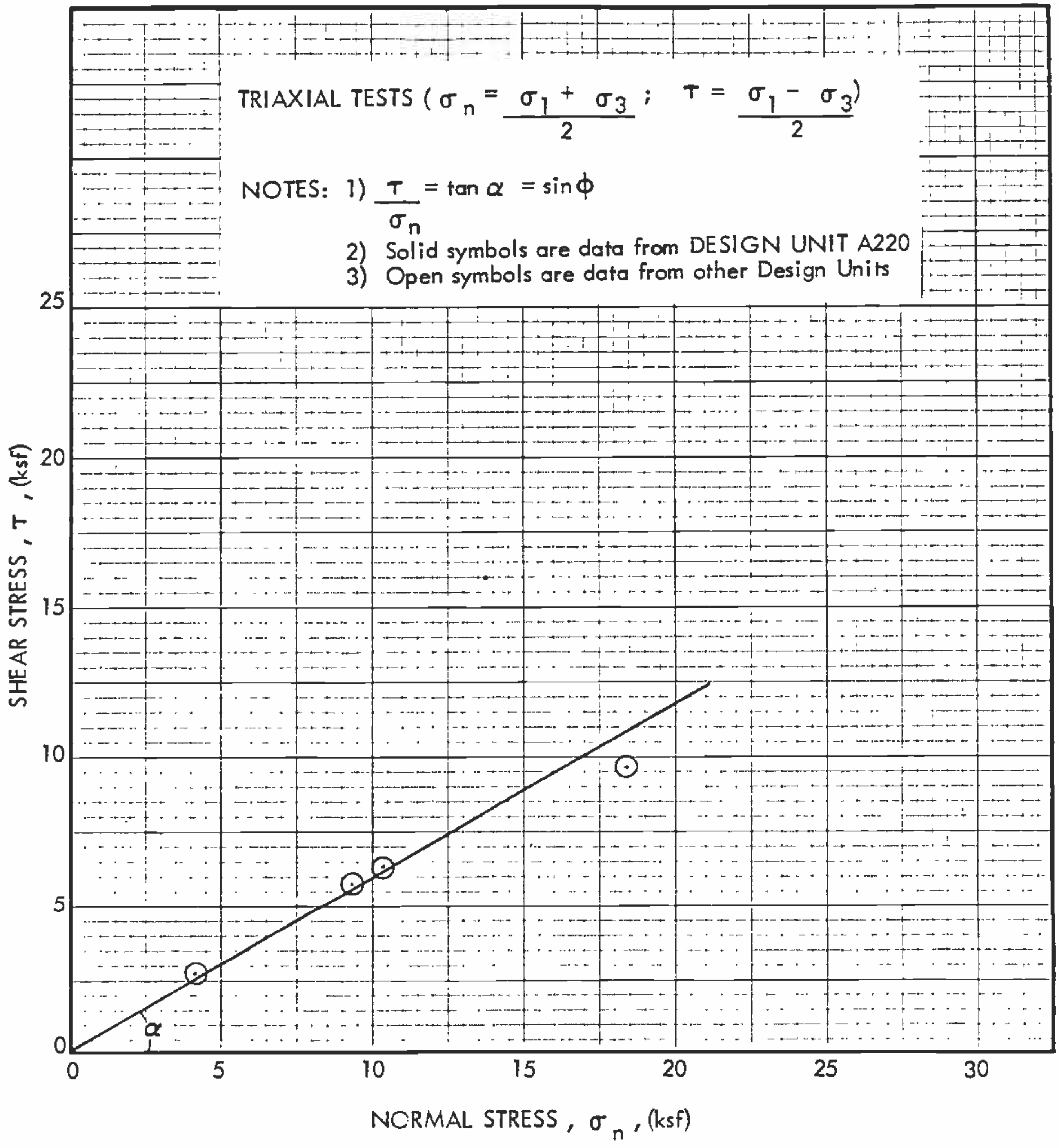
CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits LL	Atterberg Limits PI	Particle Size Cumulative % Passing Sieve No. 4	40	200	Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Compression Index/(1 + e_o) $Ce/1+e_o$ (in/in per log cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	c, ksf	One-Dimensional Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
14	C1	26	Clayey silt & clay, bedded	A_4	79	48	53	28	100	97	91		2.0E-7			53.2							cup
	S1	45	Siltstone	C	91	28	35	17					3.9E-8			45.8					X		
	S1	45	Siltstone	C	89	32																	Q
	S1	46	Claystone	C	96	29																X	
	S2	59	Claystone, beds of siltstone, sandstone	C	84	36	55	16															Q
	S3	60	Siltstone, bedded & folded with claystone	C	79	39																	Q
	S4	80	Siltstone & sandstone	C	89	32	47	12															Q
	S4	80	Siltstone & sandstone	C	91	31						81.5											
	S5	97	Siltstone	C	90	28						139											
	S6	114	Siltstone	C	93	30	47	10															Q
	S7	131	Siltstone, folded bedding	C	91	30	51	13															Q
	S8	148	Siltstone	C	86	34						157											
	S9	164	Siltstone bedding folded with sandstone	C	95	28	44	12														X	
	S9	165	Siltstone bedding folded with sandstone	C	93	29																	Q
	S9	167	Gray claystone	C	95	28																X	
	S10	181	Claystone & siltstone, folded	C	99	26						110											
	S11	198	Claystone & siltstone, folded	C	93	30						143											
15	C1	21	Mottled silty clay	A_4	94	28											31	1.09					
	C1	21	Mottled silty clay	A_4	95	29								.107					9.11				
	C2	41	Fine to medium sandy micaceous silt	A_3	98	28							2.1E-7		2.70	41.0							
	C2	41	Fine to medium sandy micaceous silt	A_3	101	25											23.5	1.25					
	C3	61	Gray clay, stiff	A_4	103	24	53	24	100	95	69		2.1E-8		2.71	39.4							cup
16	S1	18	Silty clay	A_4	96	27							1.0E-6			42.5							cu
	S1	19	Silty clay	A_4	99	23																	cu
	S1	19	Silty clay	A_4	99	25	43	19	100	98	81	39.5											
	S1	19	Silty clay	A_4	93	29													3.31				

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

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits LL	Atterberg Limits PI	Particle Size Cumulative % Passing Sieve No. 4	40	200	Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Compression Index/(1 + e₀) Cc/1+e₀(in/in per log cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	c, ksf	One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
16	S2	35	Silty clay	A_4	87	35						37.3											
	S2	35	Silty clay	A_4	87	33	49	26					3.3E-7			46.0					X		
	S2	35	Silty clay	A_4	92	31															X		
	S2	36	Silty clay	A_4	94	28																X	
	S3	51	Sandy clay	A_4	98	26	37	16	98	80	67		1.7E-6			36.9							
	S3	52	Sandy clay	A_4	98	26															X		
	S3	51	Sandy clay	A_4	114	16																X	
	S3	52	Sandy clay	A_4	106	21						40.0											
	S3	52	Sandy clay	A_4	98	16																	
	S3	53	Sandy clay	A_4	114	16																X	
	S4	71	Fine to medium sand	SP	93	14			99	84	6		3.4E-4		2.67	41.0	32	0.17			X		
	S4	71	Fine to medium sand	SP	97	23																X	
	S4	71	Fine to medium sand	SP	93	14																	
	S5	74	Fine to medium sand	SP	102	22																	
	S5	74	Gray fine to medium sand	SP	103	19											33	.07					
	S5	75	Gray fine to medium sand	SP	104	21																X	
	S6	101	Siltstone	C	90	31															X		
	S6	101	Siltstone	C	89	31	47	8	100	91	78		1.1E-7			46.8					X		
	S7	118	Siltstone, clayey & micaceous	C	96	26	44	11				131											
	S8	129	Siltstone, folded bedding	C	83	37						9.5											
	S9	134	Siltstone	C	95	27						151											
	S10	147	Siltstone	C	89	31						124											
	S11	163	Claystone, silty & micaceous	C	93	27						167											
	S12	176	Claystone, varves siltstone	C	98	26						173											
	S13	179	Siltstone	C	95	28						142											
	S14	195	Siltstone, varves sandstone	C	88	30						77.8											
17	C2	41	Siltstone, pocket of sandstone	A_4	95	28															X		

TABLE C-2 C[illegible]ensive List of Soils Engineering Properties from Laboratory Tests (contin[illegible]

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits LL	Atterberg Limits PI	Particle Size Cumulative % Passing Sieve No. 4	40	200	Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Compression Index/(1 + e₀) Cc/1+e₀(in/in per log cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	c, ksf	One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
17	S1	64	Siltstone, pocket of sandstone	C	89	32						99.3											
	S3	101	Siltstone	C	85	35						109											
	S4	124	Siltstone, folded bedding	C	84	37						124											
	S5	138	Siltstone, folded bedding	C	93	30						152											
	S6	155	Siltstone, folded bedding	C	89	33						183											
	S7	172	Siltstone, folded bedding	C	90	32						178											
	S9	200	Claystone, very fine sandstone varves	C	78	41						174											



EFFECTIVE STRENGTH DATA - GRANULAR ALLUVIUM

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Project No. 83-1140

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Figure No. C-1

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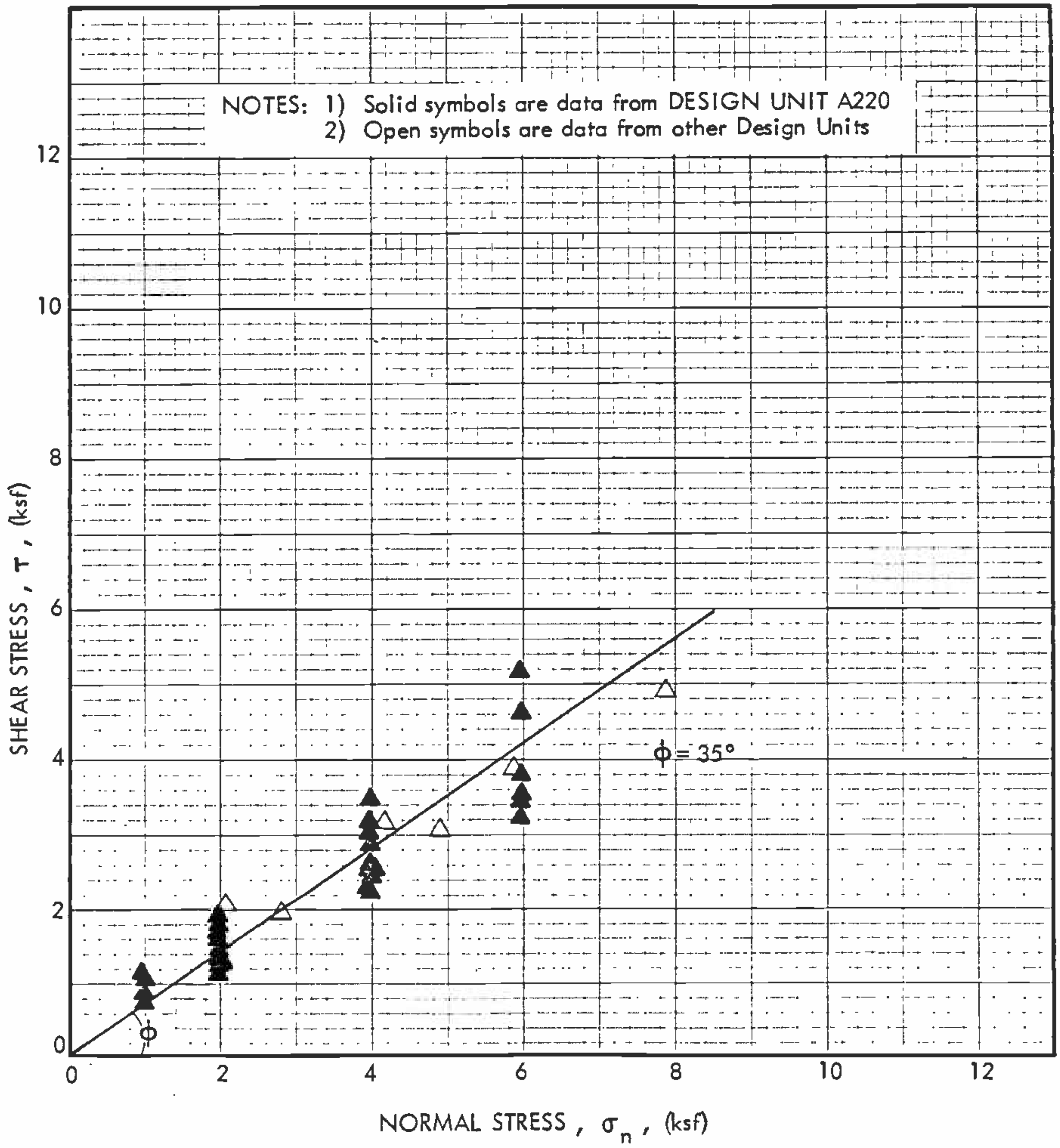

SUMMARY OF DIRECT SHEAR TESTS RESULTS - GRANULAR ALLUVIUM

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Figure No.
C-2

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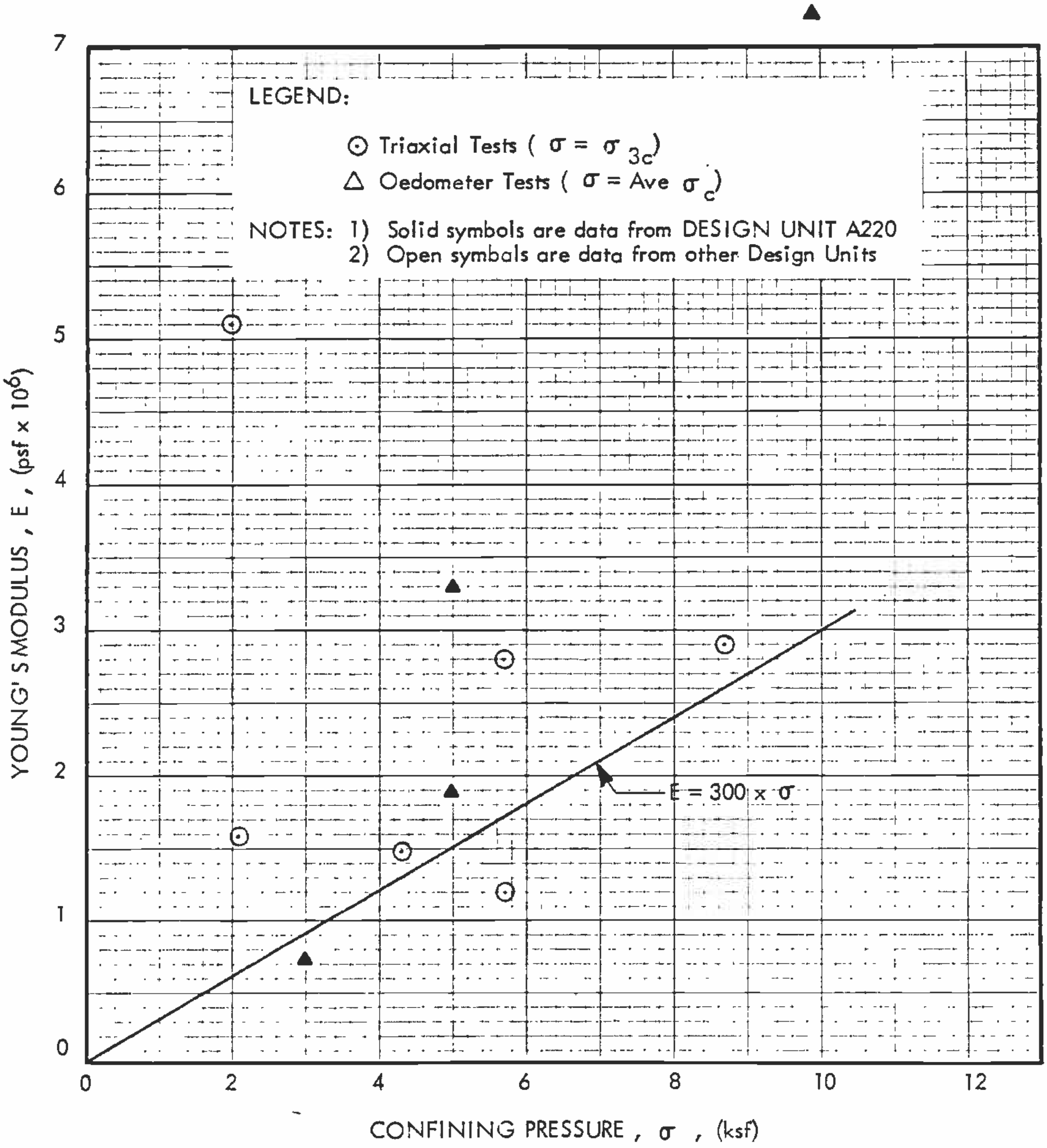

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SUMMARY OF MODULUS DATA - GRANULAR ALLUVIUM

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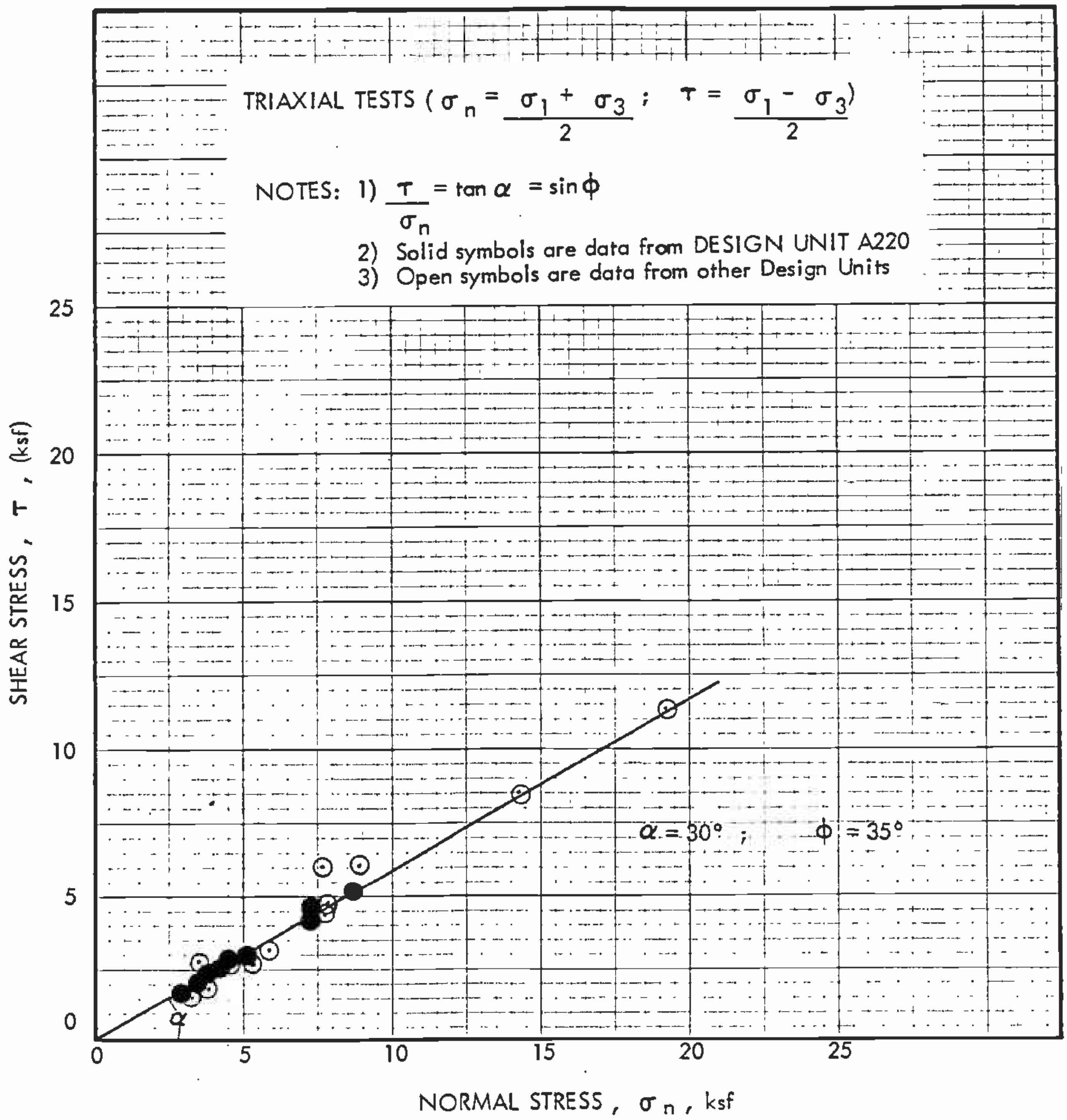
Figure No. C-3



SUMMARY OF EFFECTIVE STRENGTH DATA - FINE-GRAINED ALLUVIUM

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Figure No C-4

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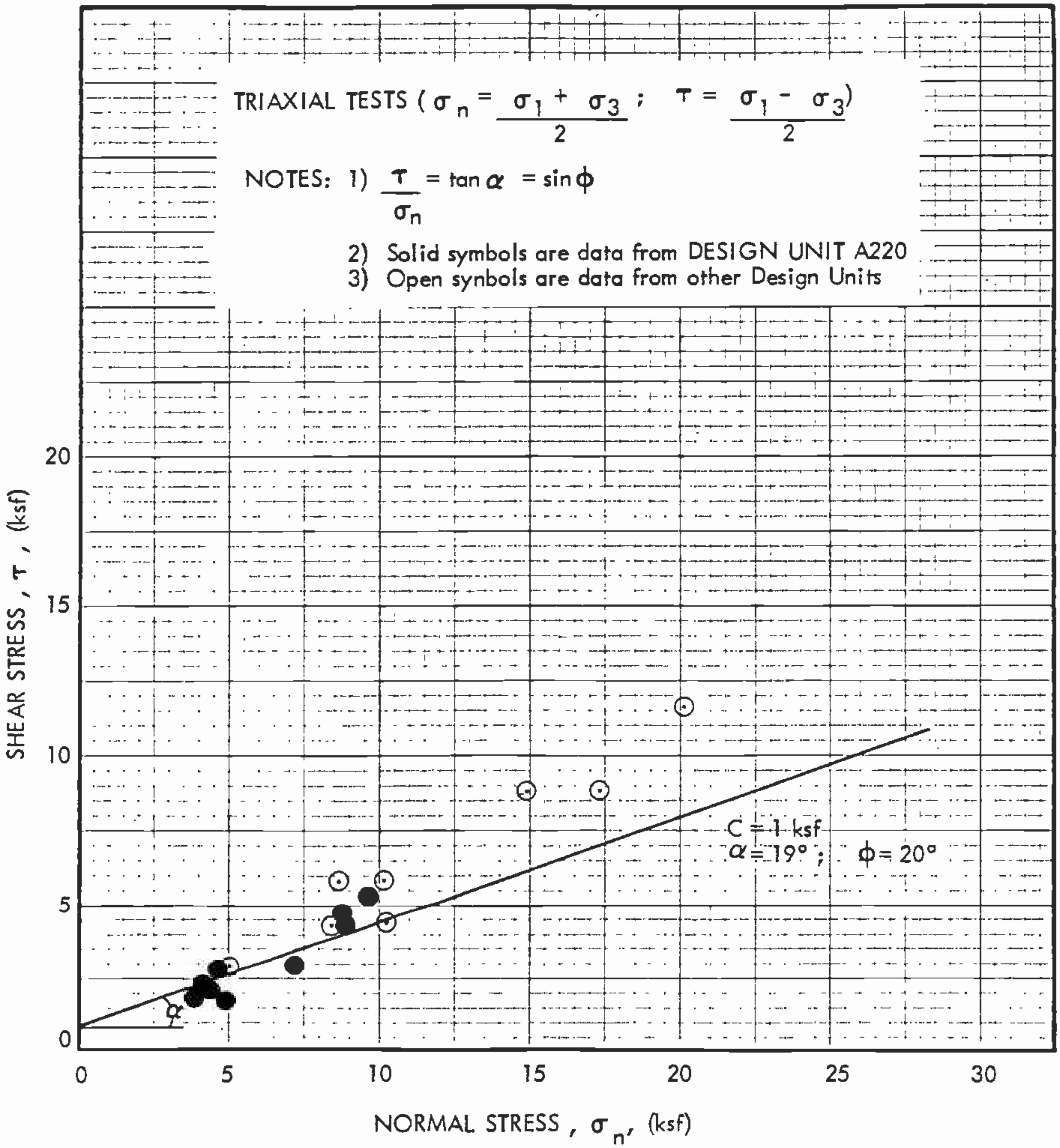

SUMMARY OF TOTAL STRENGTH DATA – FINE-GRAINED ALLUVIUM

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Figure No. C-5

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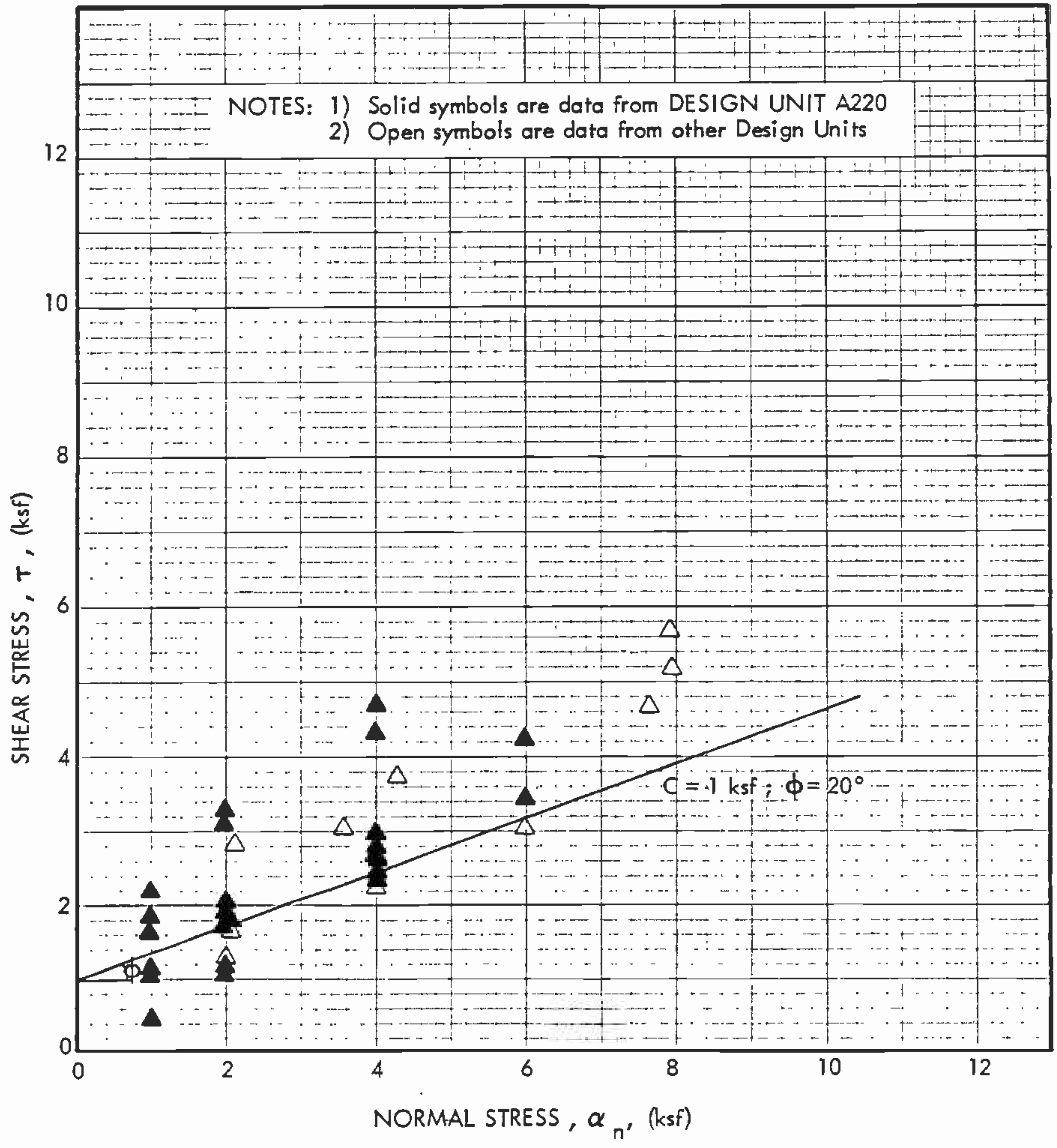

SUMMARY OF DIRECT SHEAR DATA - FINE-GRAINED ALLUVIUM

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Figure No. C-6

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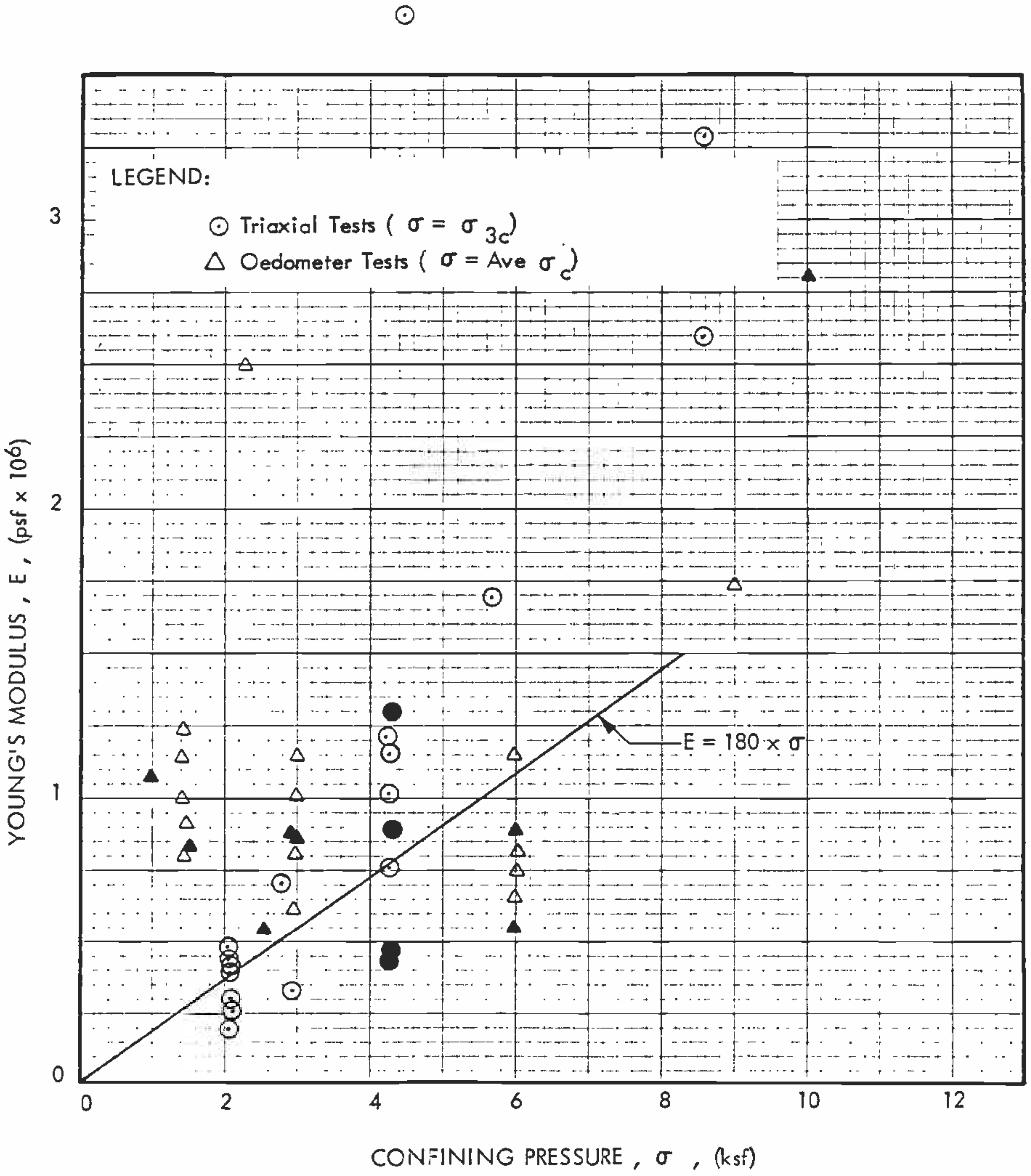

SUMMARY OF MODULUS DATA - FINE-GRAINED ALLUVIUM

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Figure No. C-7

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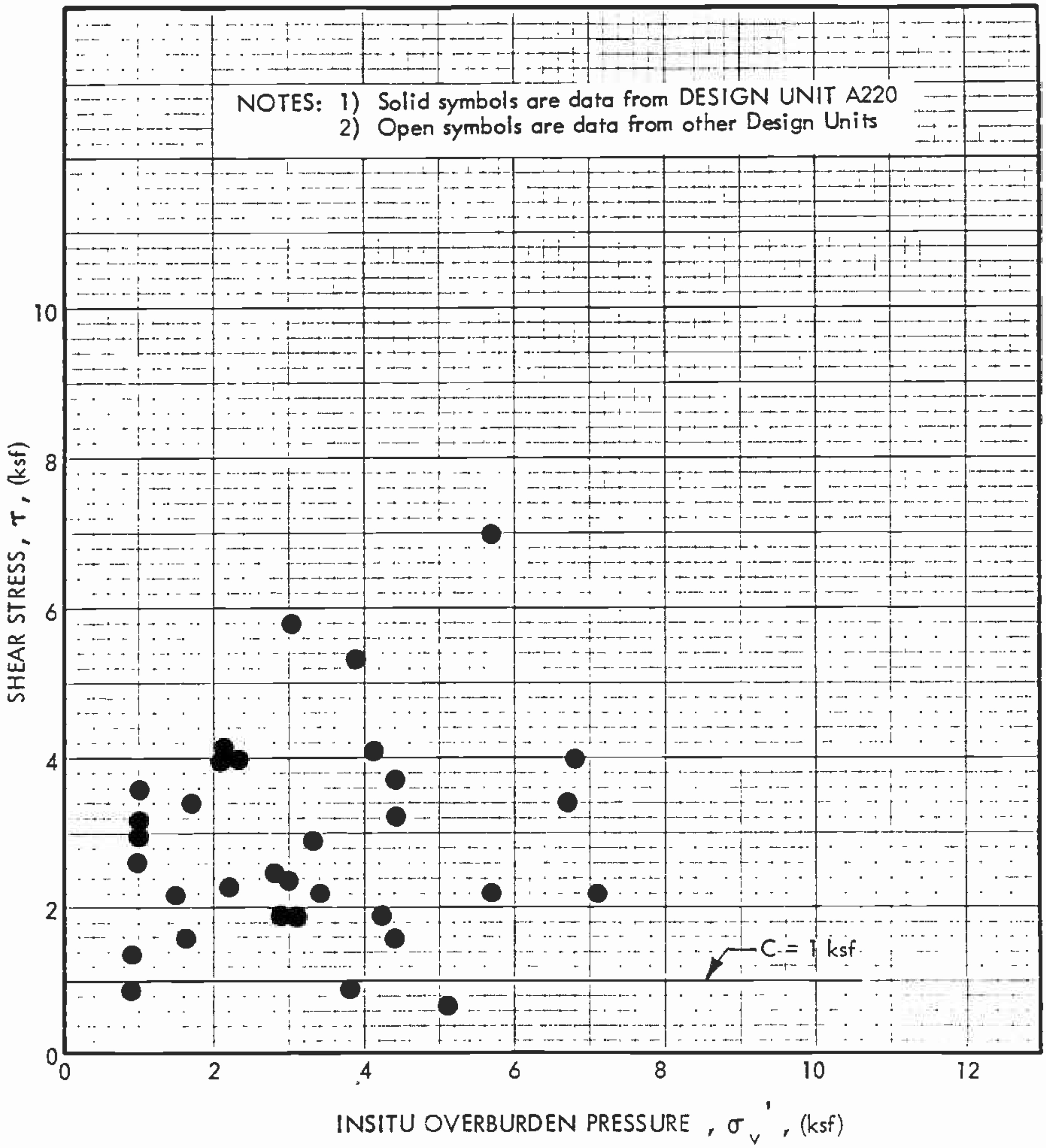
NOTE:

$C = \frac{\text{Unconfined compression strength}}{2}$



SUMMARY OF UNCONFINED COMPRESSION TESTS - ALLUVIUM

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Figure No. C-8

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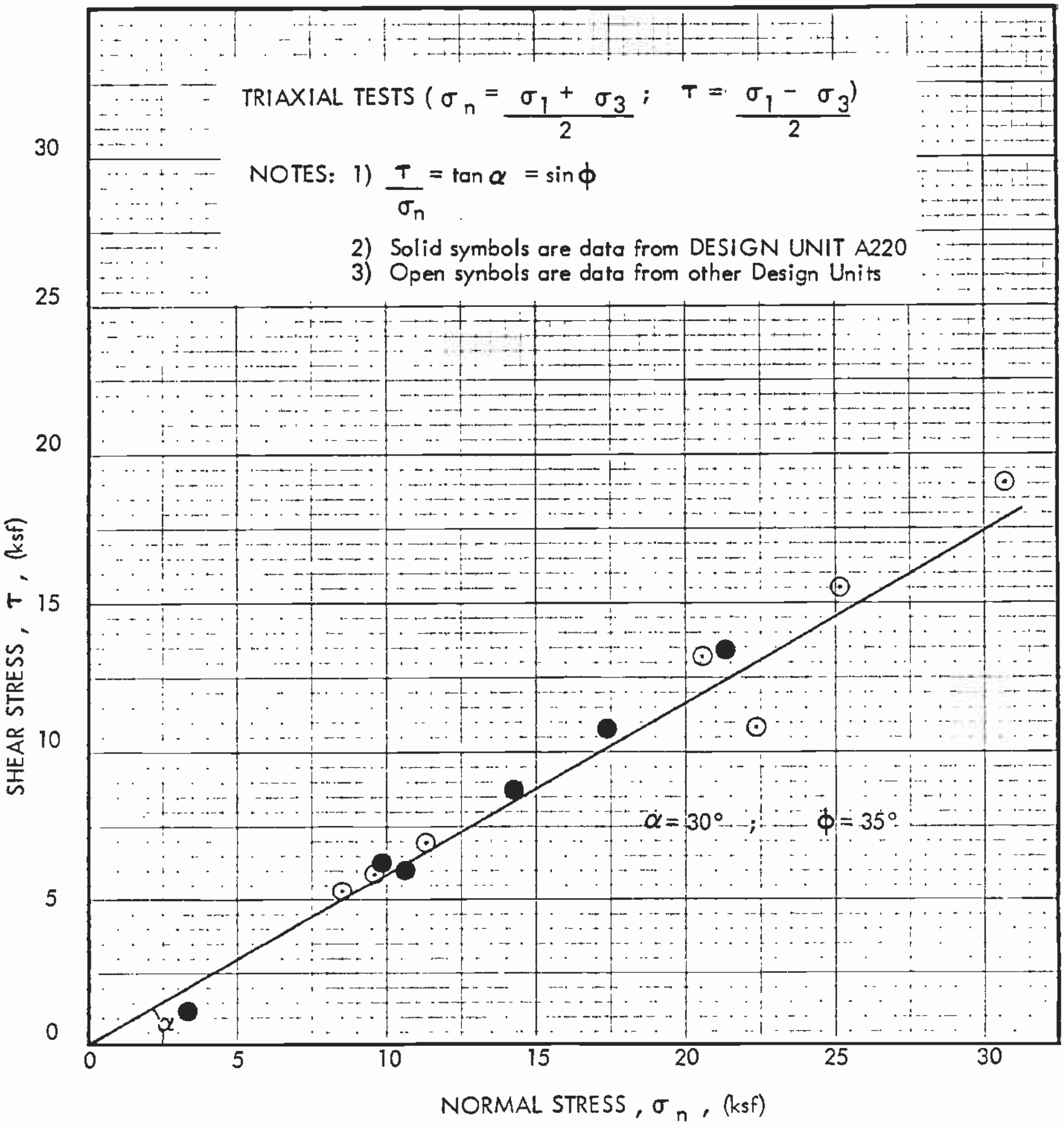

SUMMARY OF EFFECTIVE STRENGTH DATA - SAN PEDRO SAND

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Figure No. C-9

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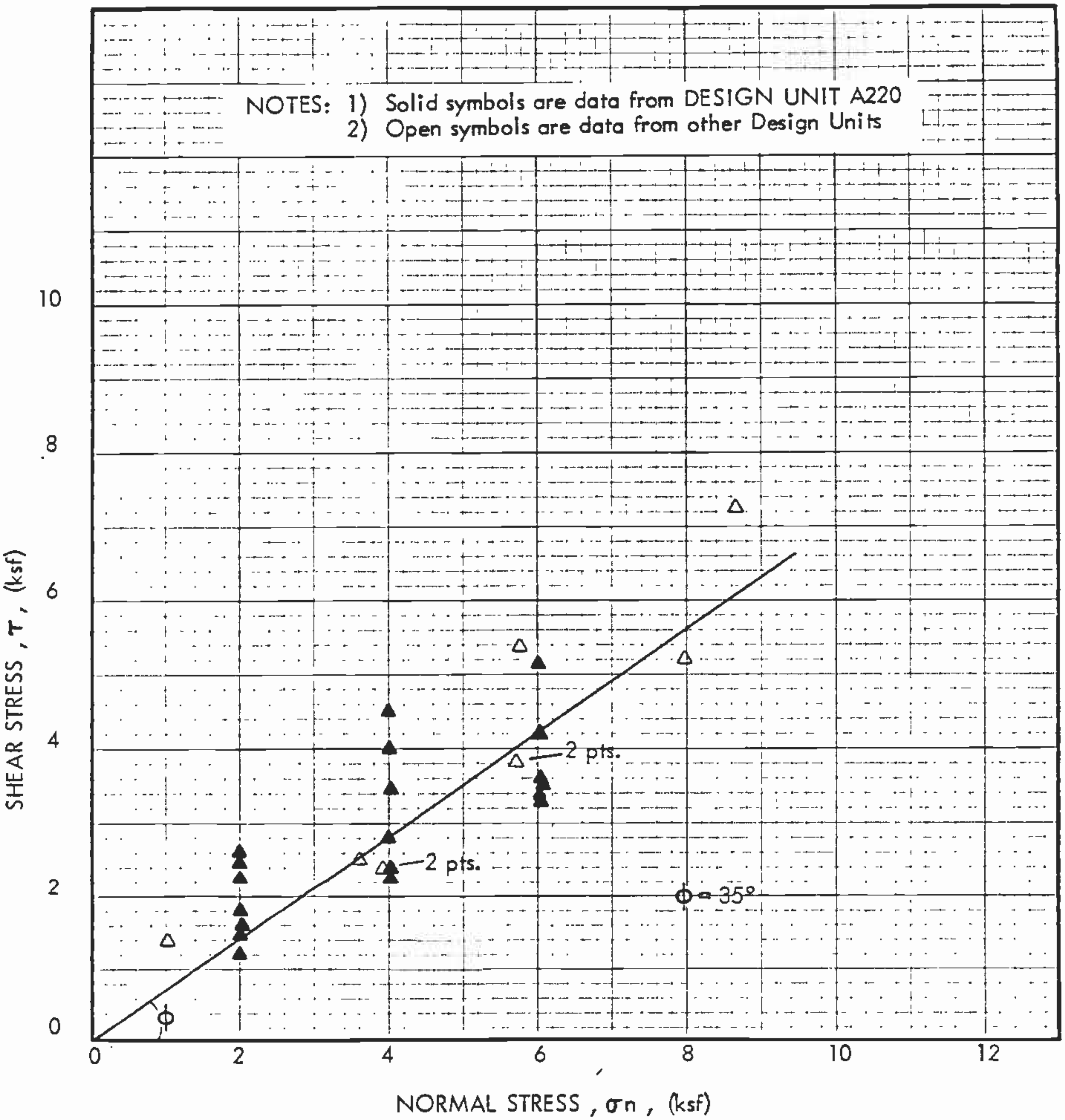

SUMMARY OF DIRECT SHEAR TEST RESULTS - SAN PEDRO SAND

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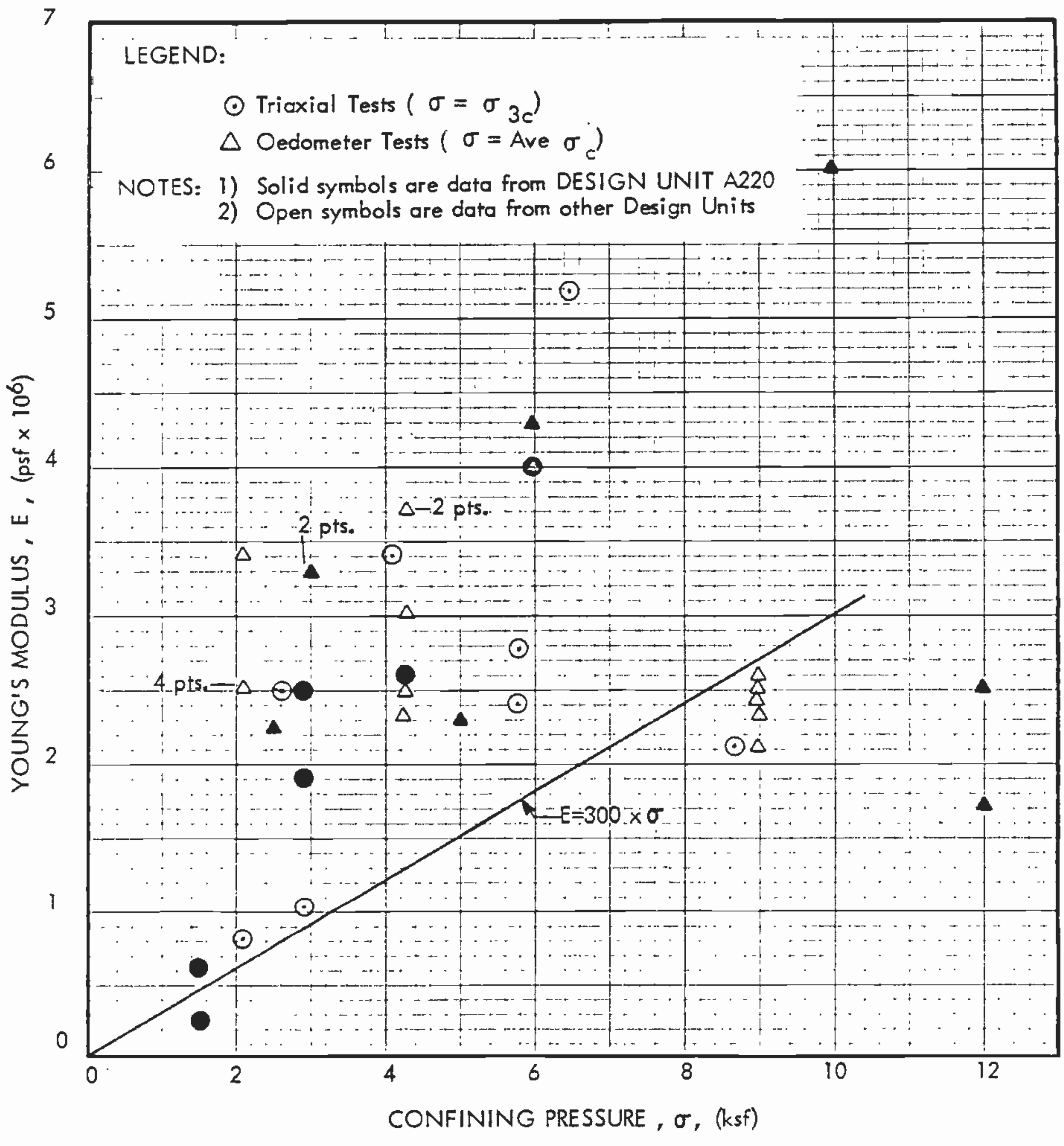
Figure No. C-10



SUMMARY OF MODULUS DATA - SAN PEDRO SAND

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Figure No.
C-11

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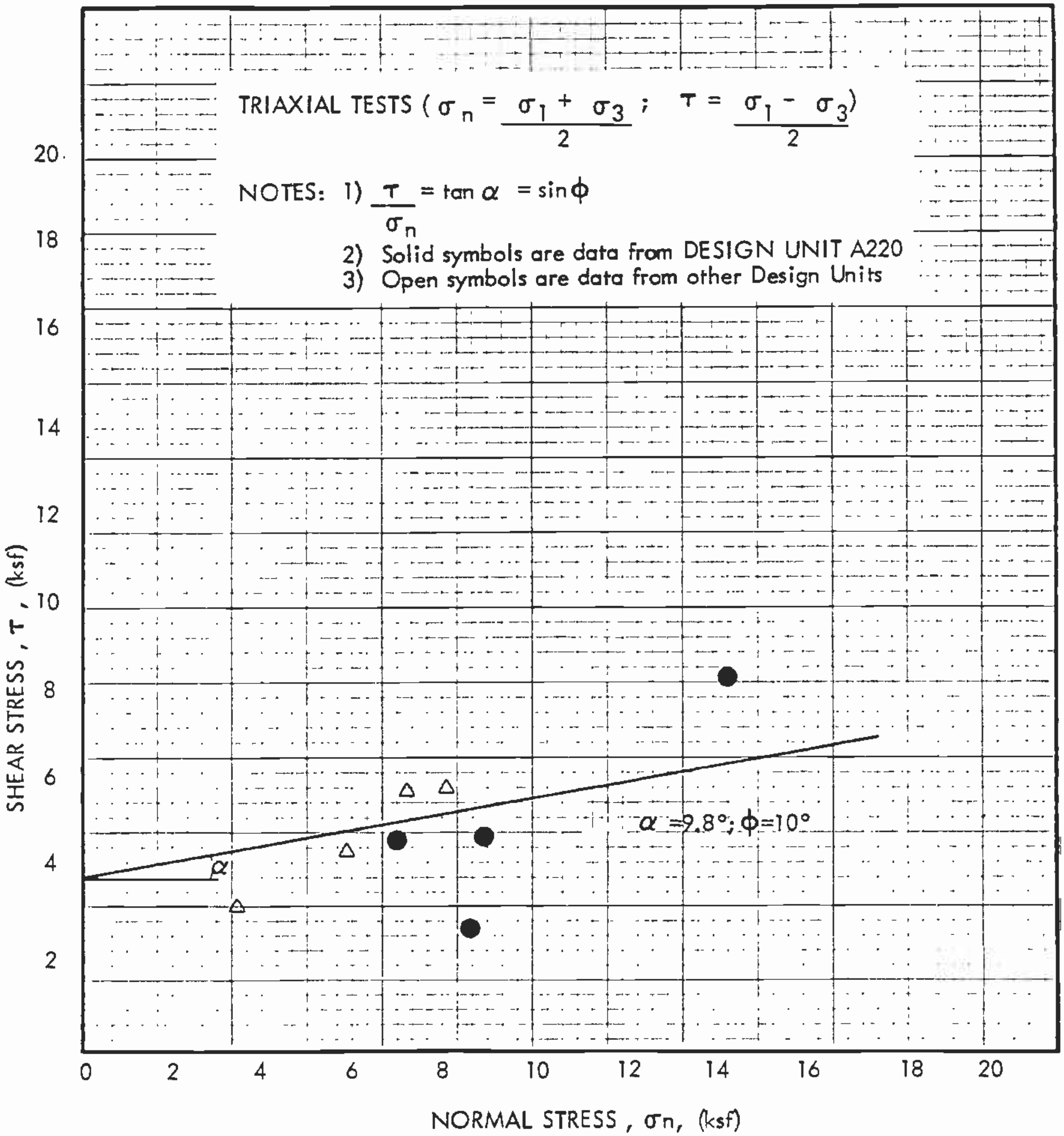

SUMMARY OF TOTAL STRENGTH DATA - BEDROCK

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83-1140

Figure No.
C-12

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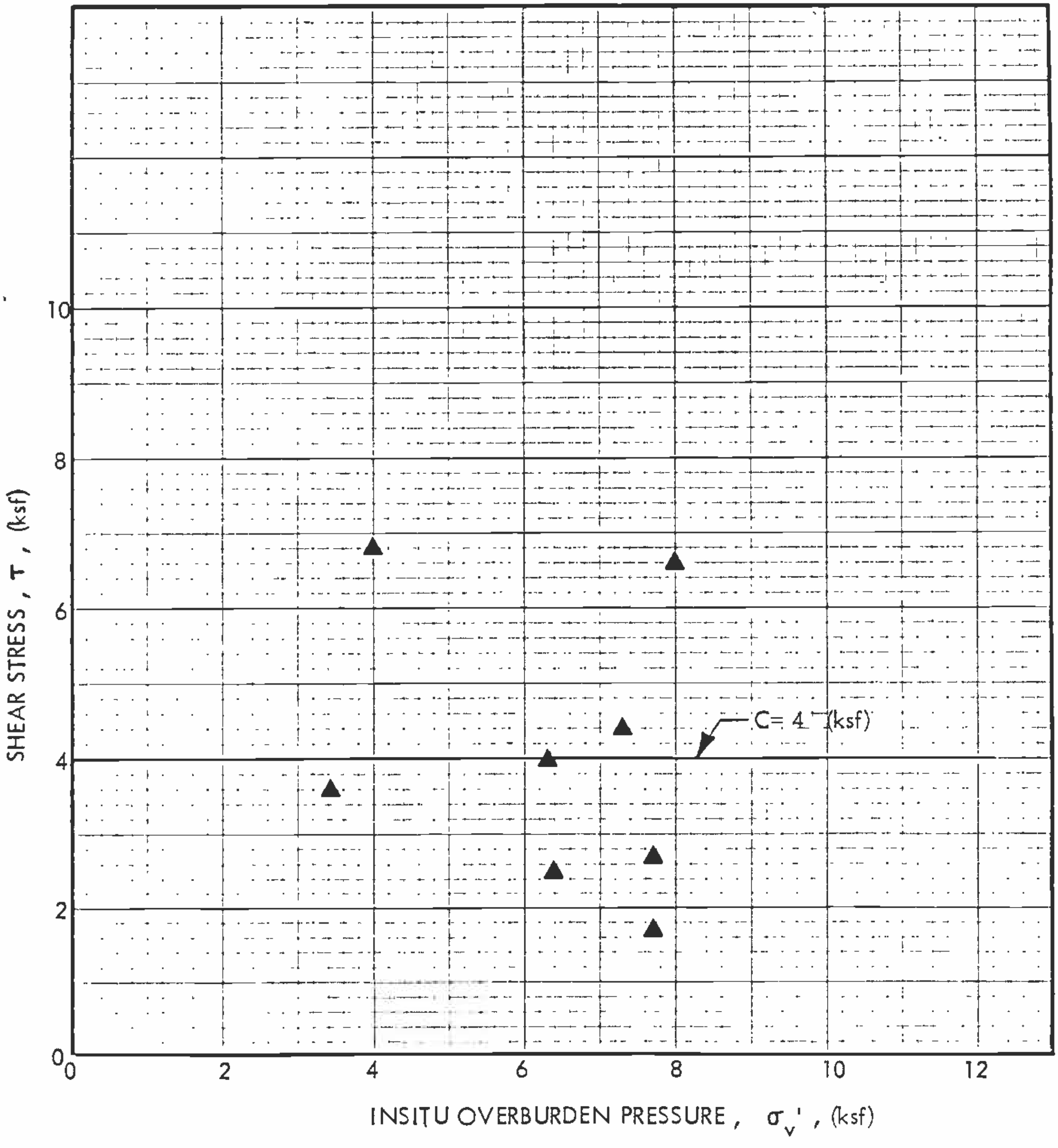
NOTE: $C = \frac{\text{Unconfined compression strength}}{2}$



SUMMARY OF UNCONFINED COMPRESSION TESTS - BEDROCK

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Figure No C-13

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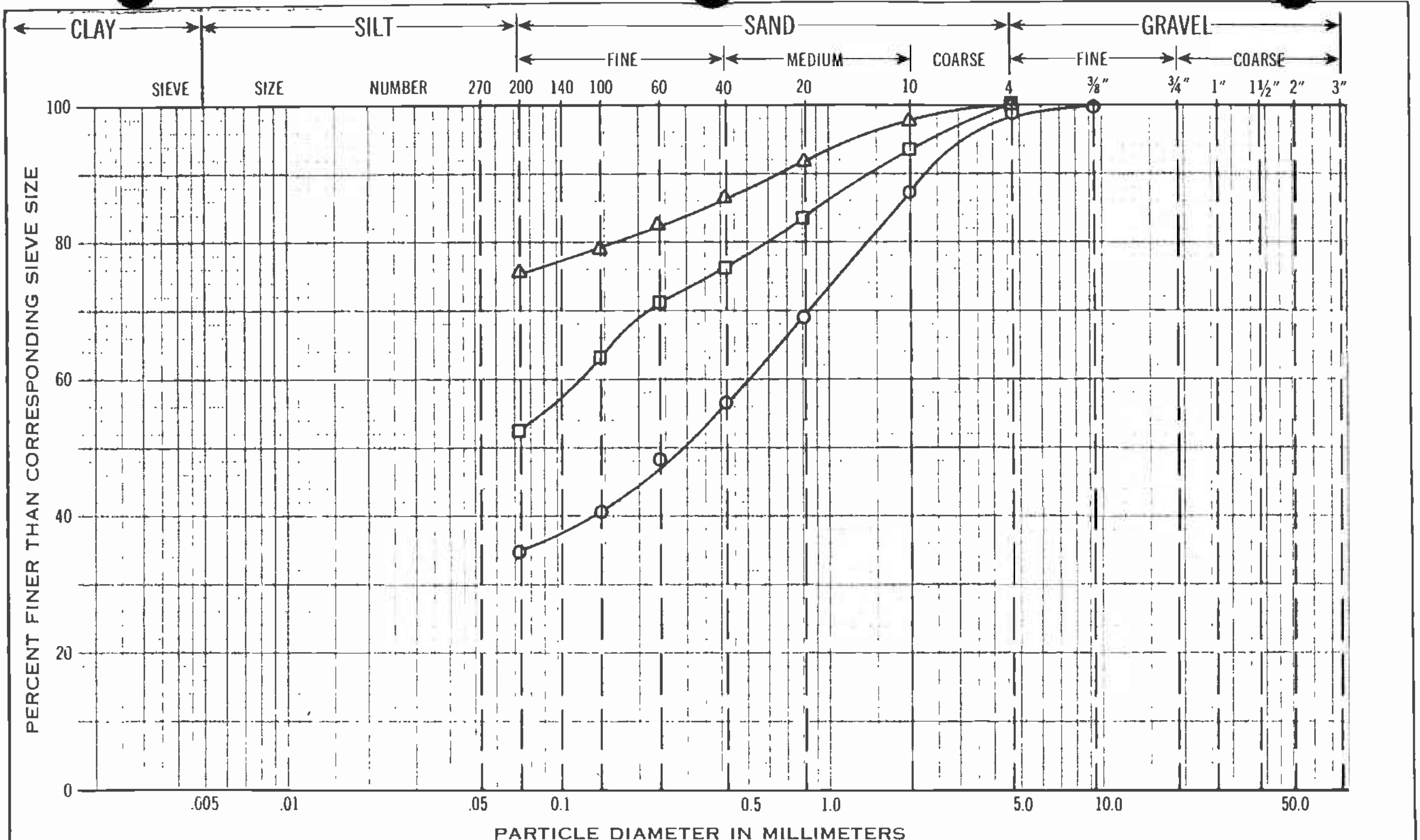

SYMBOL	BORING	SAMPLE	DEPTH
—o—	14/1	C-11	46.0'
—△—	14/1	C-12	52.0'
—□—	14/1	C-14	62.0'

GRAIN-SIZE DISTRIBUTION CHART

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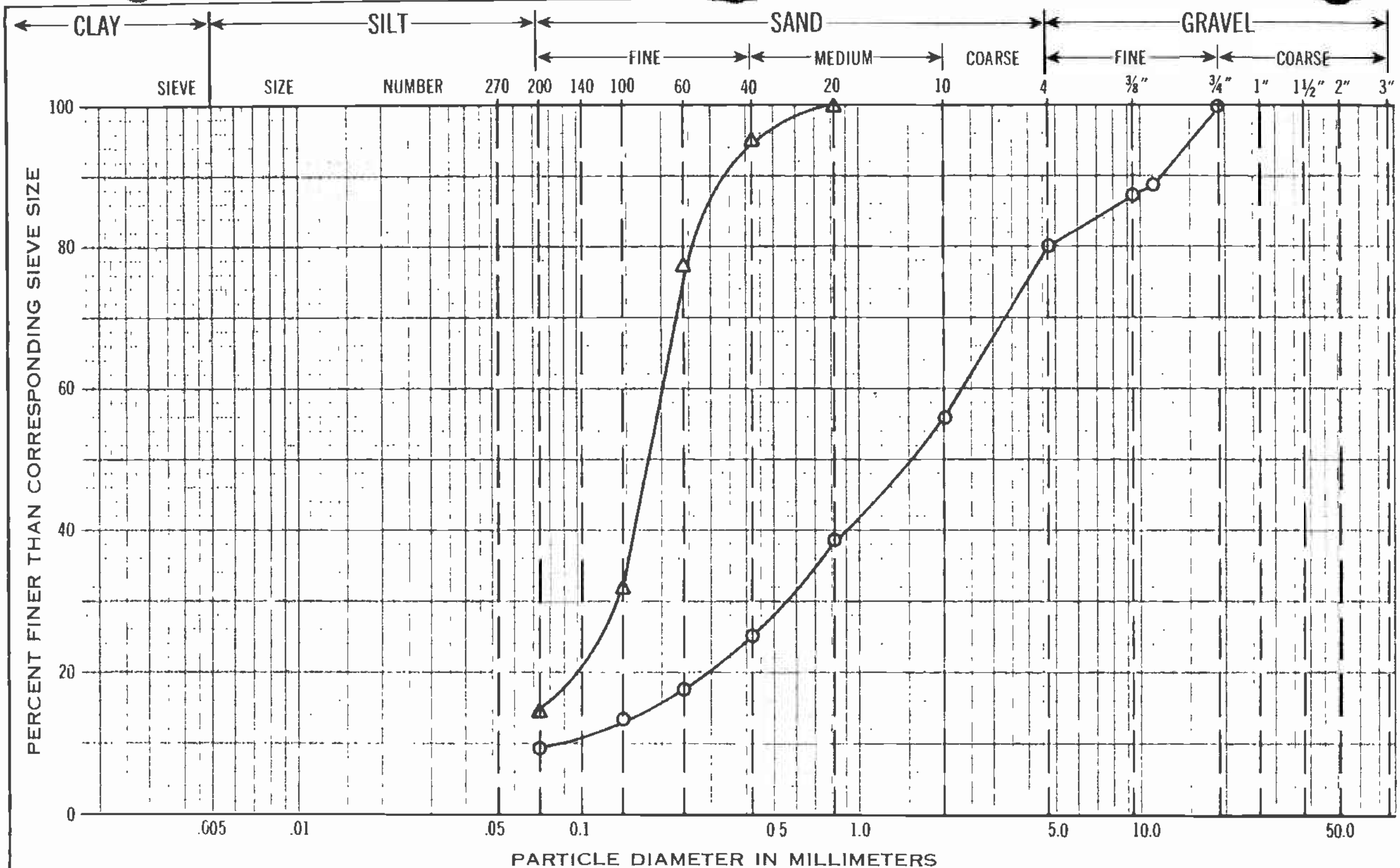
Figure No
C-14

FORM NO. D-3-10/8_ Approved for publication ________ by ________



SYMBOL	BORING	SAMPLE	DEPTH
—○—	14/2	C-9	42.5
—△—	14/2	C-16	79.5

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-15

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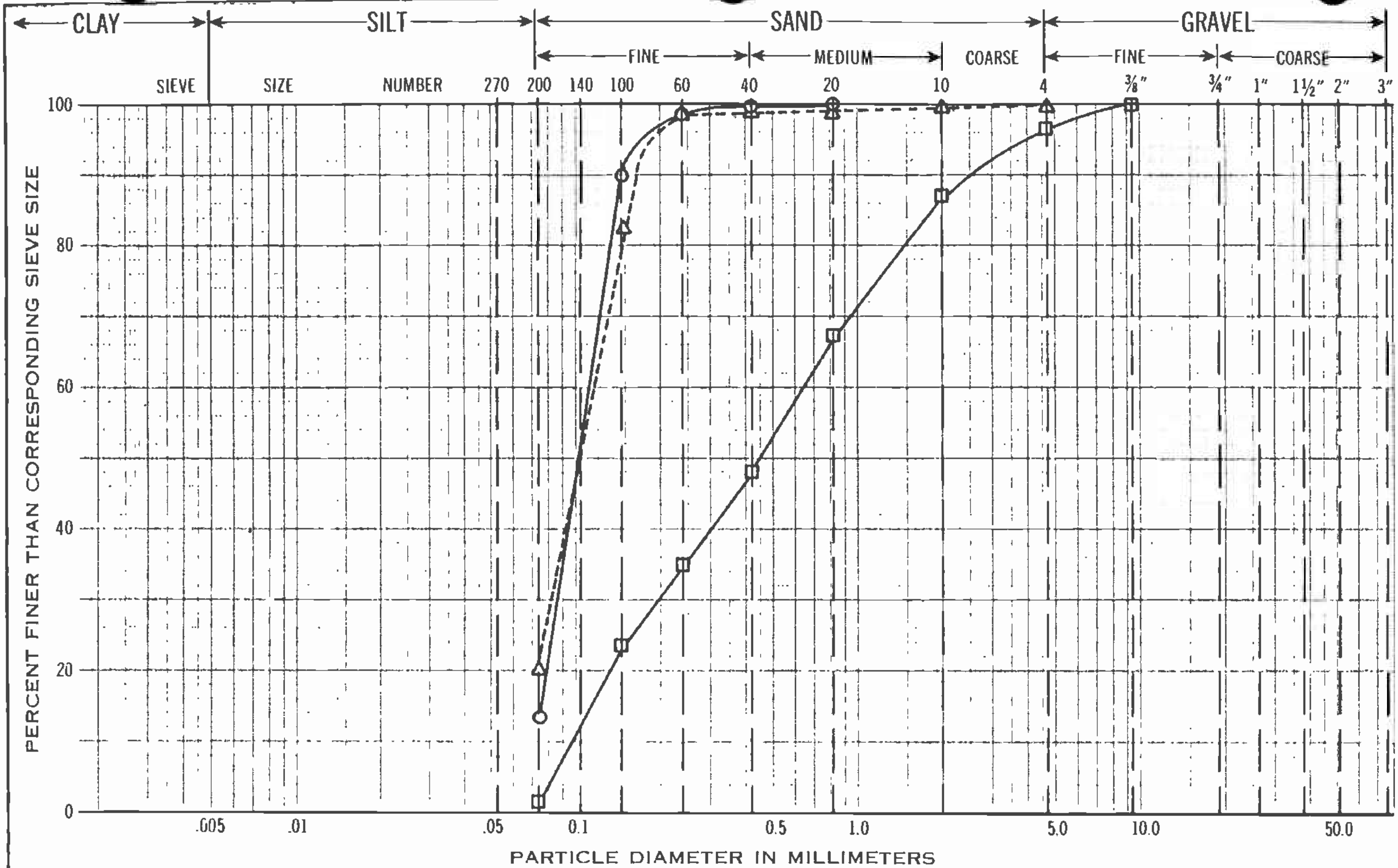

SYMBOL	BORING	SAMPLE	DEPTH
—□—	14/3	C-9	42.0'
—○—	14/3	C-12	57.5'
--△--	14/4	C-8	37.0'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No C-16

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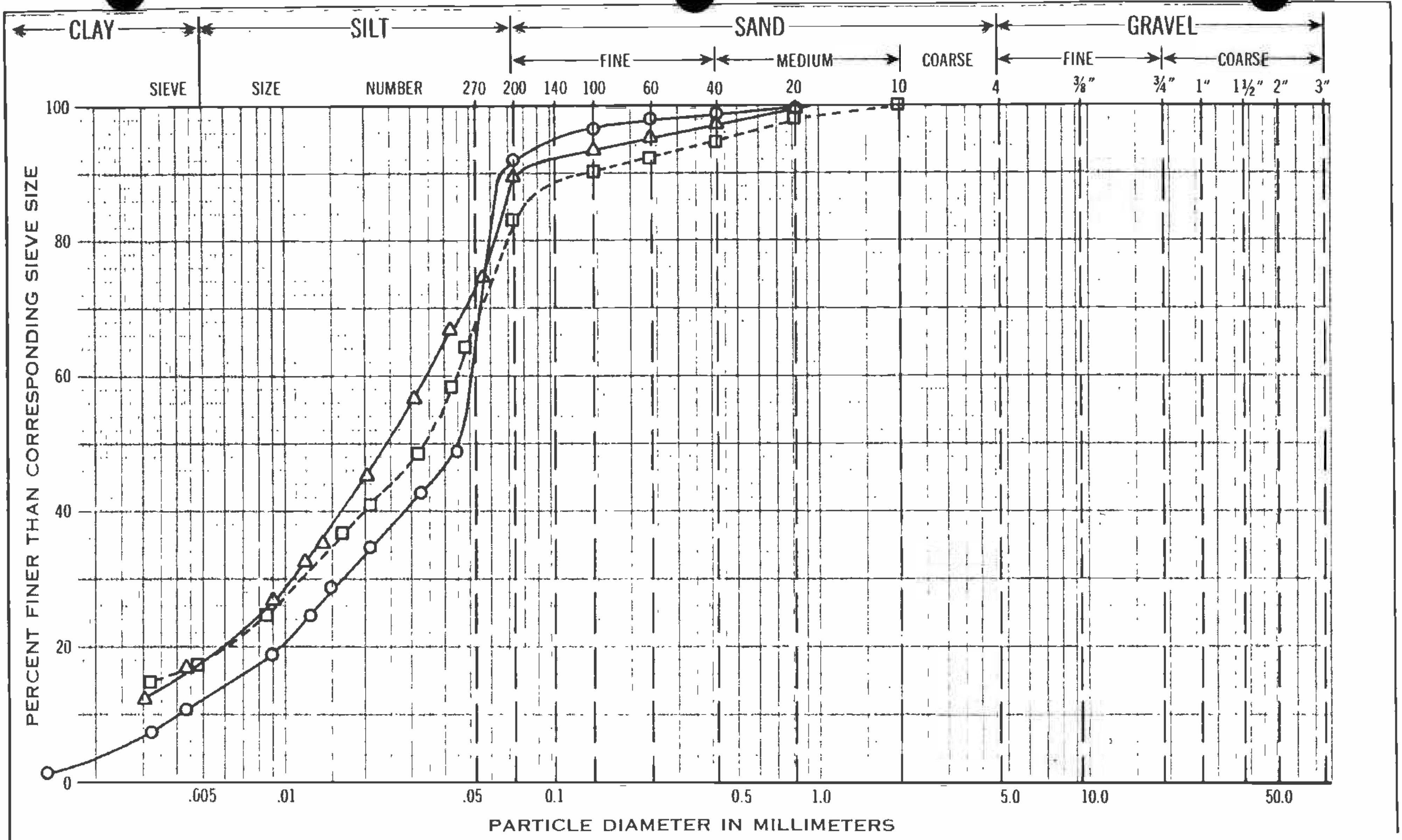

SYMBOL	BORING	SAMPLE	DEPTH
—○—	14/5	C-8	44.5'
—△—	14/5	C-11	65.0'
—□—	14/5	C-13	84.5'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-17

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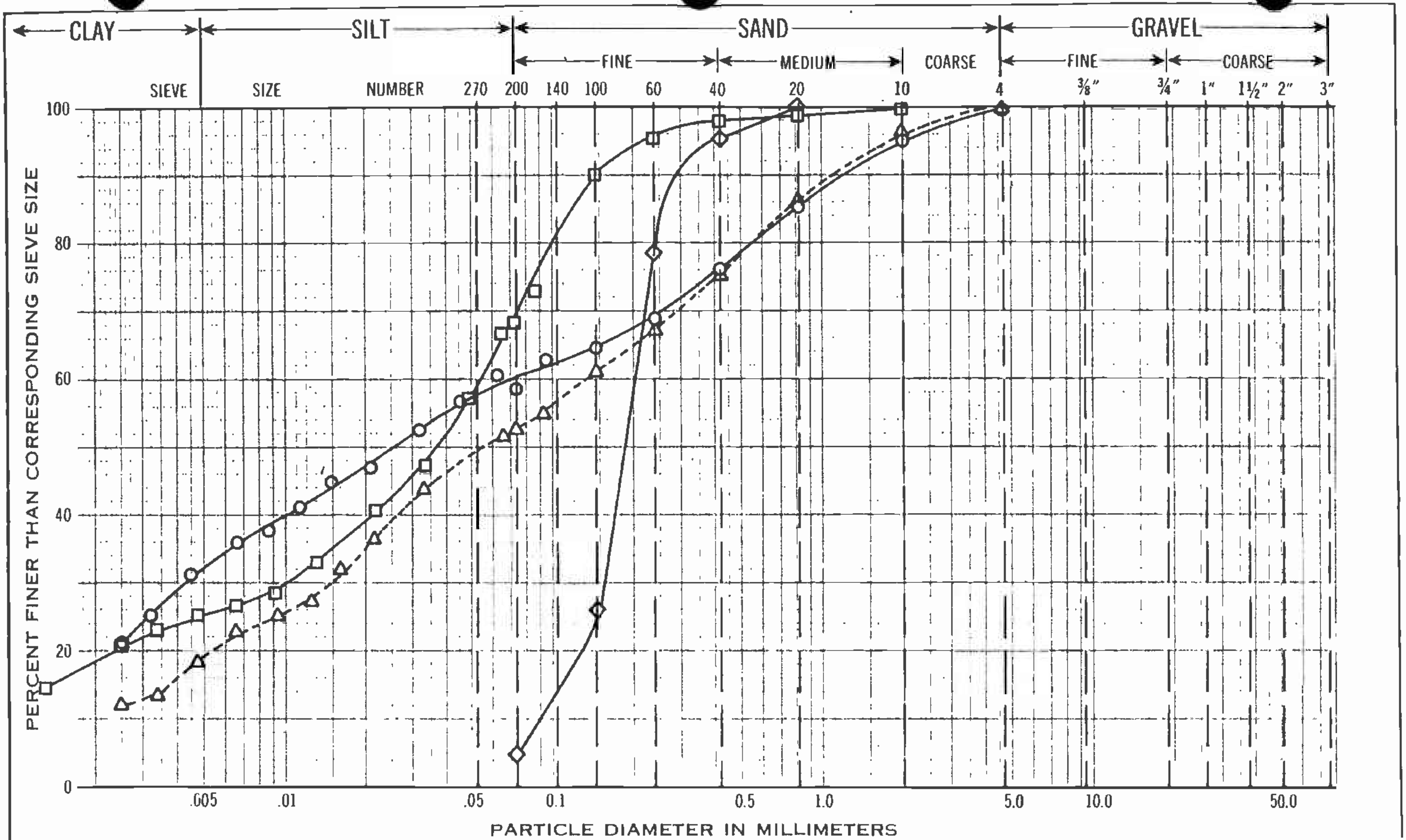

SYMBOL	BORING	SAMPLE	DEPTH
—○—	15/1	C-10	51.5'
- - -△- - -	15/2	C-11	57.5'
—◇—	15/3	C-15	76.5'
—□—	15/4	C-6	27.5'

GRAIN-SIZE DISTRIBUTION CHART

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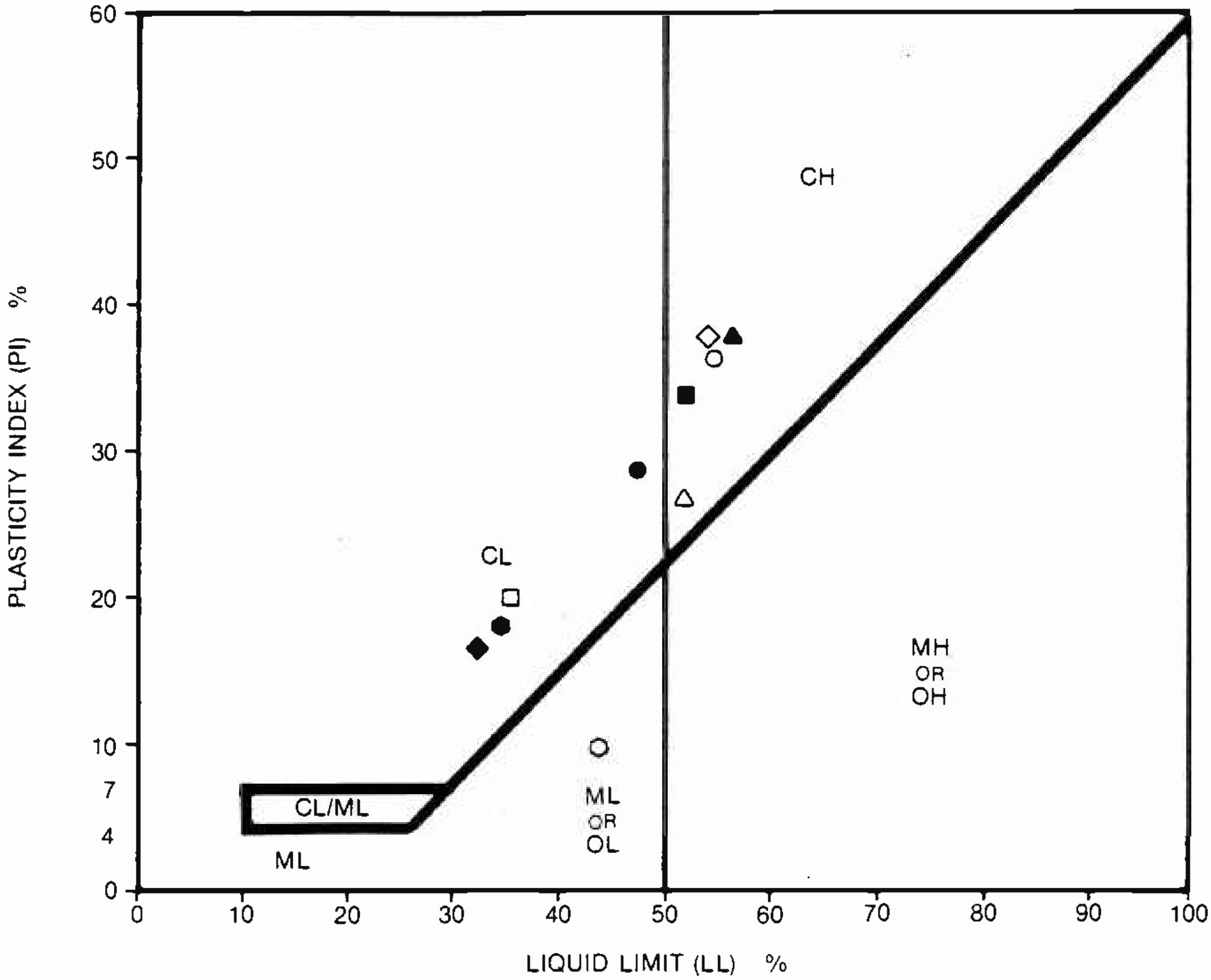
Figure No.
C-18



Symbol	Classification and Source				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
△	B-14/1	C-12	51.5'	CH	51	24	27	71.5
○	B-14/2	C-4	17.5'	CH	54	17	37	---
□	B-14/3	C-3	12.0'	CL	36	16	20	---
◇	B-14/4	C-2	7.0'	CH	54	16	38	---
⬡	B-14/5	C-11	64.5'	ML	45	35	10	89.4
▲	B-15/1	C-7	31.5	CH	56	18	38	---
●	B-15/1	C-10	51.5'	CL	48	19	29	58.3
■	B-15/2	C-3	12.0'	CH	51	17	34	---
◆	B-15/2	C-11	57.5'	CL	33	16	17	52.5
⬢	B-15/4	C-6	27.5'	CL	35	17	18	68.2

PLASTICITY CHART

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Figure No C-19

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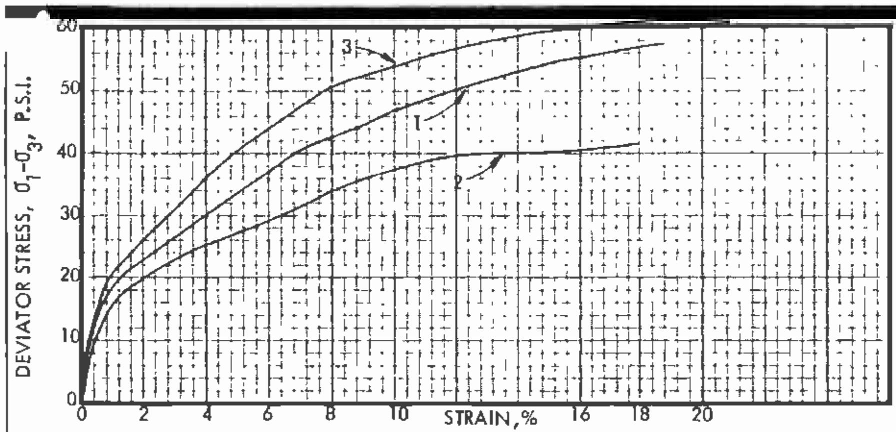
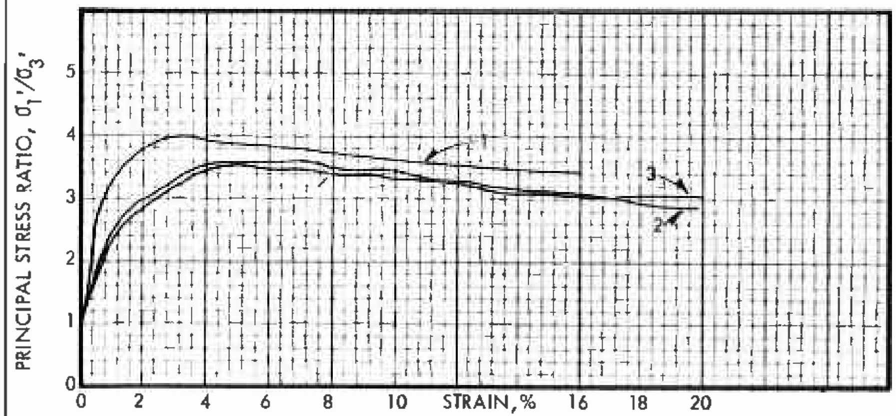
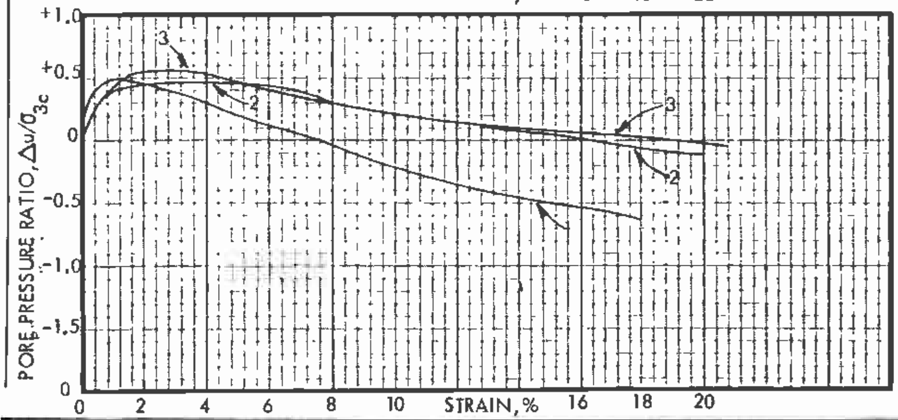
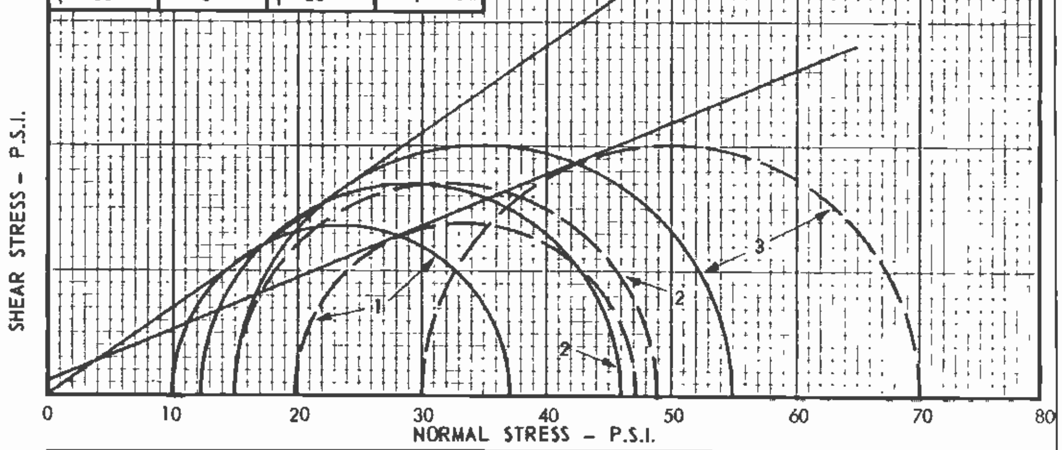






SHEAR VALUES

EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — — —)	
ϕ'= 35°	C'= 0 P.S.I.	ϕ = 23°	C= 1 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-11	14/1	46'	CL/SC	5.0	2.42	118.3	15.1	5 RING CONVERSE
C-12	14/1	52'	CL	5.0	2.42	95.6	30.5	5 RING CONVERSE
C-14	14/1	62'	SC/CL	5.0	2.42	112.0	18.1	5 RING CONVERSE

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-11	15	28.9	5.3	9.7	38.6	TX CUE
2	C-12	20	32.1	7.7	12.3	44.4	TX CUE
3	C-14	30	40.3	14.0	16.0	56.8	TX CUE

TRIAXIAL COMPRESSION TEST

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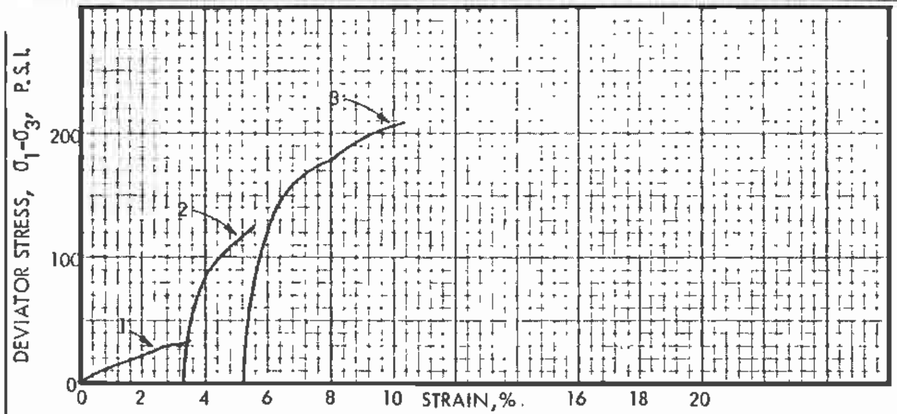
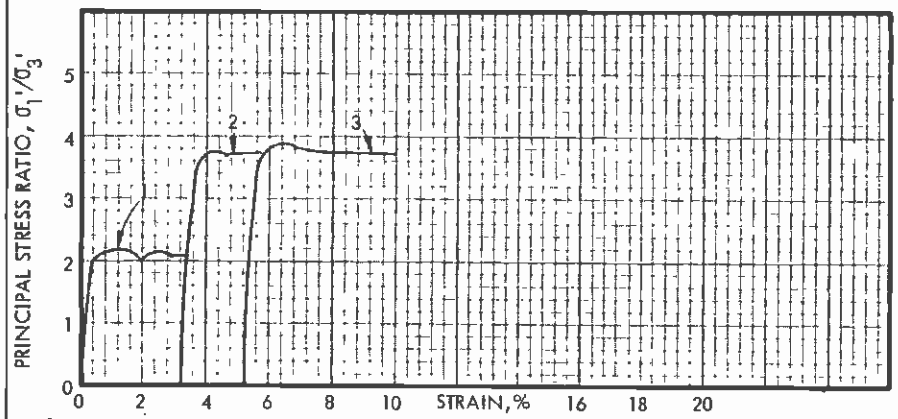
METRO RAIL PROJECT

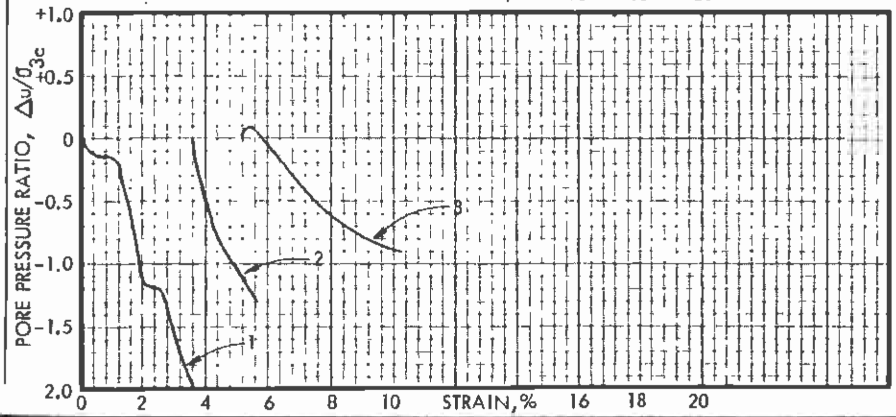
Converse Consultants — Geotechnical Engineering and Applied Sciences

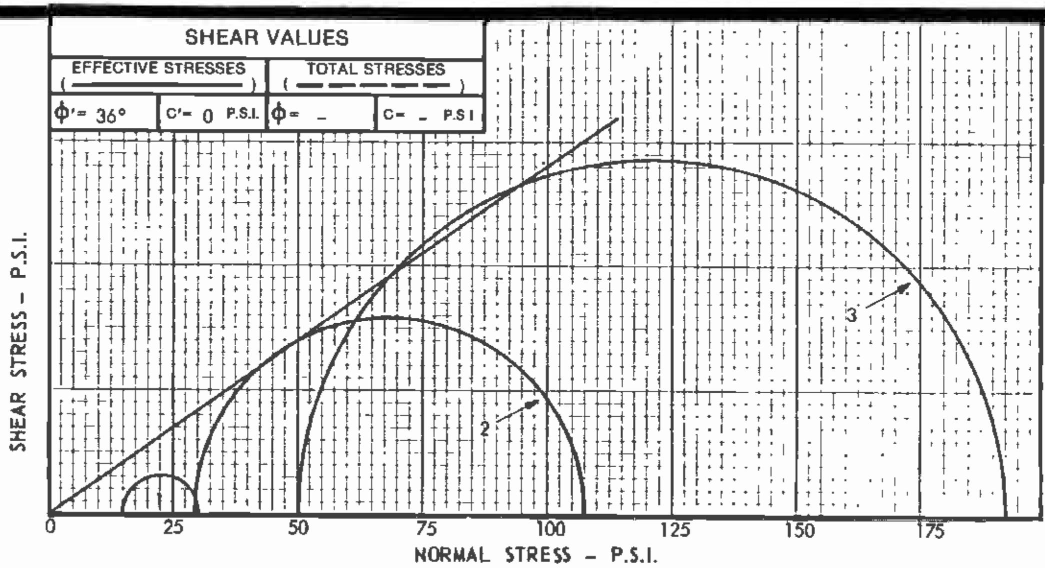
Scale: As Shown
Project No: 83-1140-26
Date:
Prepared by: APT
Figure No:
Checked by: KDM
Approved by: JAD

C-20









SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-16	14/2	79.5'	SP	5.0	2.42	103.5	18.7	5 RING CONVERSE

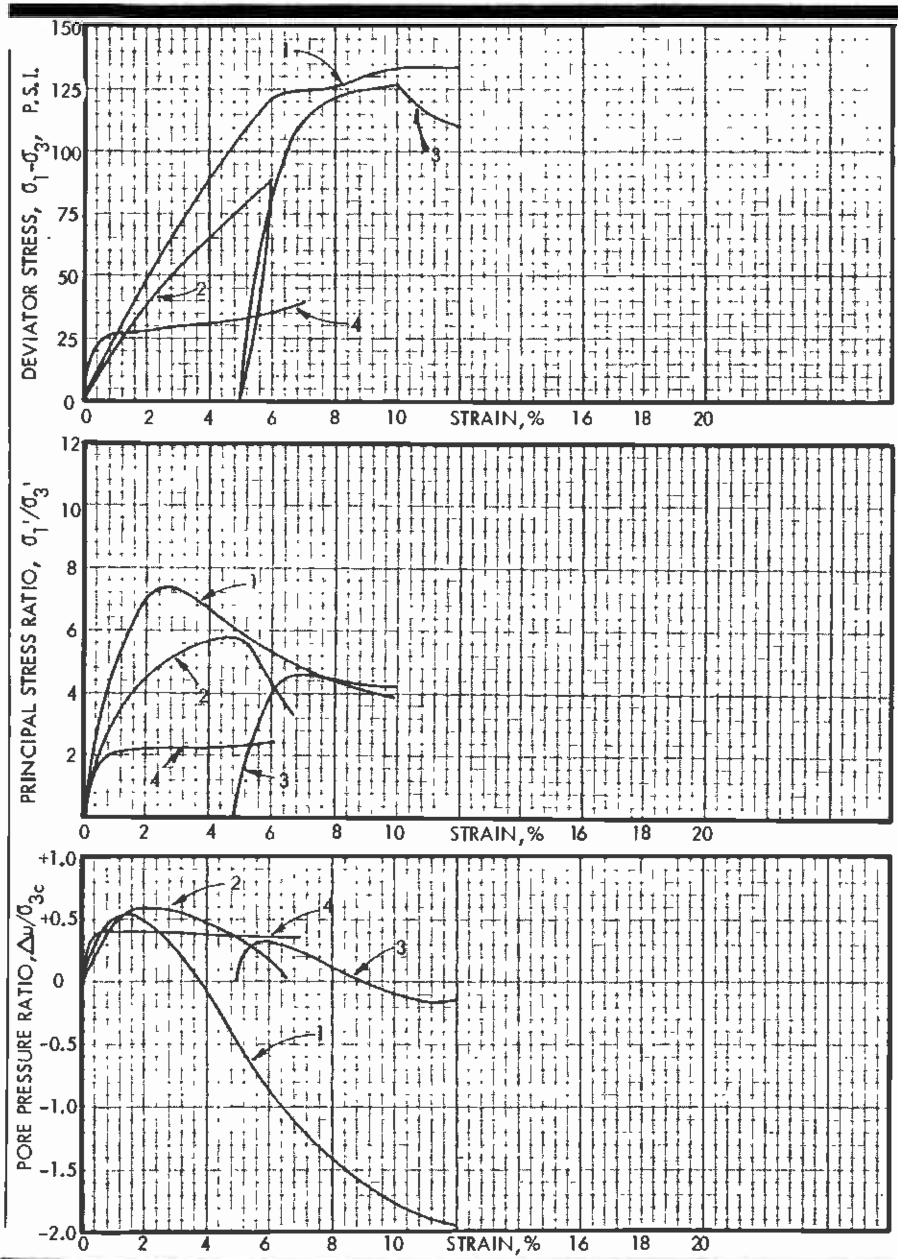
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE Δu (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-16	10	17.5	-4.8	14.8	32.3	TX CUE PROGRESSIVE
2	C-16	20	83.1	-9.8	29.8	112.9	
3	C-16	40	142.7	-10.0	50.0	192.7	

TRIAXIAL COMPRESSION TEST

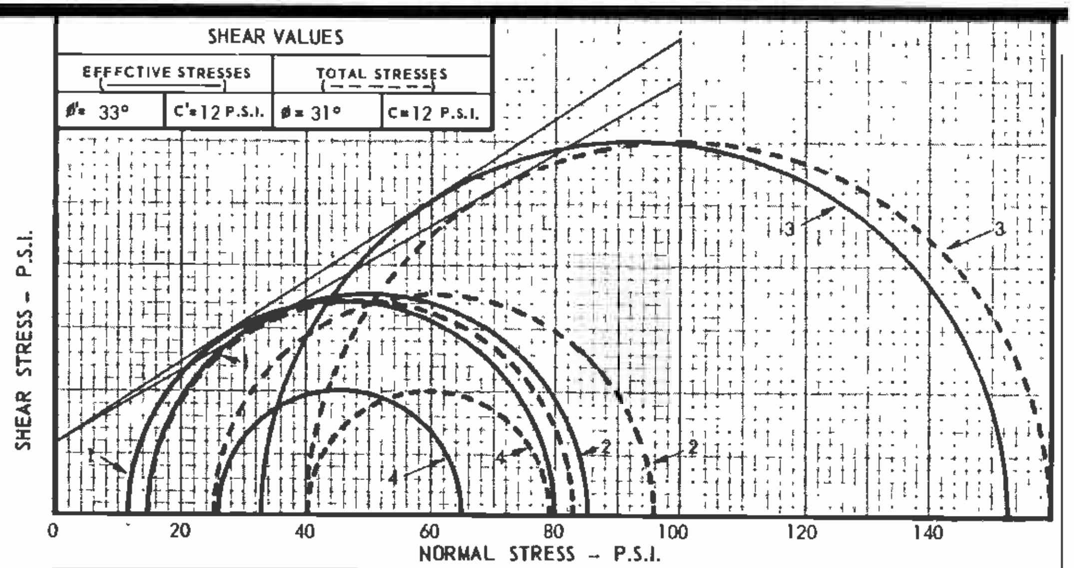
DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale: As Shown
Project No. 83-1140-26
Date:
Prepared by: APT
Figure No. C-21
Checked by: KDM
Approved by: JAD



SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ= 33°	C'=12 P.S.I.	ϕ= 31°	C=12 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-13	14/5	84'4"-84'9"	ML	5.0	2.42	95.7	27.9	5 RING CONVERSE
C-11	14/5	64'6"-64'11"	ML/CL	5.0	2.42	90.4	30.7	5 RING CONVERSE
C-11	14/5	64'6"-64'11"	ML/CL	5.0	2.42	90.4	30.7	5 RING CONVERSE
C-8	14/5	44'3"-44'9"	ML/CL	5.0	2.42	86.3	32.6	5 RING CONVERSE

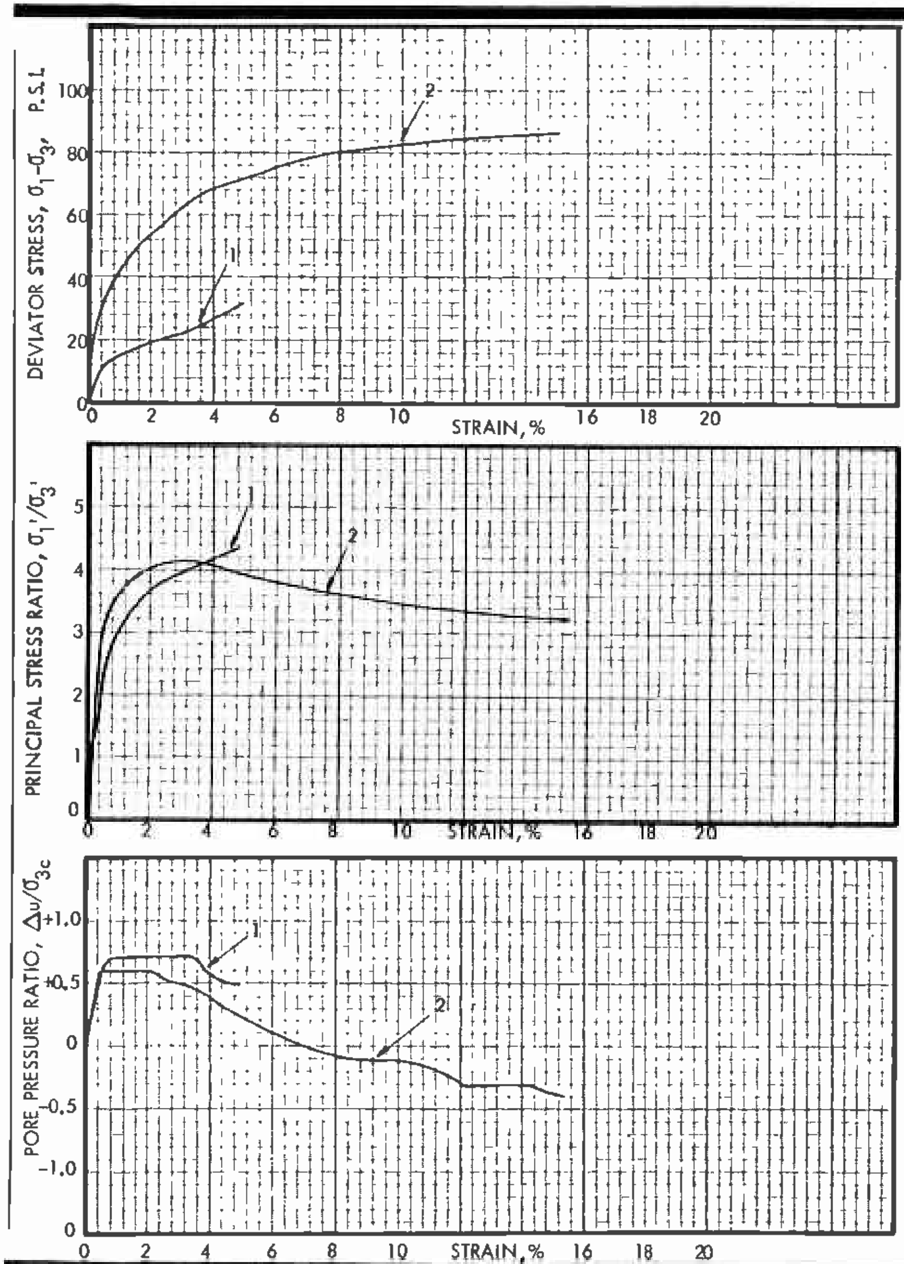
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-13	15	68.4	4.1	10.9	79.3	SINGLE STAGE CUE
2	C-11	25	70.5	10.4	14.6	85.1	PROGRESSIVE CUE
3	C-11	40	119.0	7.0	33.0	152.1	PROGRESSIVE CUE
4	C-8	40	39.0	14.0	26.0	65.0	SINGLE STAGE CUE

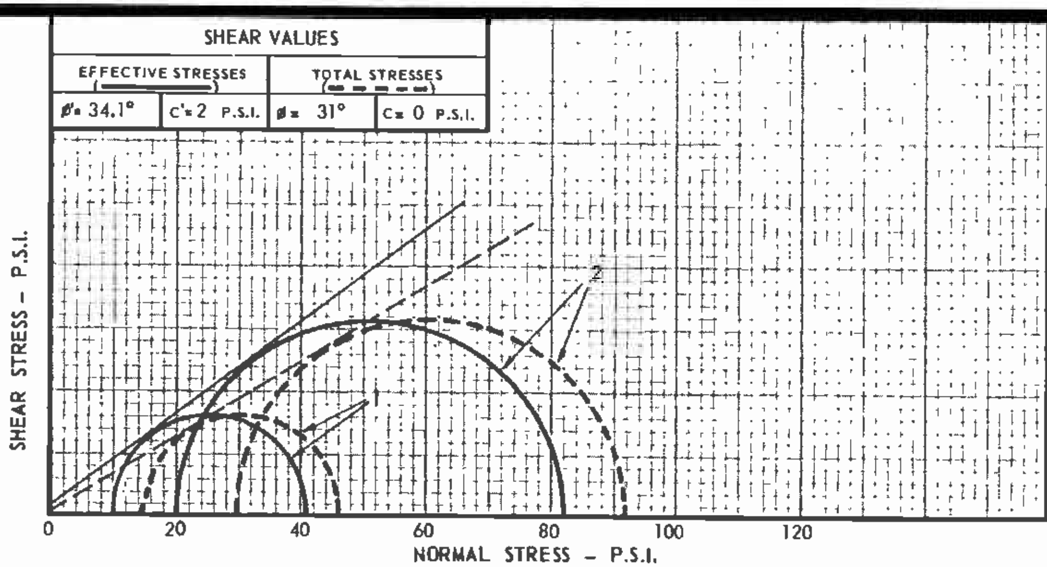
TRIAXIAL COMPRESSION TEST

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale	As Shown	Project No	83-1140-26
Date		Figure No	C-22
Prepared by	APT		
Checked by	KDM		
Approved by	JAD		





SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-10	B-15/1	51.5'-52.0	CL	5.0	2.42	111.6	17.2	5 RING CONVERSE

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE Δu (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-10	15	32.4	5.3	9.7	42.1	TX CUE PROGRESSIVE
2	C-10	30	62.1	10.0	20.0	82.1	

TRIAXIAL COMPRESSION TEST

DESIGN UNIT A220
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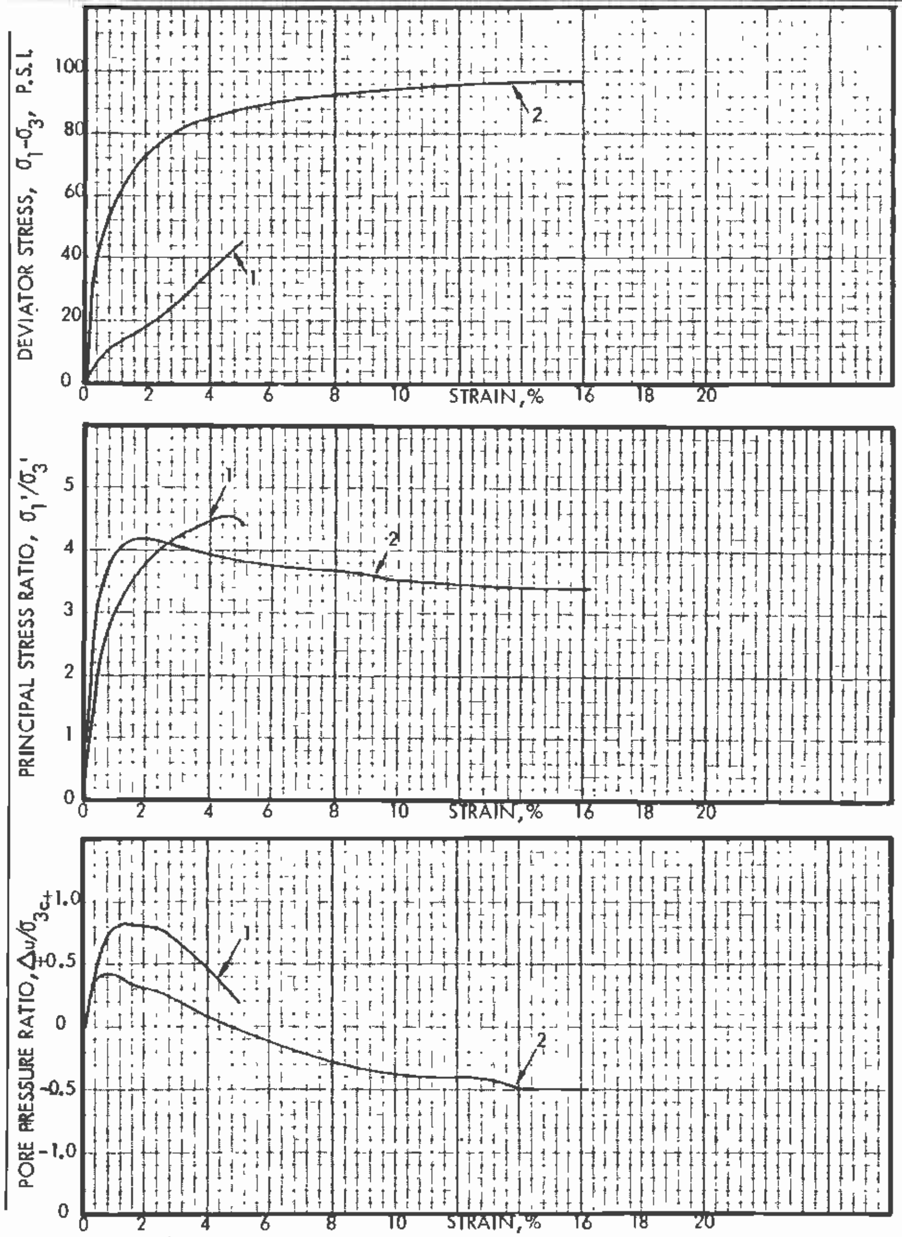
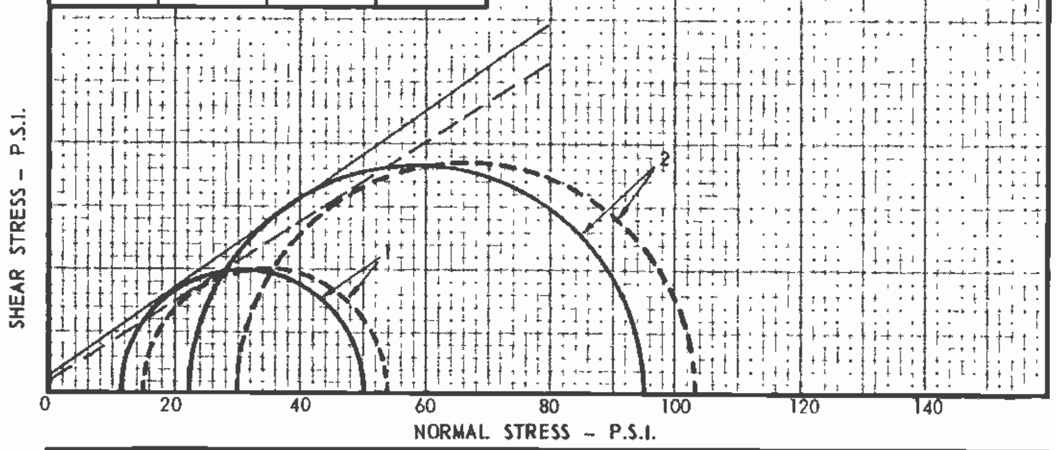
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale: As Shown
Date:
Prepared by: APT
Checked by: KDM
Approved by: JAD
Project No: 83-1140-26
Figure No: C-23



SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 34.5°	c'= 4 P.S.I.	ϕ = 32°	c = 2 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-11	B - 15/2	57.5'-58'	CL	5.0	2.42	111.6	16.0	5 RING CONVERSE

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-11	15	38.9	3.8	11.2	50.1	TX CUE PROGRESSIVE
2	C-11	30	72.7	7.1	22.9	95.6	

TRIAXIAL COMPRESSION TEST

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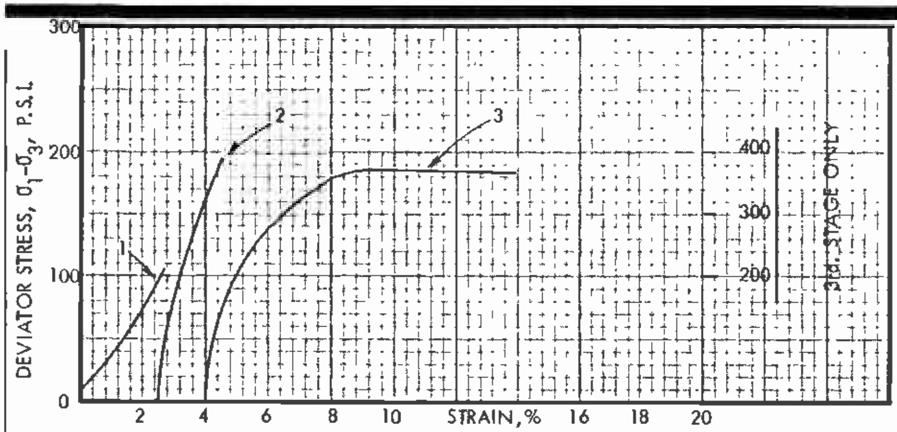
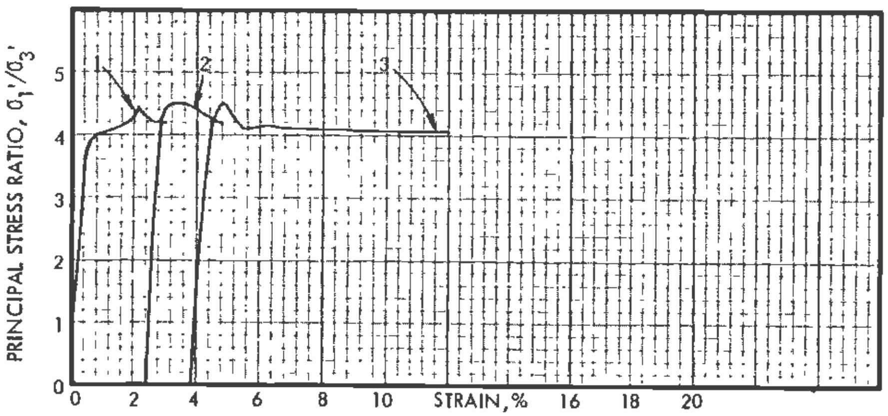
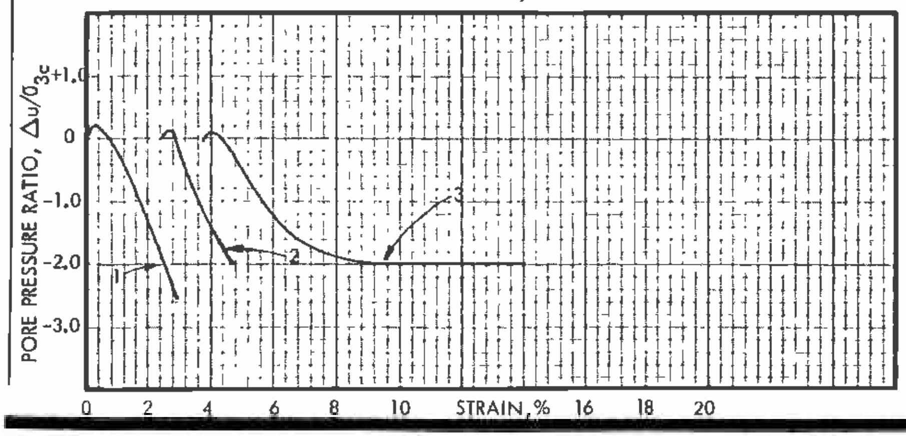
Converse Consultants — Geotechnical Engineering and Applied Sciences

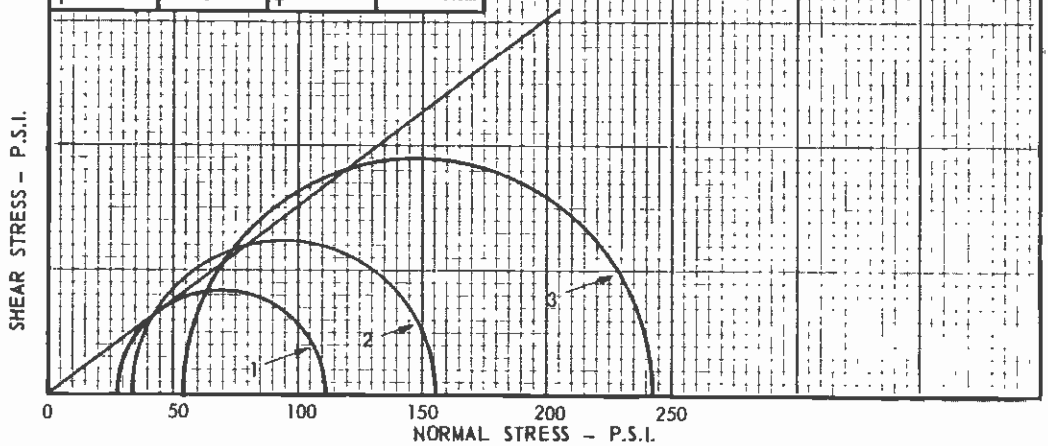
Scale: As Shown	Project No.: 83-1140-26
Date:	Figure No: C-24
Prepared by: APT	
Checked by: KDM	
Approved by: JAD	







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



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-15	15/3	76'4"-76'9"	SP	5.0	2.42	122.3	4.0	5 RING CONVERSE

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE Δu (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-15	10	86.5	-14.8	24.8	111.3	PROGRESIVE CUE
2	C-15	20	122.0	-14.3	34.3	156.3	
3	C-15	40	188.5	-14.0	54.0	242.5	

TRIAXIAL COMPRESSION TEST

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Southern California Rapid Transit District
METRO RAIL PROJECT

Scale: As Shown
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Approved by: JAD
Project No. 83-1140-26
Figure No. C-25

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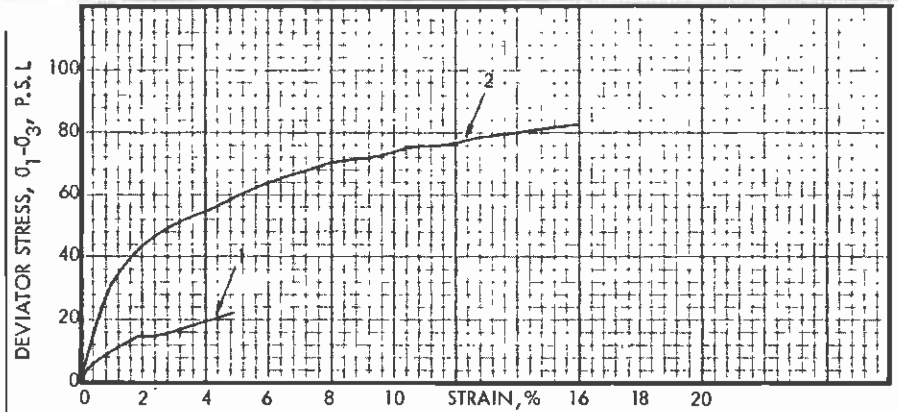
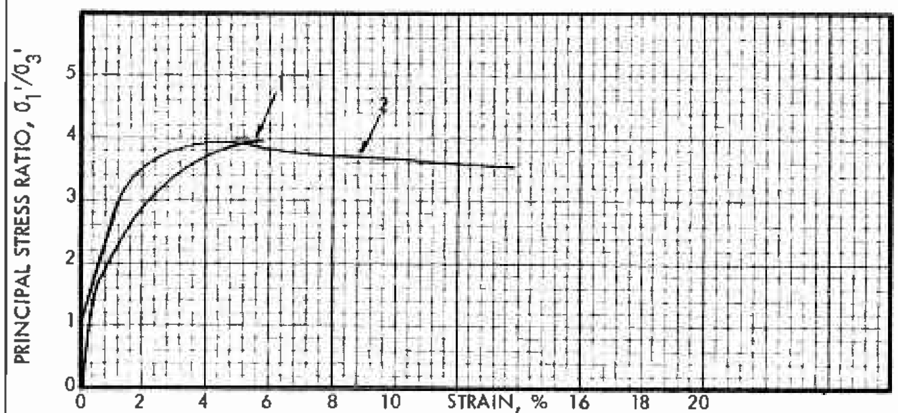
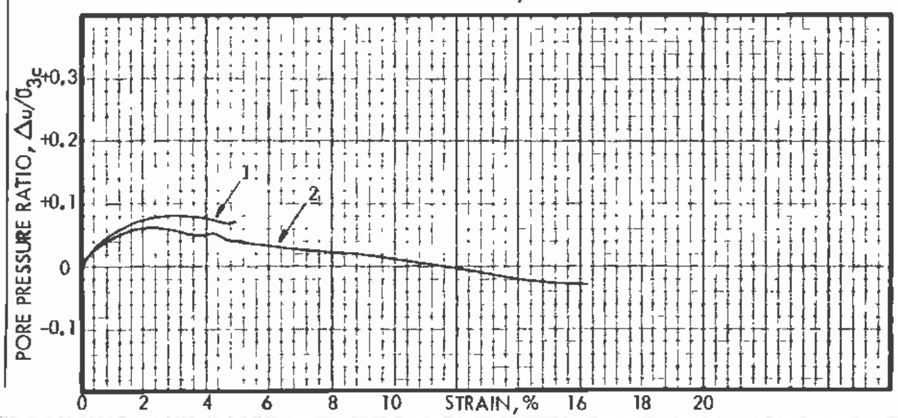
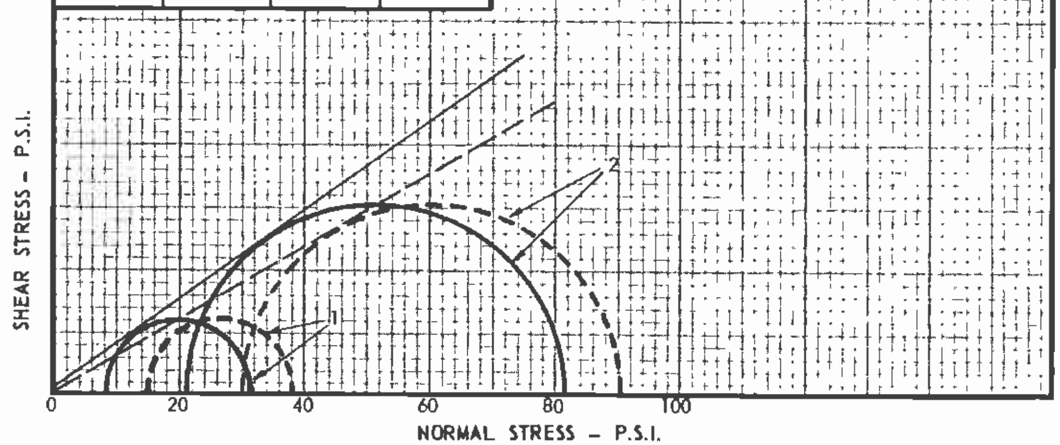






SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 34.5°	c'= 2 P.S.I.	ϕ = 30°	c = 0 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-6	B- 15/4	27.5'-28.0'	CL	5.0	2.42	106.6	18.4	5 RING CONVERSE

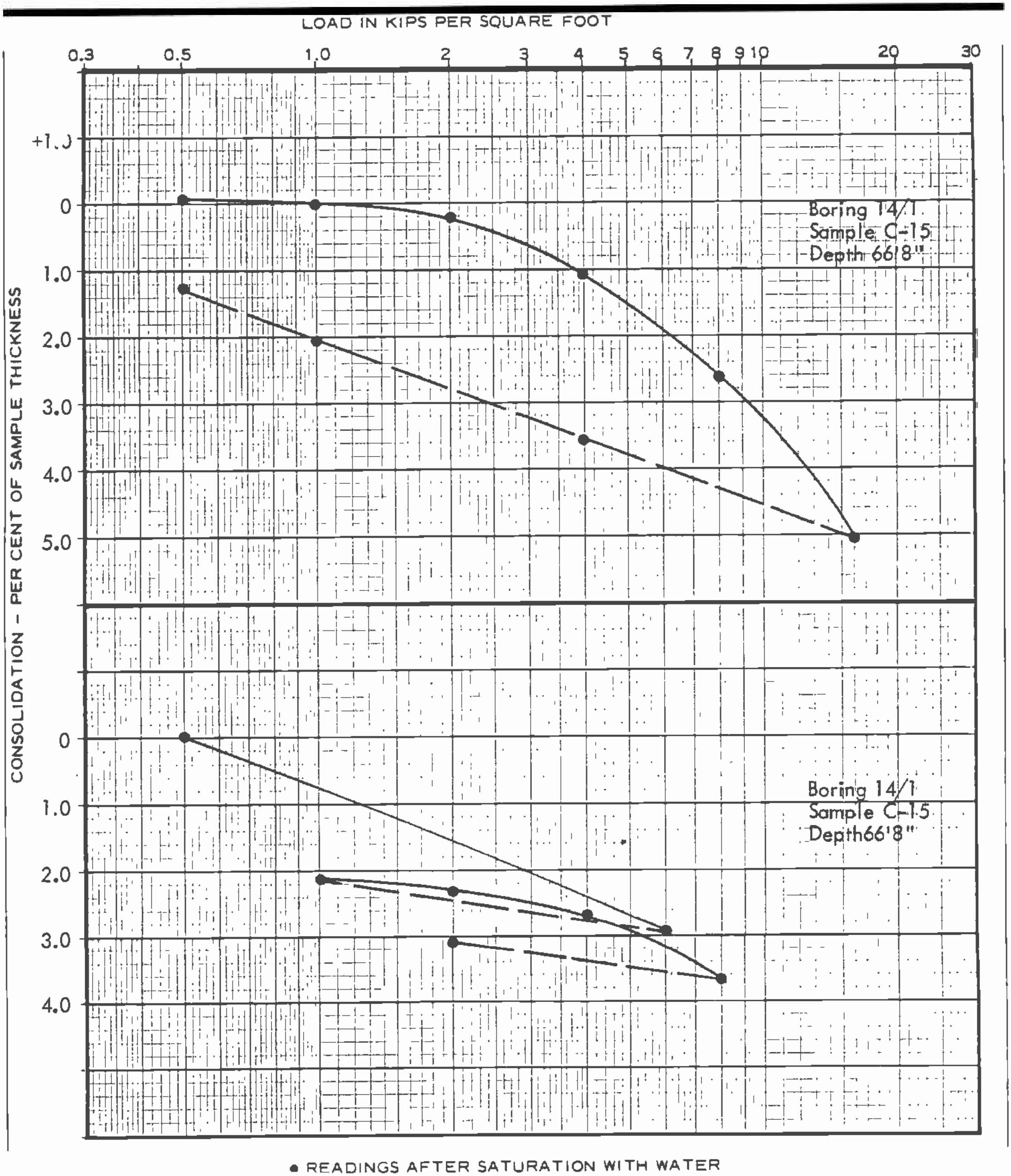
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TYPE OF TEST
			MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	C-6	15	23.2	7.1	7.9	31.2	TX CUE PROGRESSIVE
2	C-6	30	60.3	9.2	20.8	81.1	

TRIAXIAL COMPRESSION TEST

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale: As Shown
Date:
Prepared by: APT
Checked by: KDM
Approved by: JAD
Project No. 83-1140-26
Figure No. C-26



CONSOLIDATION TESTS

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Southern California Rapid Transit District
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Project No
83-1140-26

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Figure No.
C-27

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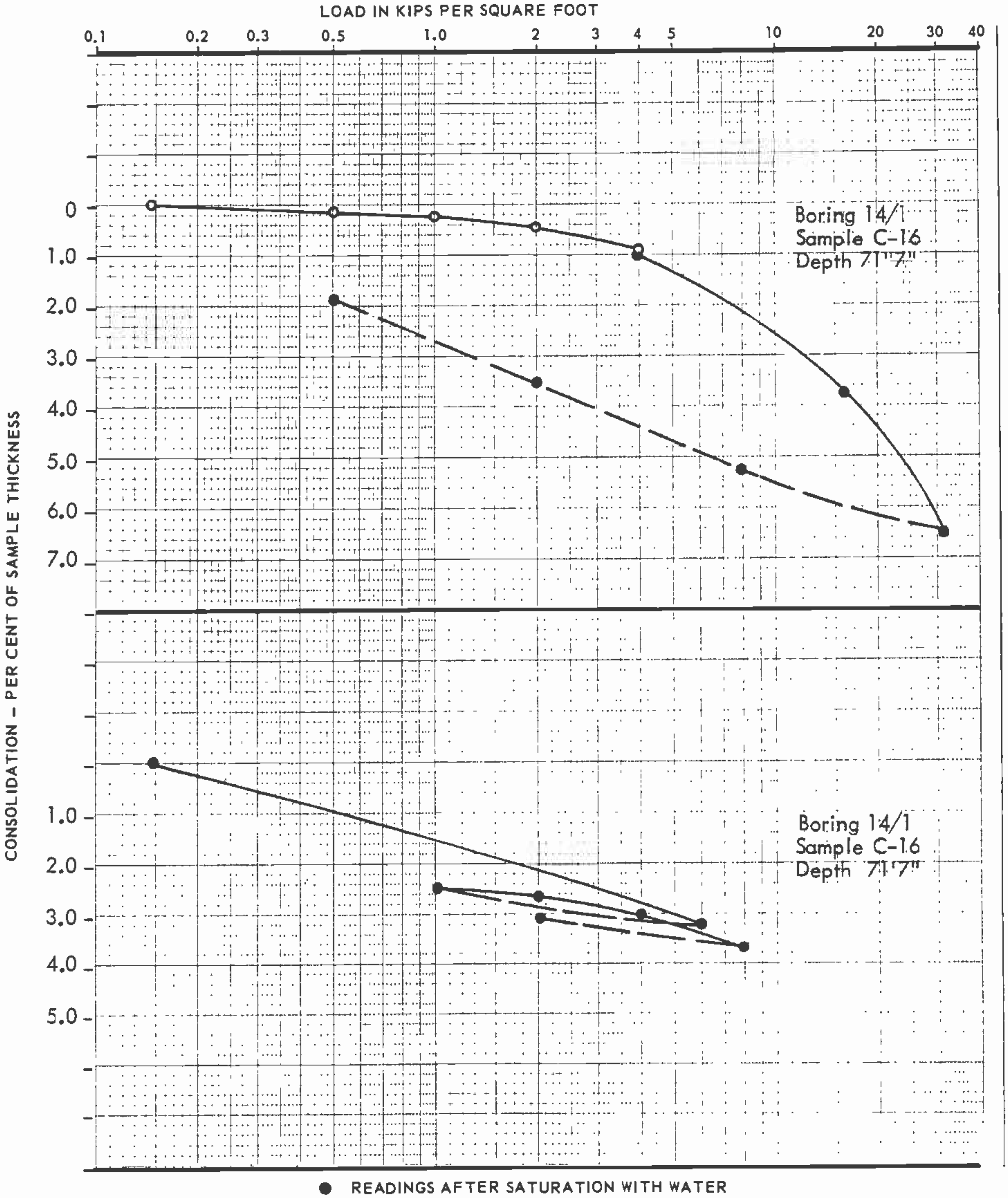

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Figure No.
C-28

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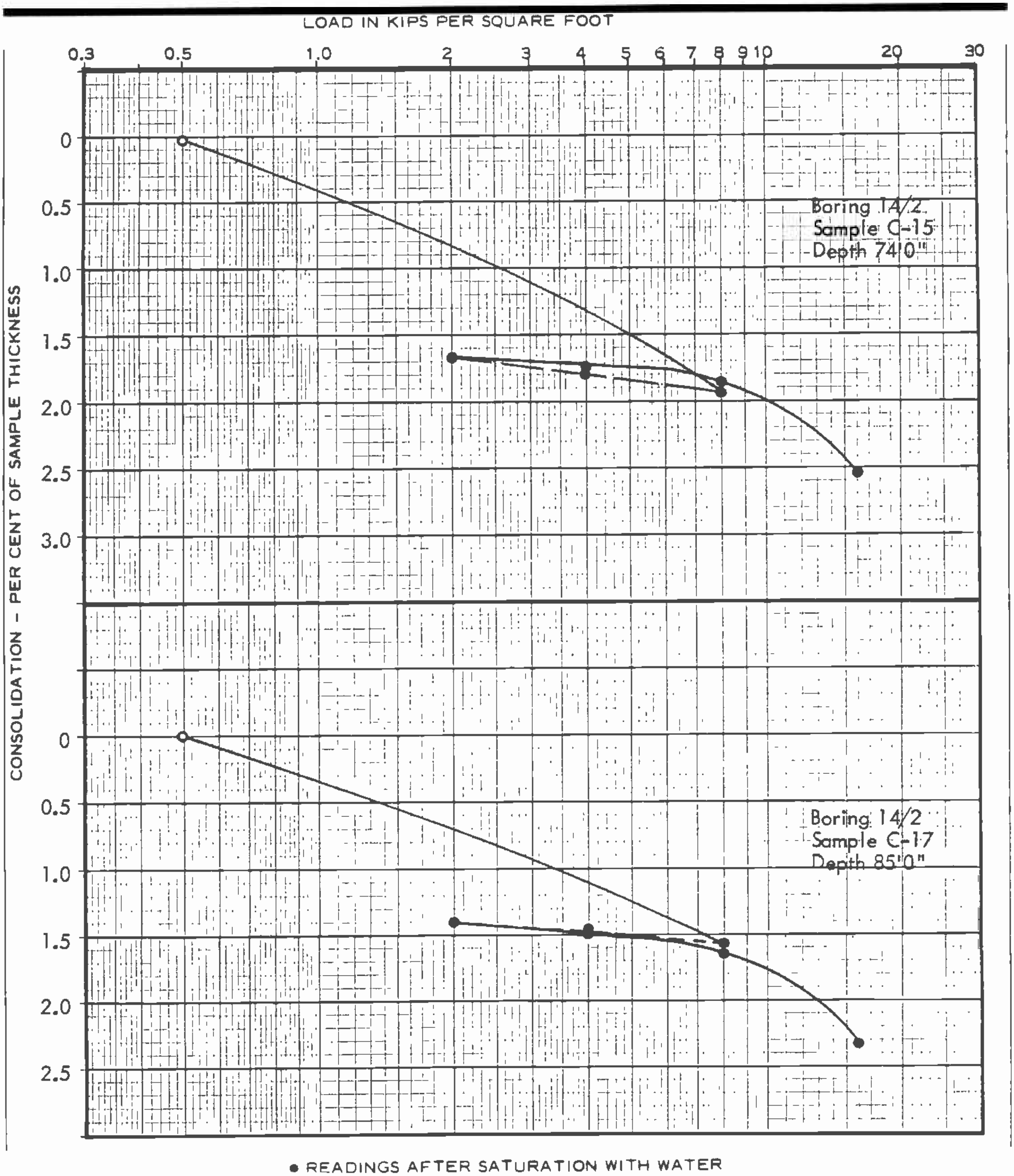

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METRO RAIL PROJECT

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Figure No. C-29

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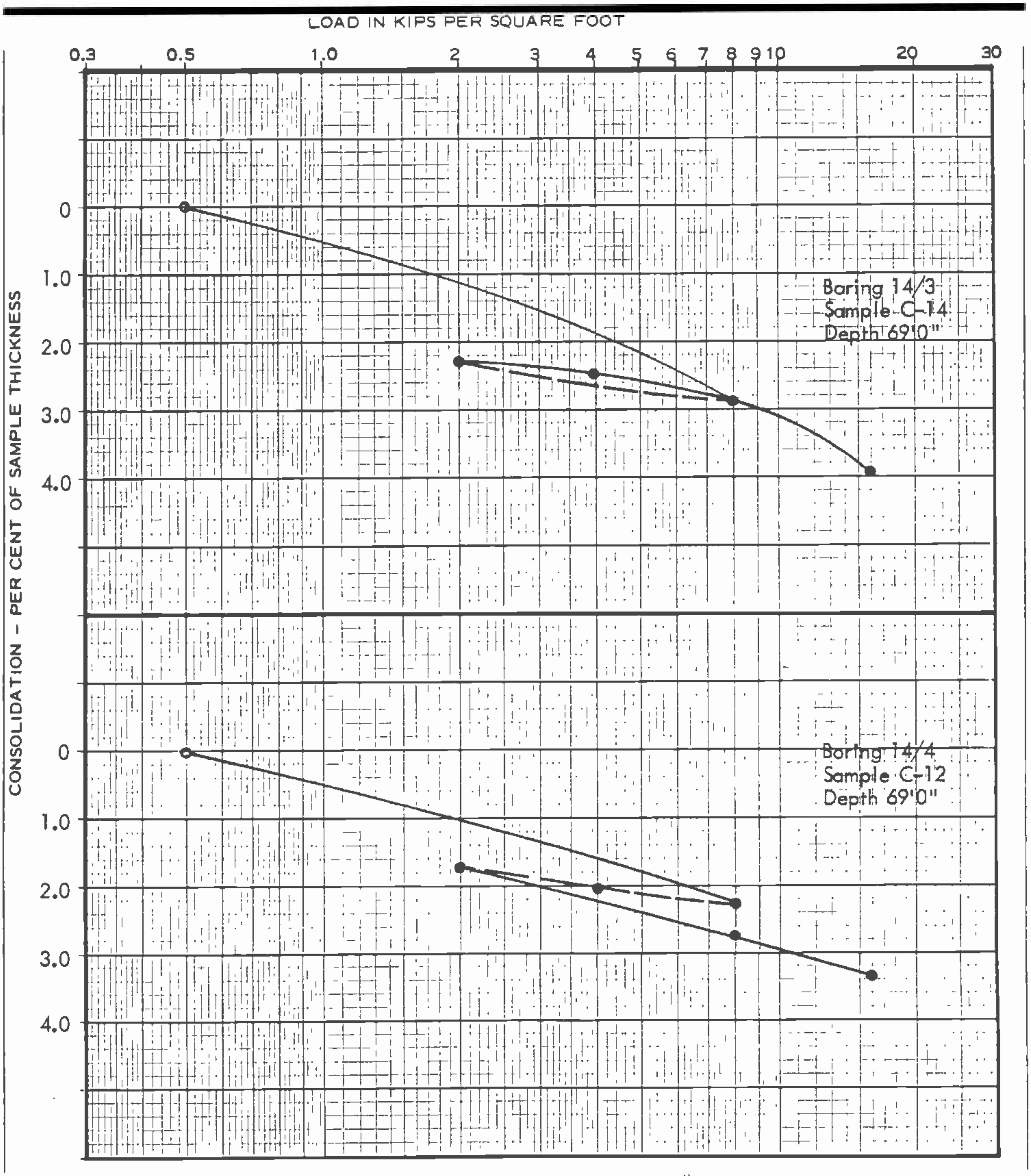

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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83-1140-26

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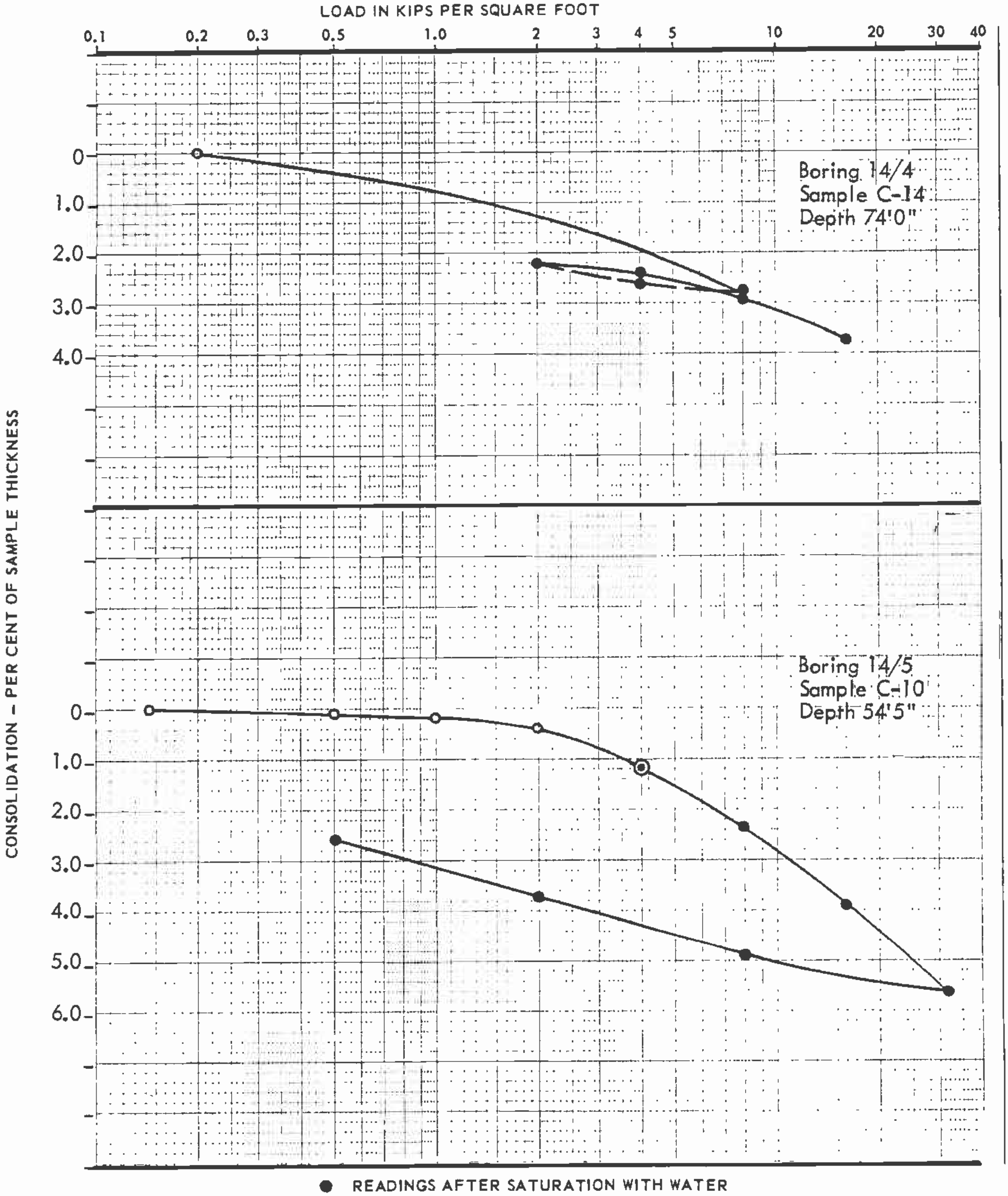
Figure No.
C-30

Approved for publication ______ by ______



CONSOLIDATION TESTS

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Figure No.
C-31

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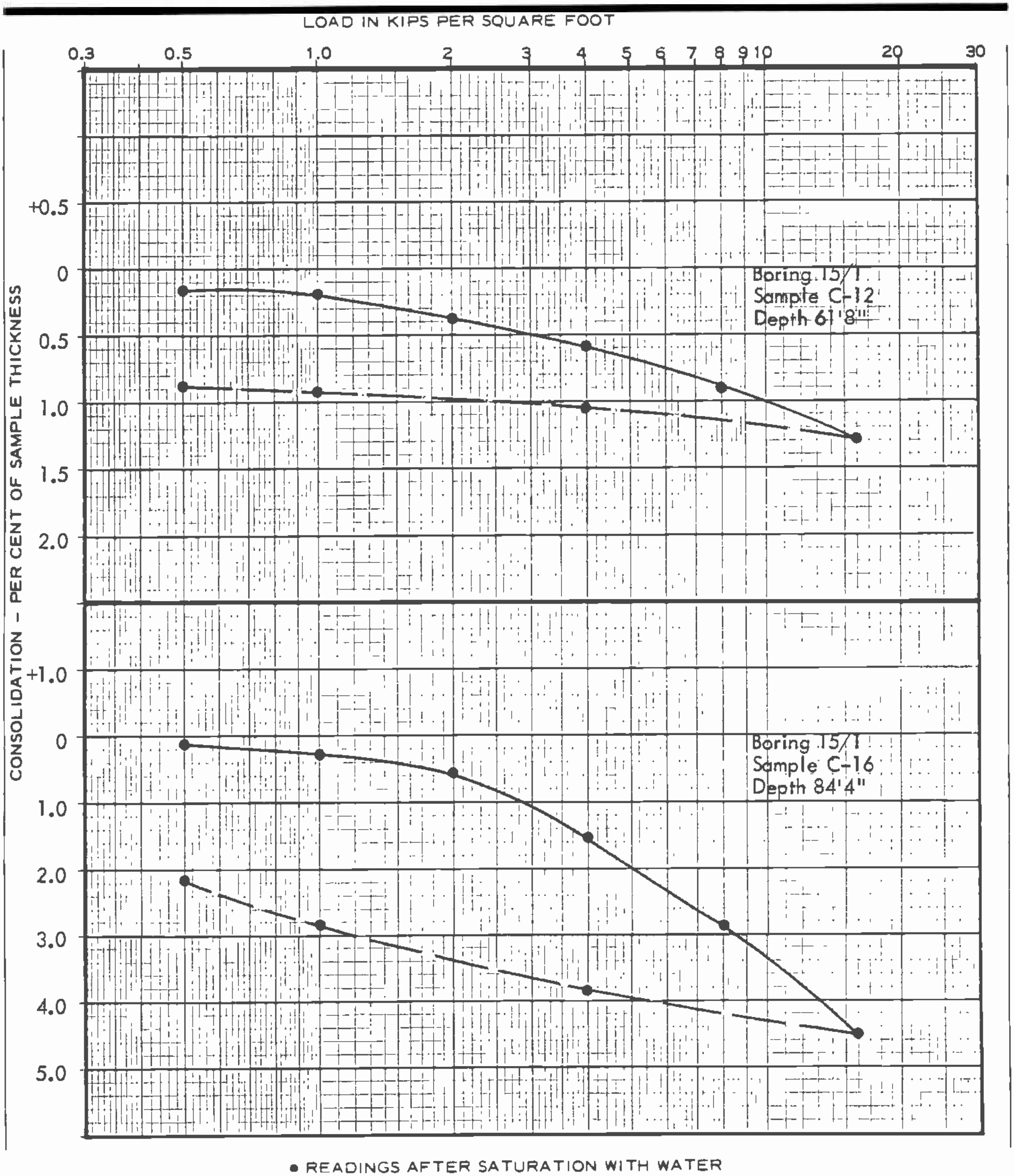

CONSOLIDATION TESTS

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METRO RAIL PROJECT

Project No
83-1140-26

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Figure No.
C-32

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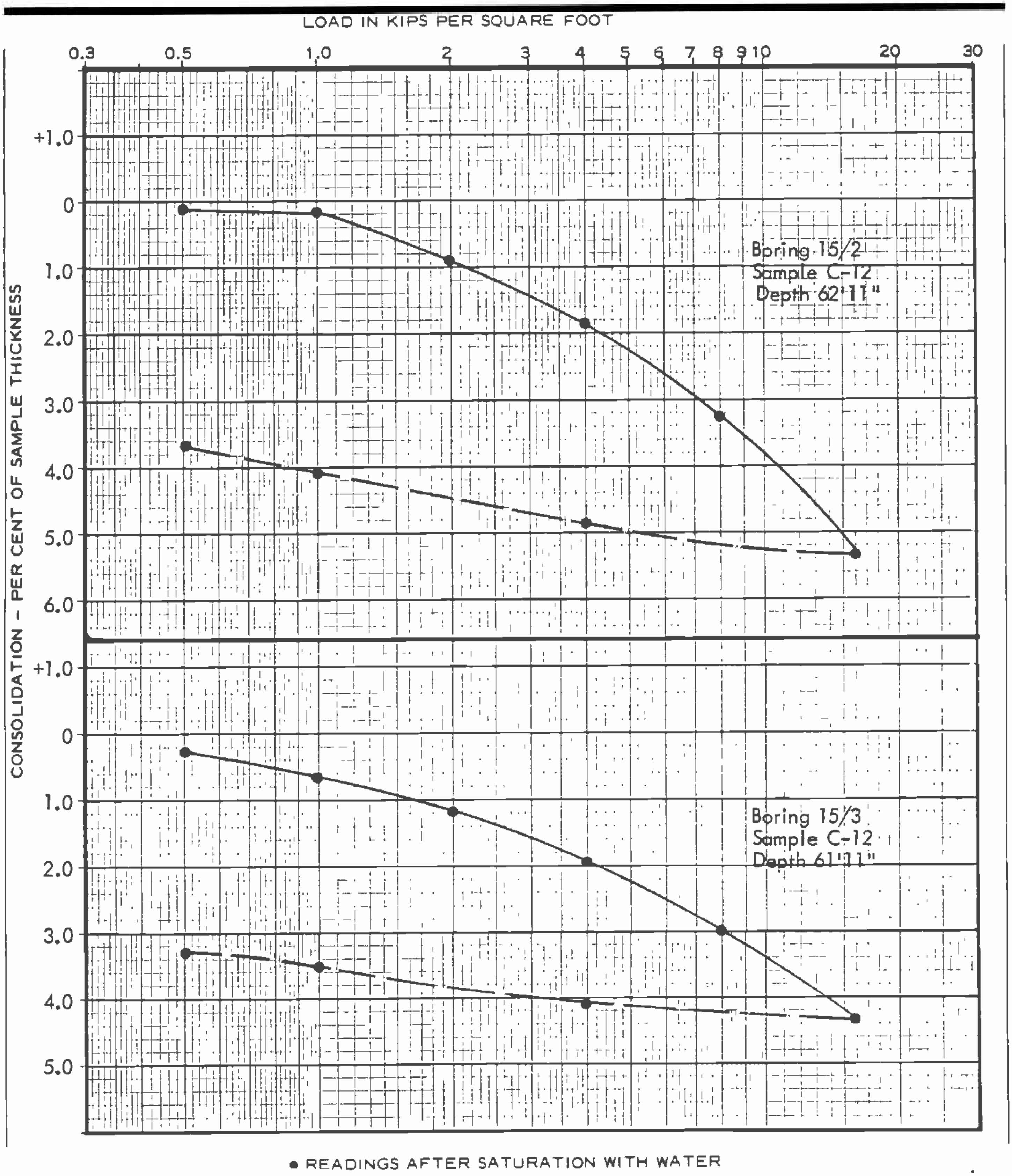

CONSOLIDATION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

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Figure No C-33

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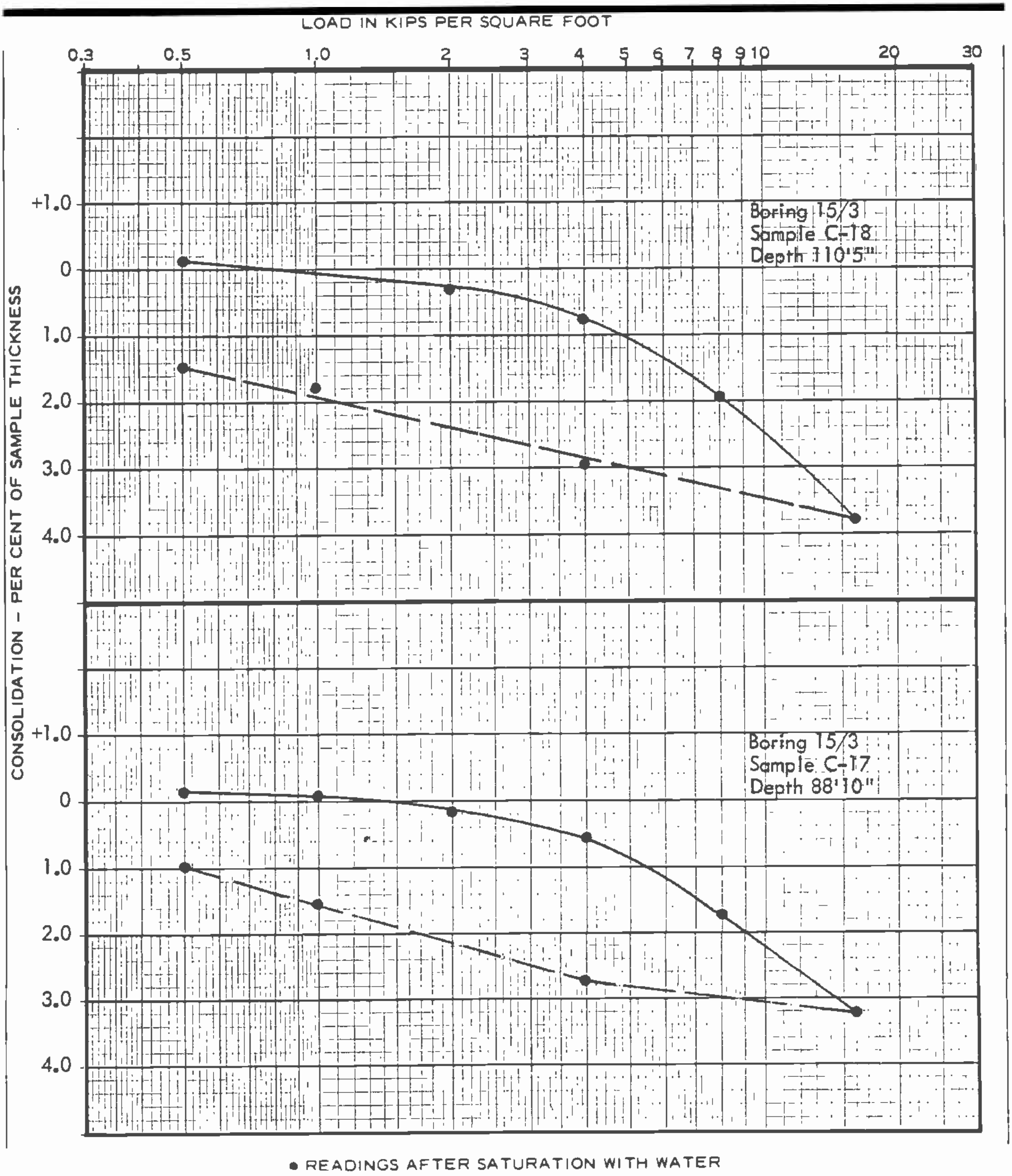

CONSOLIDATION TESTS

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Project No
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Figure No.
C-34

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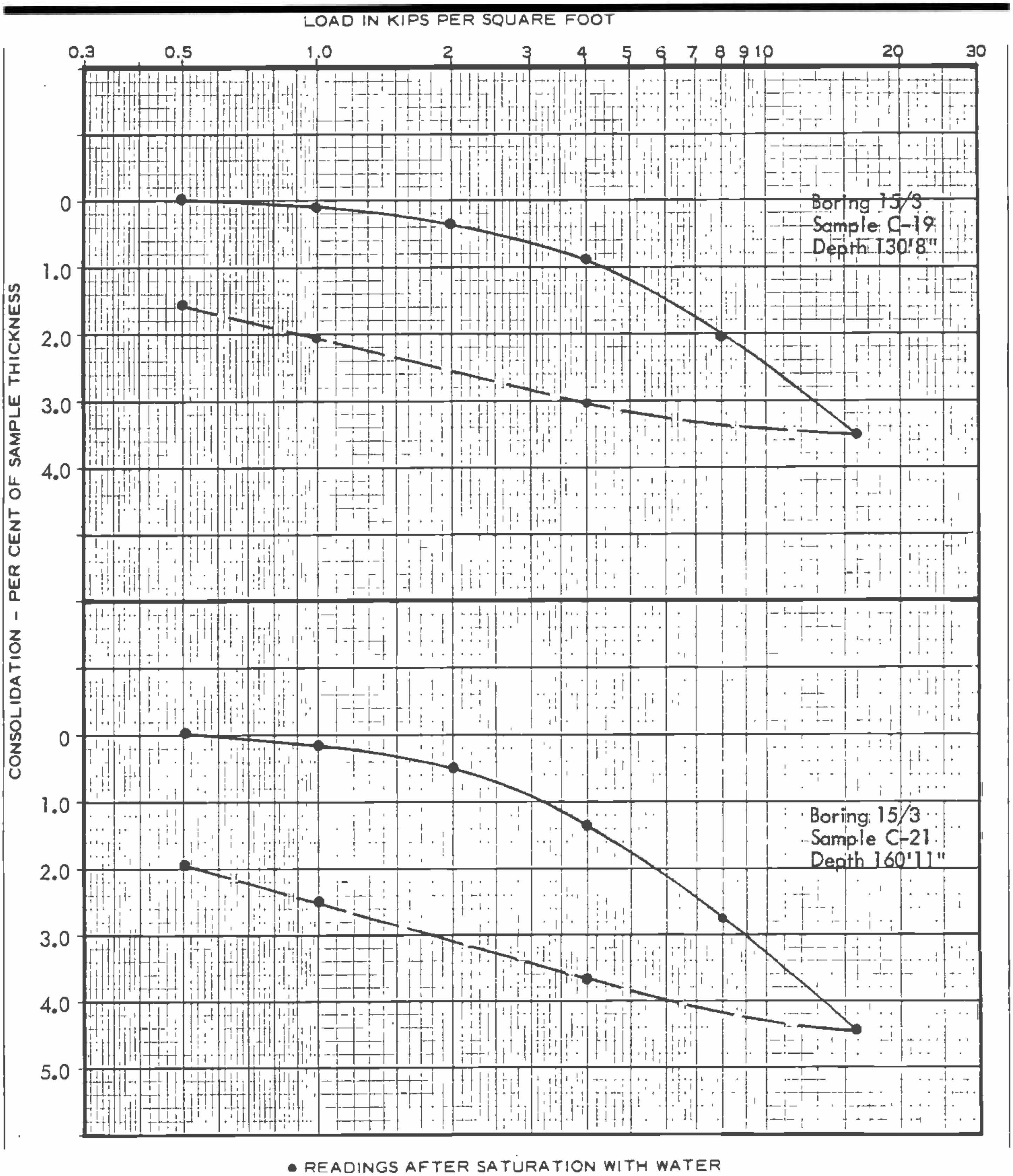

CONSOLIDATION TESTS

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Project No
83-1140-26

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Figure No
C-35

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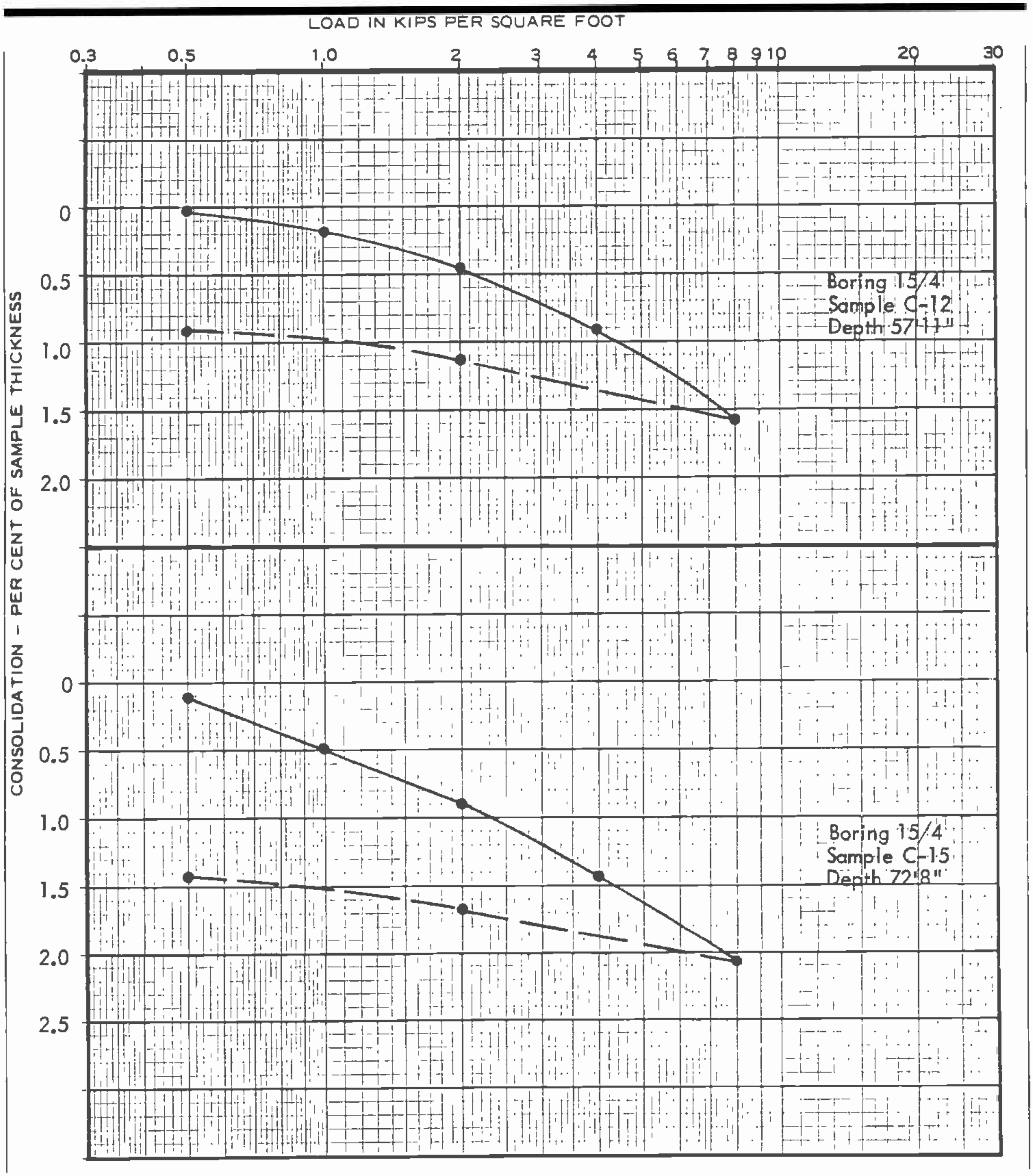

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140-26

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Geotechnical Engineering and Applied Sciences

Figure No.
C-36

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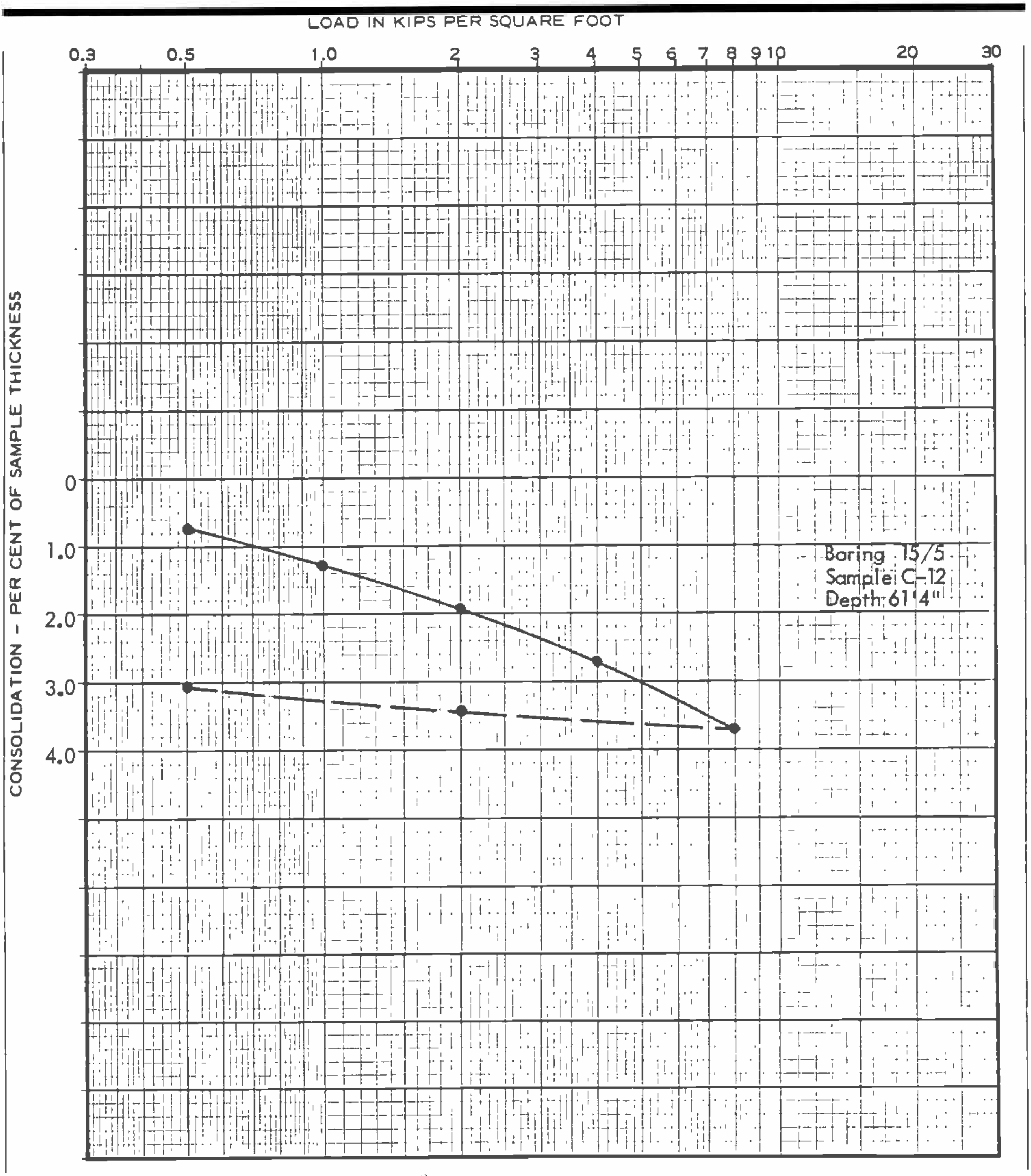

• READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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METRO RAIL PROJECT

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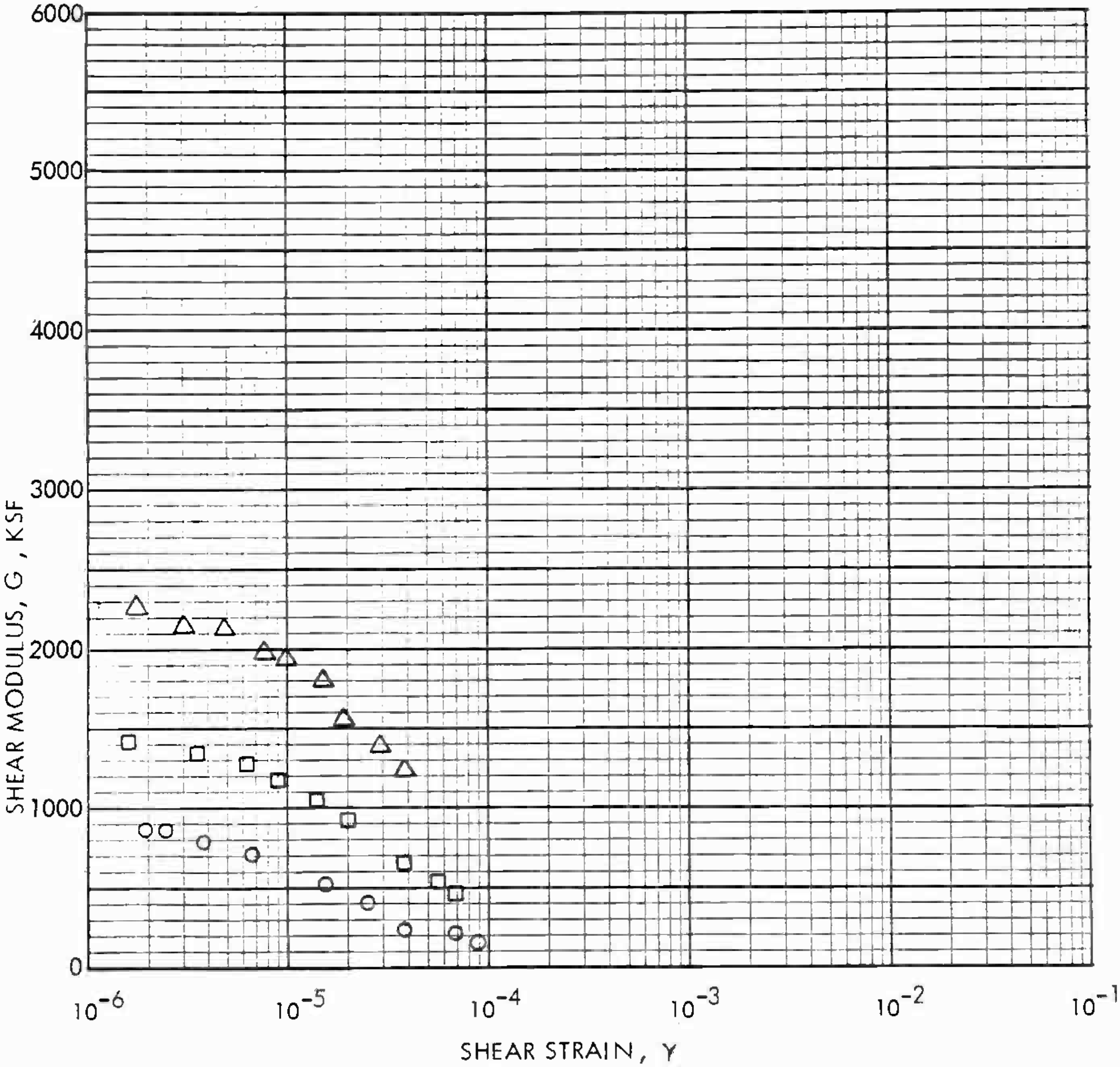
83-1140-26

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-37

Approved for publication ______ by ______

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
14	S-1	45½	96	29	15	○
					30	□
					50	△

Sample Description: Gray Claystone with trace fine sand; moist

RESONANT COLUMN TEST

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Southern California Rapid Transit District
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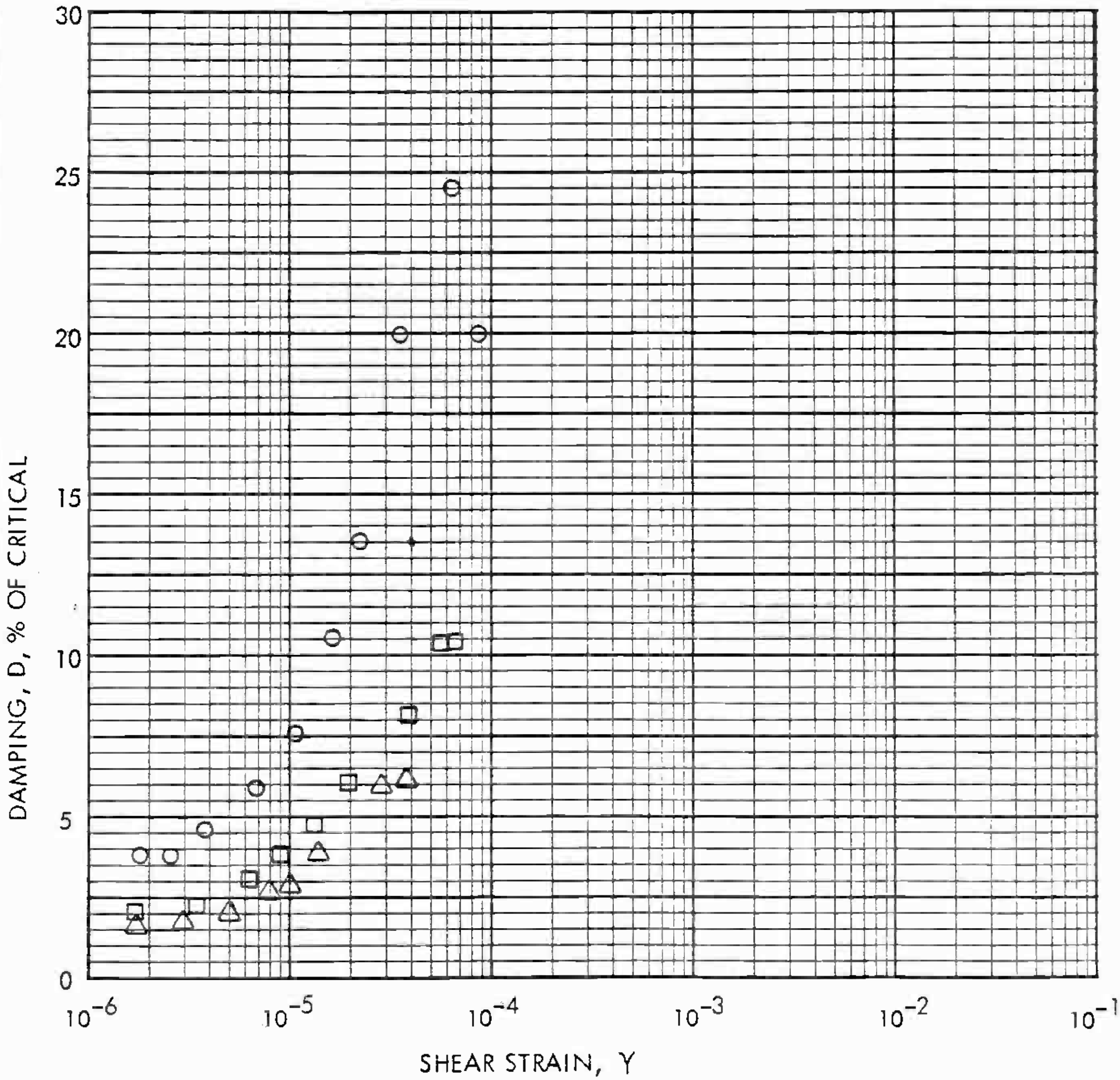
Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-38

Approved for publication ______ by ______

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\tilde{\sigma}_c$(PSI)	SYMBOL
14	S-1	45½	96	29	15	○
					30	□
					50	△

Sample Description: Gray Claystone with trace fine Sand; moist

RESONANT COLUMN TEST

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

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Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-39

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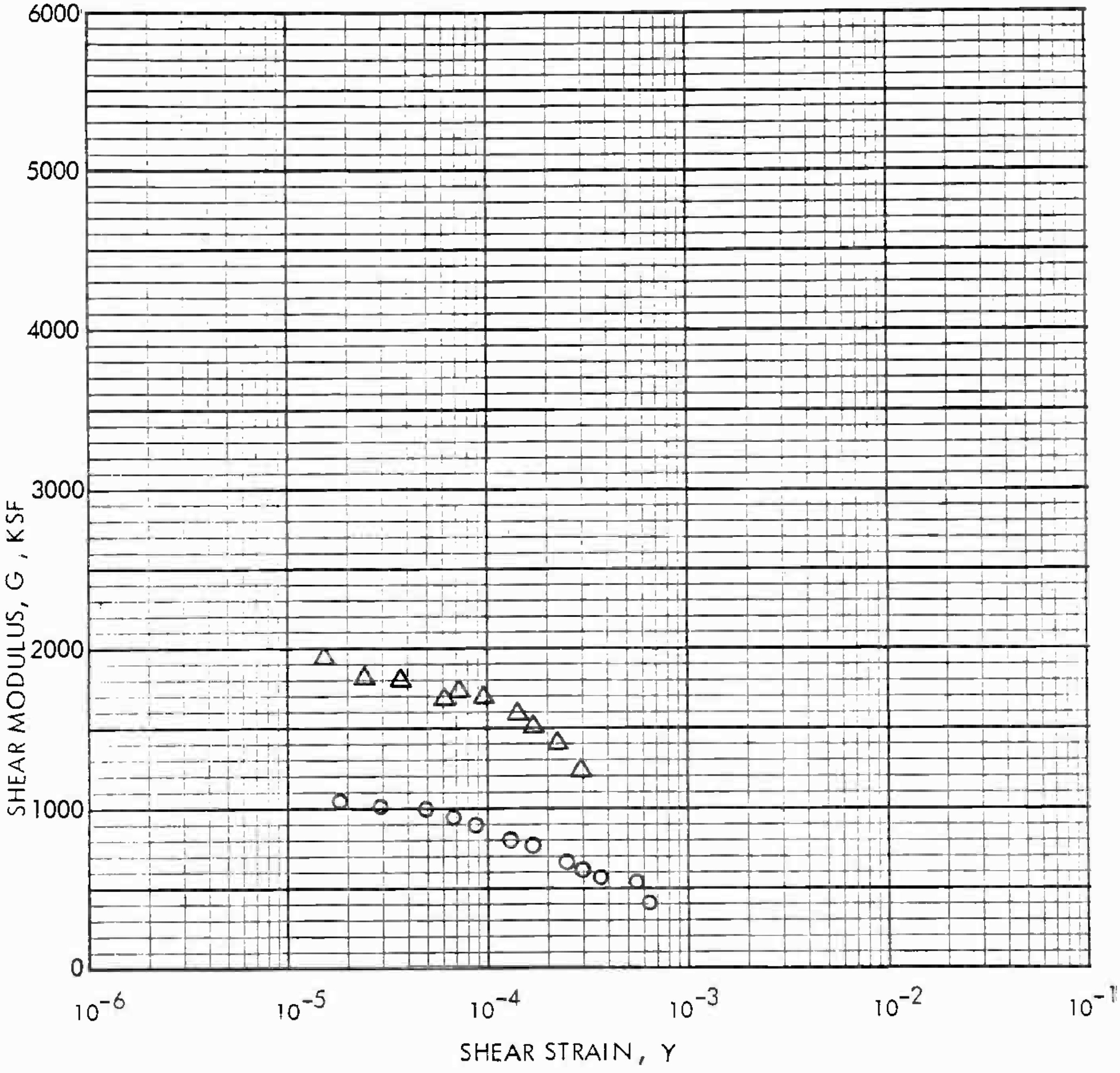
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
14	S-9	167	95	28	15	○
					50	△

Sample Description: Gray Claystone with layers of Siltstone and occasional thin laminations of fine sand; moist

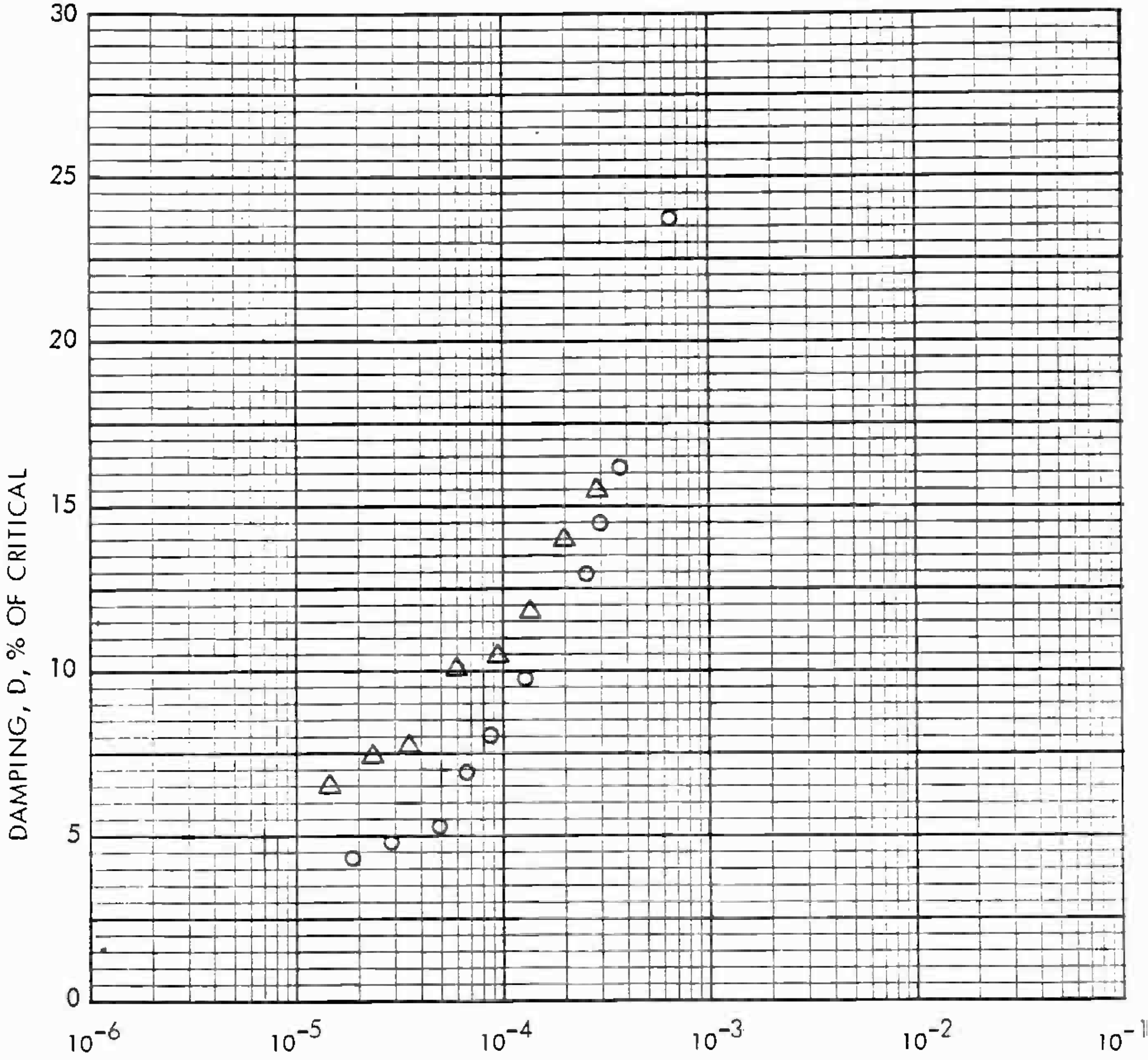
RESONANT COLUMN TEST

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-40



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
14	S-9	167	95	28	15	○
					50	△

Sample Description: Gray Claystone with layers of Siltstone and occasional thin laminations of fine sand; moist

RESONANT COLUMN TEST

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Southern California Rapid Transit District
METRO RAIL PROJECT

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Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. C-41

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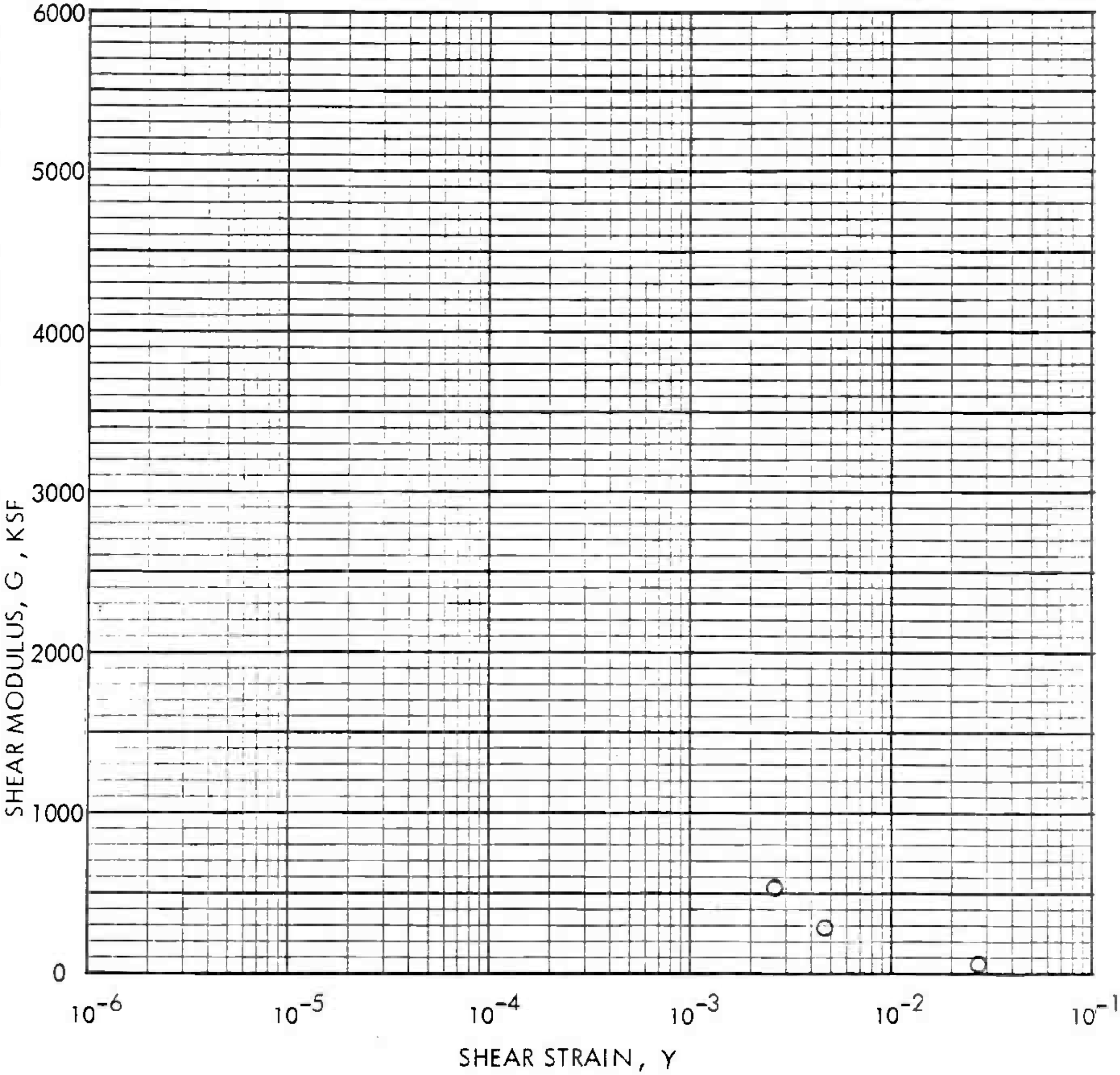
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
14	S-1	45	91	28	30

Sample Description: Gray Silt; very soft

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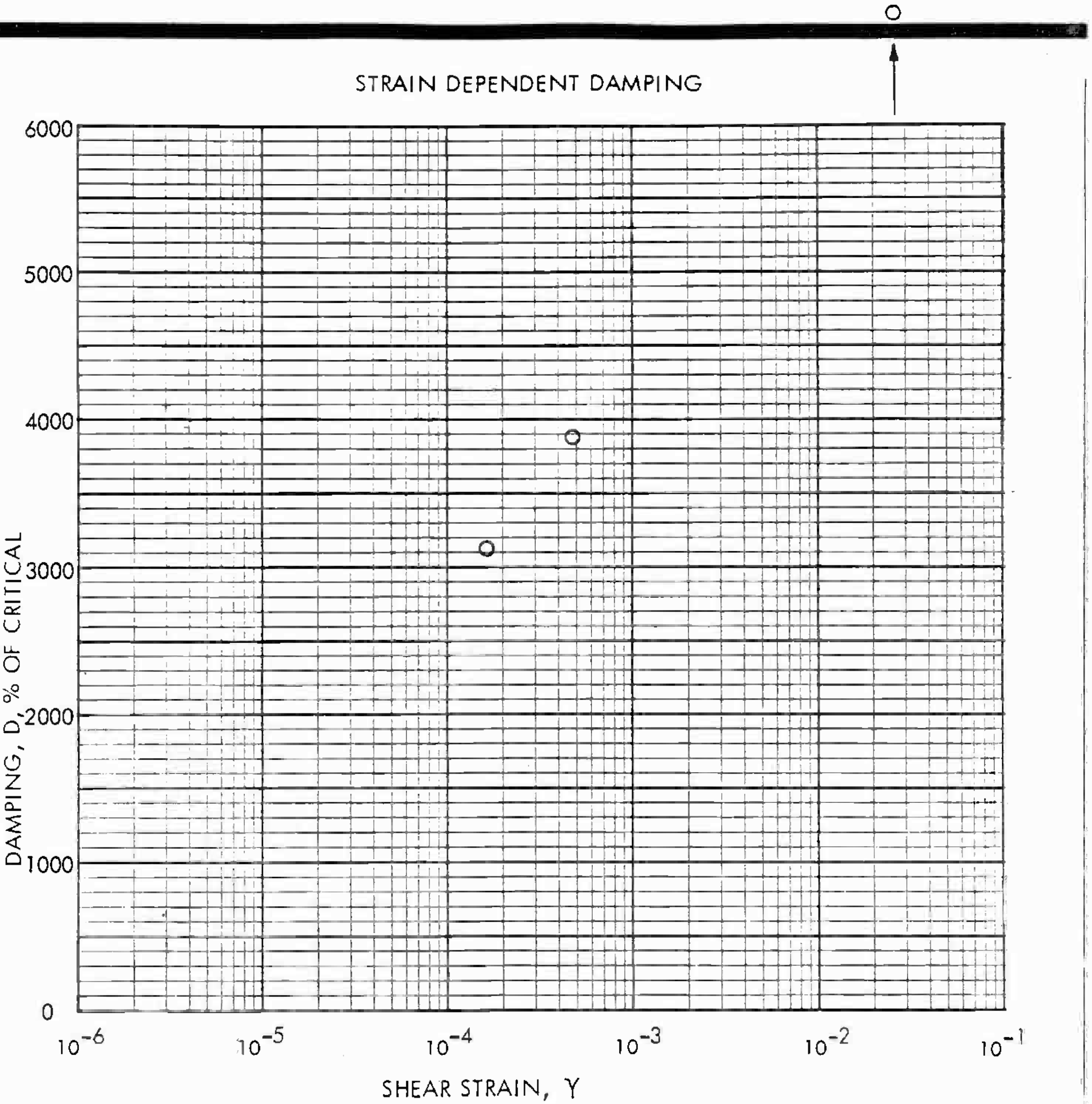
DYNAMIC TRIAXIAL TEST

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
C-42



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
14	S-1	45	91	28	30

Sample Description: Gray Silt; very soft

DYNAMIC TRIAXIAL TEST

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Project No.
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Figure No.
C-43

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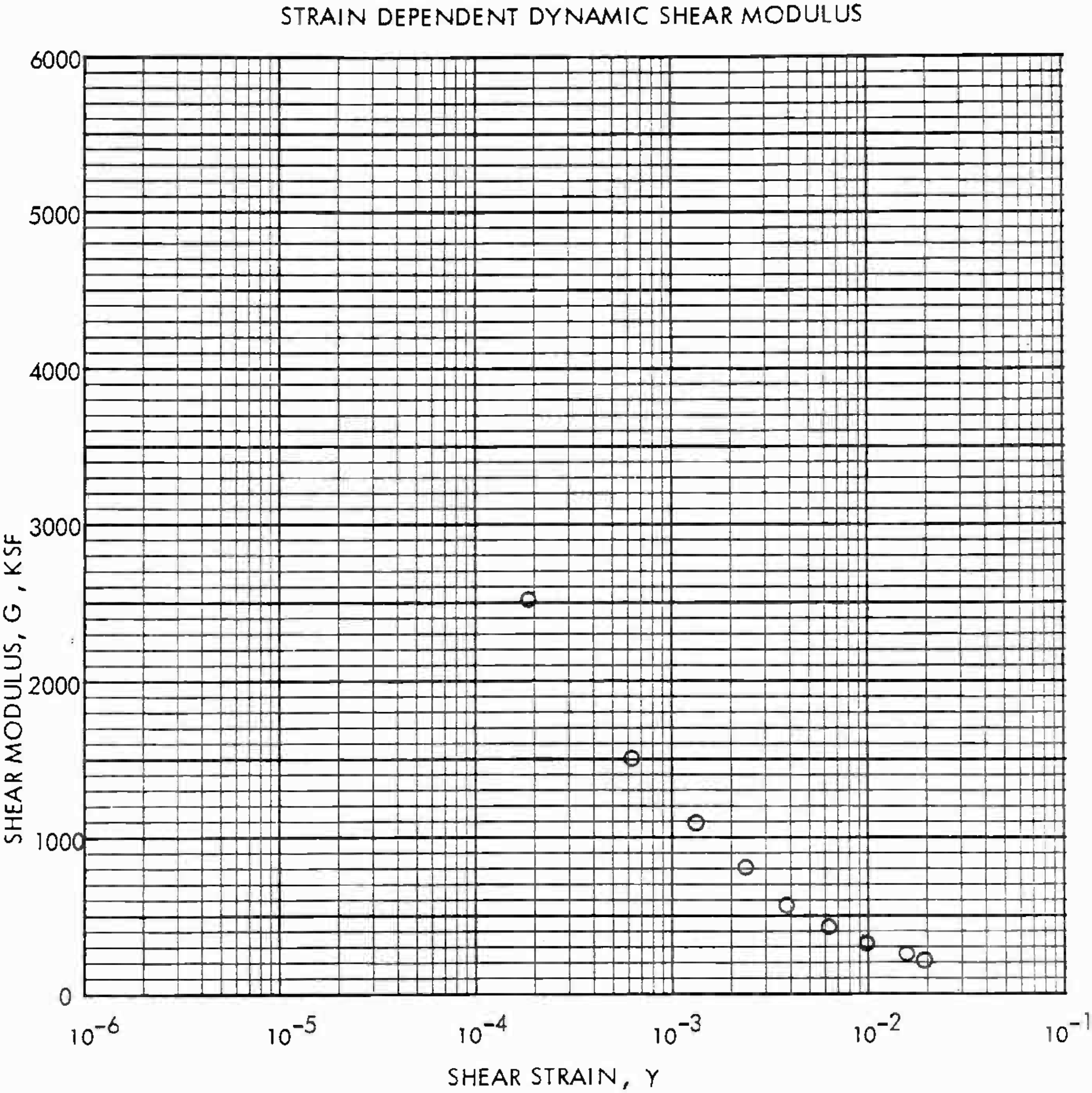

BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
17	C-2	41	95	28	30	O

Sample Description: Light Gray Clayey Silt; very stiff

DYNAMIC TRIAXIAL TEST

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Figure No. C-44

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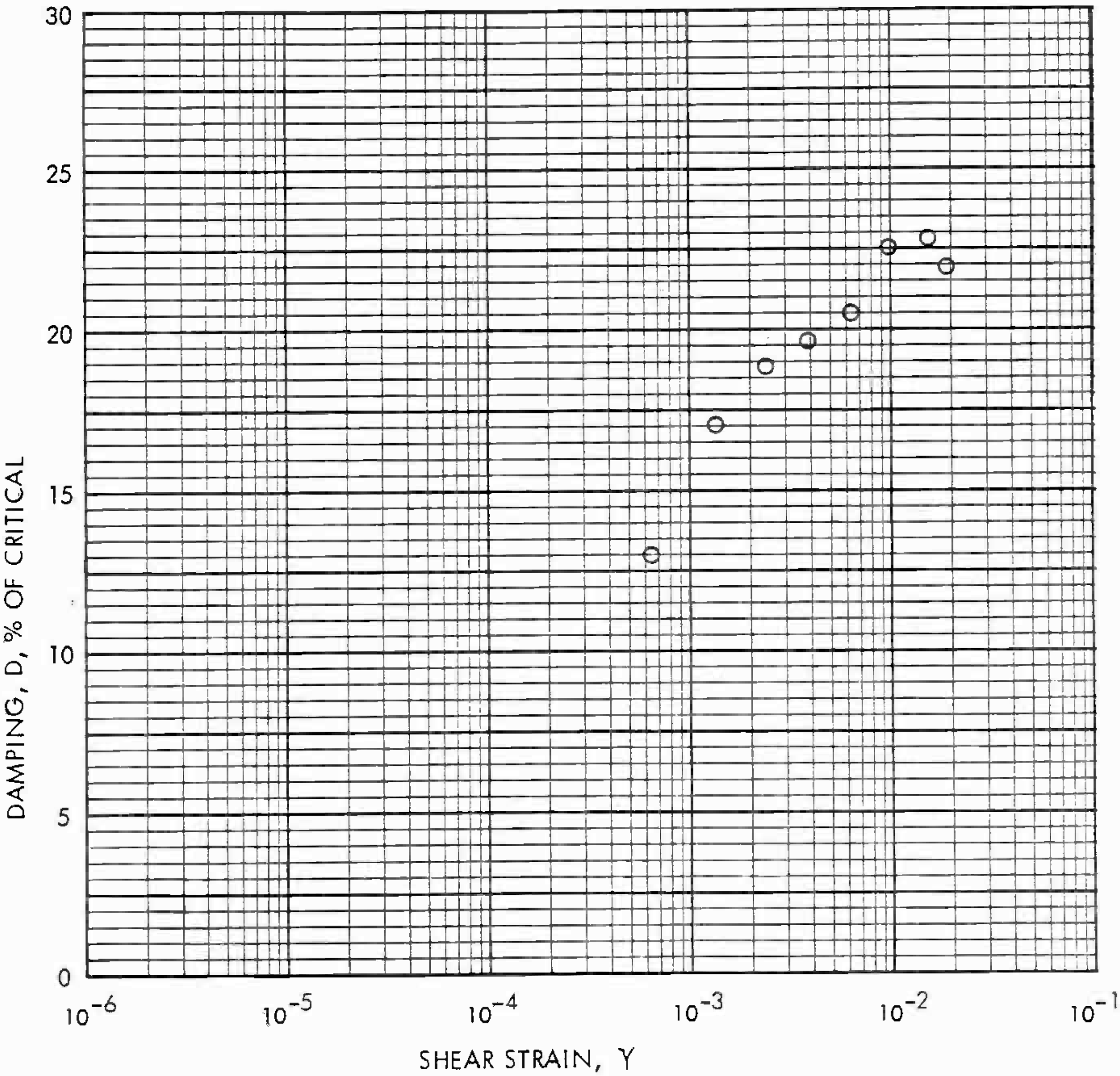
STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
17	C-2	41	95	28	30	O

DYNAMIC TRIAXIAL TEST

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Figure No
C-45

Approved for publication ____ by ____

Appendix D

Water Quality Analysis

APPENDIX D WATER QUALITY ANALYSIS

D.1 RESULTS

Water samples were taken from Borings CEG-14 and CEG-17 during the 1981 investigation and Borings 16A, 17A and 17B during the 1983 investigation. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for future tunneling activities. The chemical constituents tested are attached.

D.2 FIELD PROGRAM

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

Converse Consultants, Inc.

Lab No. P81-02-159-3

No. Samples : 5
Sampled By : Client
Brought By : Client
Date Received: 2-18-81

Sample labeled: "HOLE 14-2"

Conductivity: 1,120 µ mhos/cm

Turbidity: NTU

pH 7.9 @ 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	29		1.45
Magnesium, Mg	5		0.41
Sodium, Na	216		9.40
Potassium, K	17		0.43
		Total	11.69
Anions determined:			
Bicarbonate, as HCO_3	382		6.26
Chloride, Cl	120		3.49
Sulfate, SO_4	67		1.40
Fluoride, F	0.5		0.03
Nitrate, as N	0.7		0.05
		Total	11.23
Carbon dioxide, CO_2, Calc.	7		
Hardness, as $CaCO_3$	93		
Silica, SiO_2	29		
Iron, Fe	< 0.01		
Manganese, Mn	< 0.01		
Boron, B	0.22		
Total Dissolved Minerals, (by addition: $HCO_3 \rightarrow CO_3$)	677		

Converse Consultants, Inc.

Lab No. P81-02-159-2

No. Sampled : 5
Sampled By : Client
Brought By : Client
Date Received: 2-18-81

Sample labeled : HOLE 17-2"

Conductivity: 1,430 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	15.7	0.78
Magnesium, Mg	45	3.70
Sodium, Na	177	7.70
Potassium, K	3.8	0.10
		Total 12.28
Anions determined:		
Bicarbonate, as HCO_3	375	6.15
Chloride, Cl	240	6.66
Sulfate, SO_4	87	1.81
Fluoride, F	0.3	0.02
Nitrate, as N	0.9	0.06
		Total 14.70
Carbon dioxide, CO_2, Calc.	14	
Hardness, as $CaCO_3$	366	
Silica, SiO_2	34	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.12	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	795	

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. P83-02-162-2

Date Sampled 2/22/83
Date Received
Date Reported

Reported To:
Converse Consultants
126 West Del Mar Avenue
Pasadena, CA 91105

Attn: Al Minas

cc.

Edward W. Wilson
Laboratory Director

Sample Description 83-1101-21 BH 16A -45' Wilshire @ Irving St.

Anions	Miligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligrams per liter
Nitrate Nitrogen (as NO_3)	1.4	0.02	Hydroxide Alkalinity (as $CaCO_3$)	0.0		
Chloride	210	5.98	Carbonate Alkalinity (as $CaCO_3$)	17		
Sulfate (as SO_4)	100	2.10	Bicarbonate Alkalinity (as $CaCO_3$)	440		
Bicarbonate (as HCO_3)	540	8.82	Calcium Hardness (as $CaCO_3$)	320		
Carbonate (as CO_3)	9.8	0.33	Magnesium Hardness (as $CaCO_3$)	260		
Total Milliequivalents per Liter		17.25	Total Hardness (as $CaCO_3$)	580		
Cations	**Milligrams per liter**	**Milliequiv. per liter**	Iron			
Sodium	150	6.61	Manganese			
Potassium	3.7	0.09	Copper			
Calcium	130	6.44	Zinc			
Magnesium	64	5.26	Foaming Agents (MBAS)			
Total Milliequivalents per Liter		18.40	Dissolved Residue, Evaporated @ 180°C	914		
			Specific Conductance, micromhos @ 25°C	1630	pH	7.9

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

GENERAL MINERAL ANALYSIS*

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

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DEC 8 1983
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Log No. P83-11-056
Date Sampled 10-27-83
Date Received 11-04-83
Date Reported 12-07-83

Invoice No. 1627, separate cover

Reported To:
Converse Consultants
126 West Del Mar Avenue
Pasadena, California 91105

Attention: James A. Doolittle

cc.

Laboratory Director

Sample Description 83-1140-71, BH 17A Wilshire/Mullen St. A220

Anions	Milligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligrams per liter
Nitrate Nitrogen (as NO_3)	11	0.18	Hydroxide Alkalinity (as $CaCO_3$)	-0-		
Chloride	84	2.37	Carbonate Alkalinity (as $CaCO_3$)	-0-		
Sulfate (as SO_4)	180	3.78	Bicarbonate Alkalinity (as $CaCO_3$)	530		
Bicarbonate (as HCO_3)	640	10.5	Calcium Hardness (as $CaCO_3$)	190		
Carbonate (as CO_3)	-0-	-0-	Magnesium Hardness (as $CaCO_3$)	160		
Total Milliequivalents per Liter		16.83	Total Hardness (as $CaCO_3$)	350		
Cations	**Milligrams per liter**	**Milliequiv. per liter**	Iron	< 0.09		
Sodium	170	7.31	Manganese	< 0.04		
Potassium	1.3	0.03	Copper	< 0.07		
Calcium	75	3.75	Zinc	< 0.015		
Magnesium	40	3.28	Foaming Agents (MBAS)	< 0.1		
Total Milliequivalents per Liter		14.37	Dissolved Residue, Evaporated @ 180°C	850		
			Specific Conductance, micromhos @ 25°C	1460	pH	7.

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

lah

GENERAL MINERAL ANALYSIS*

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

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DEC 8 1983
CONVERSE CONSULTANTS

Log No. P83-11-056

Date Sampled 10-27-83
Date Received 11-04-83
Date Reported 12-07-83

Reported To: Converse Consultants

cc.

Laboratory Director

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

Anions	Miligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligrams per liter
Nitrate Nitrogen (as NO_3)	20	0.32	Hydroxide Alkalinity (as $CaCO_3$)	-0-		
Chloride	140	3.92	Carbonate Alkalinity (as $CaCO_3$)	-0-		
Sulfate (as SO_4)	70	1.47	Bicarbonate Alkalinity (as $CaCO_3$)	320		
Bicarbonate (as HCO_3)	400	6.40	Calcium Hardness (as $CaCO_3$)	230		
Carbonate (as CO_3)	-0-	-0-	Magnesium Hardness (as $CaCO_3$)	160		
Total Milliequivalents per Liter		12.11	Total Hardness (as $CaCO_3$)	390		
Cations	**Milligrams per liter**	**Milliequiv. per liter**	Iron	< 0.09		
Sodium	82	3.53	Manganese	< 0.04		
Potassium	0.8	0.02	Copper	< 0.07		
Calcium	91	4.55	Zinc	< 0.015		
Magnesium	38	3.12	Foaming Agents (MBAS)	< 0.1		
Total Milliequivalents per Liter		11.22	Dissolved Residue, Evaporated @ 180°C	670		
			Specific Conductance, micromhos @ 25°C	1200	pH	7.9

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

Appendix E

Technical Considerations

APPENDIX E TECHNICAL CONSIDERATIONS

E.1 SHORING PRACTICES IN THE LOS ANGELES AREA

E.1.1 General

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

E.1.2 Atlantic Richfield Project (Nelson, 1973)

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

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

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

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

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

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

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

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

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

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

- Basic subsurface materials were stiff clays and dense silty sands and sands. The permanent ground water table was below the level of the excavation, although minor seeps from perched ground water were encountered.

- Shoring system consisted of steel WF soldier piles placed in 36-inch diameter drilled holes spaced 6 feet on center.

- As the excavation proceeded, pneumatic concrete was placed incrementally in horizontal strips to create the finished exterior wall. The concrete which was shot against the earth acted as the lagging between soldier piles.

- Tieback anchors consisted of high-capacity 12- and 16-inch diameter friction anchors.

- Actual load imposed on the wall by the adjacent bridge was computed and added to the design wall pressures as a triangular pressure distribution.

- Maximum horizontal deflection at the top of the wall was 3 inches, while the typical deflection was less than 1 inch. Adjacent to the existing bridge, the deflections were essentially zero, with the tops of most of the soldier piles actually moving into the ground due to the high prestress loads in the anchors.

- Survey of the bridge pile caps indicated practically no movement.

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

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

- Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20° to 40°. Although the permanent ground water level was below the excavation level, perched zones of significant water seepage were encountered.

- Shoring system consisted of steel WF soldier piles placed in 20-inch diameter drilled holes spaced at 6 feet on center.

- Tieback anchors consisted of high-capacity friction anchors.

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

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

E.1.5 Design Lateral Load Practices

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

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

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

TABLE E-1

SHORING LOADS IN LOS ANGELES AREA

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

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

Note:

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

E.2 SEISMICALLY INDUCED EARTH PRESSURES

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

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

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

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

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

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

Where:

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

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

γ = unit weight of soil

ϕ = angle of internal friction of soil

i = angle of soil slope to horizontal

β = angle of wall slope to vertical

k_h = horizontal earthquake coefficient

K_v = vertical earthquake coefficient

δ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes,

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

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 6-8).

E.3 LIQUEFACTION EVALUATION METHODS

E.3.1 Standard Penetration Resistance

The use of the Standard Penetration Test (SPT) in estimating the liquefaction potential of saturated cohesionless soil deposits has been the topic of many previous investigations. Results of these investigations have recently been summarized by Seed et al (1983). Basically, the method utilizes empirical relationships which have been developed from a comprehensive collection of SPT blow count data obtained from sites where liquefaction or no liquefaction was known to have taken place during past earthquakes. Empirical relationships that have been recently proposed by Seed et al. (1983) are shown in Figure E-1.

While results of the Standard Penetration Test have been generally accepted as a good index upon which to estimate the liquefaction potential of saturated sand deposits, it should be noted that the SPT results cannot be utilized to evaluate the liquefaction potential of soils containing gravels, cobbles or boulders. However, for those soils which did not include significant percentages of gravel-sized particles, SPT blow count data were utilized along with the relationships shown in Figure E-1. In general, the SPT blow count measurements in the San Pedro Sands are greater than 50 blows per foot, indicating that these soils are generally very dense. These blow counts along with the relationship shown in Figure E-1 suggest that liquefaction of the San Pedro Sands would be unlikely during ground shaking from the maximum design earthquake. The alluvial soils generally exhibit SPT blow counts great enough to conclude that liquefaction of these soils also would be unlikely during shaking from the maximum design earthquake. Lower SPT blow counts in the alluvial soils generally reflected greater percentages of clay particles. The behavior of these clayey soils is governed by the clay characteristics discussed in E.3.3

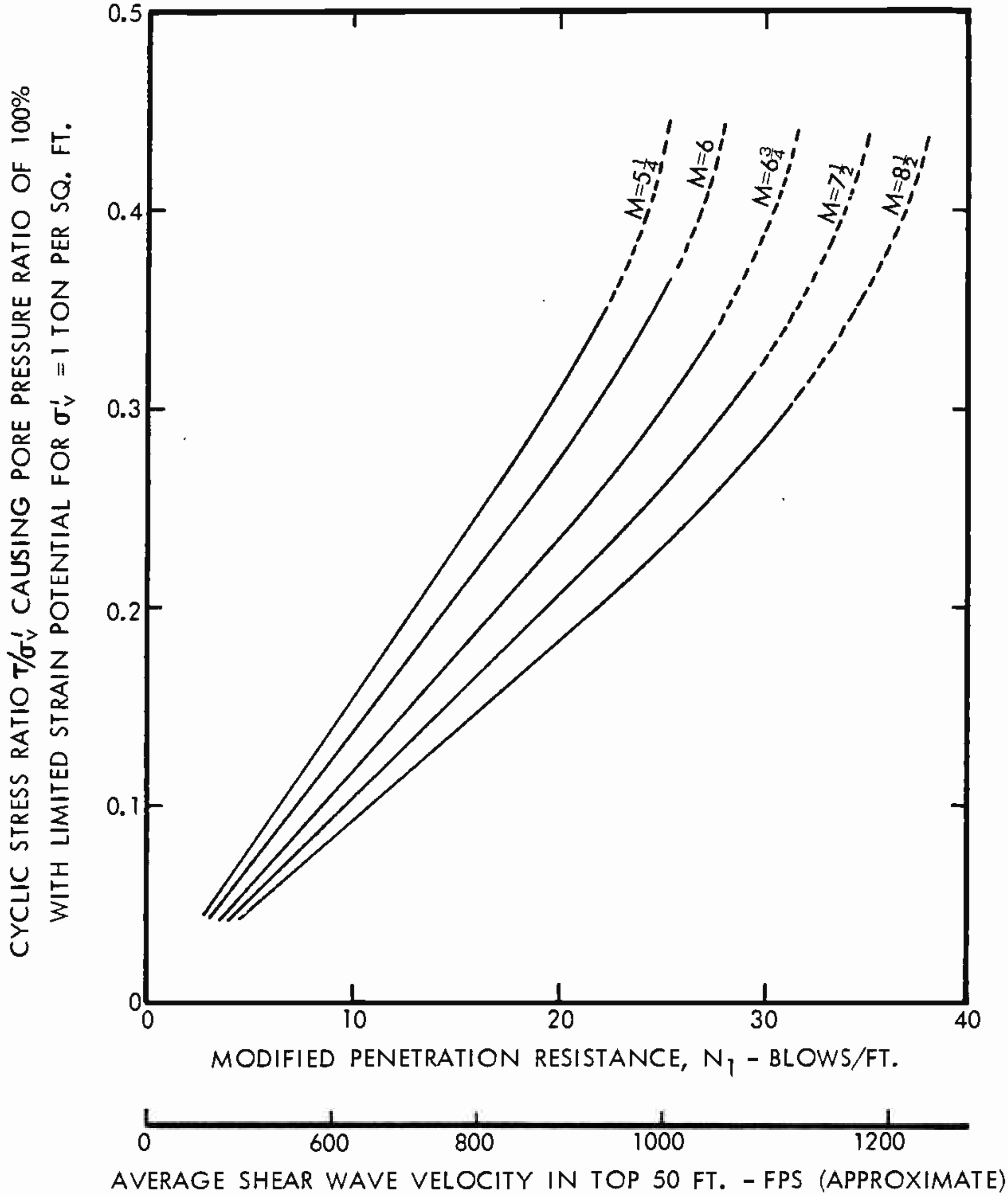
E.3.2 Shear Wave Velocity Measurements

Crosshole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project tunnel alignment were performed as part of the initial 1981 geotechnical investigation. One of the crosshole surveys was performed at Borings CEG-14 and CEG-15 near the Wilshire/Normandie and Wilshire/Western Station sites. Shear wave velocities measured in the Alluvium (approximately the upper 30 feet of the borehole) range between 890±60 fps to 990±90 fps for the crosshole measurements and 1280±90 fps for the downhole measurements.

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

E.3.3 Gradation/Plasticity Characteristics

Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figure E-2. The ranges shown in this figure have been complied by Lee and Fitton (1968), Seed and Idriss (1967), Kishida (1969), and Youd (1982) and appear to indicate that the soil types most susceptible to liquefaction consist of primarily poorly graded silty sands and sandy silts. It is important to note that all the gradational ranges shown in Figure E-2 have less than 20% by weight clay size particles



(after Seed, 1983)

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

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

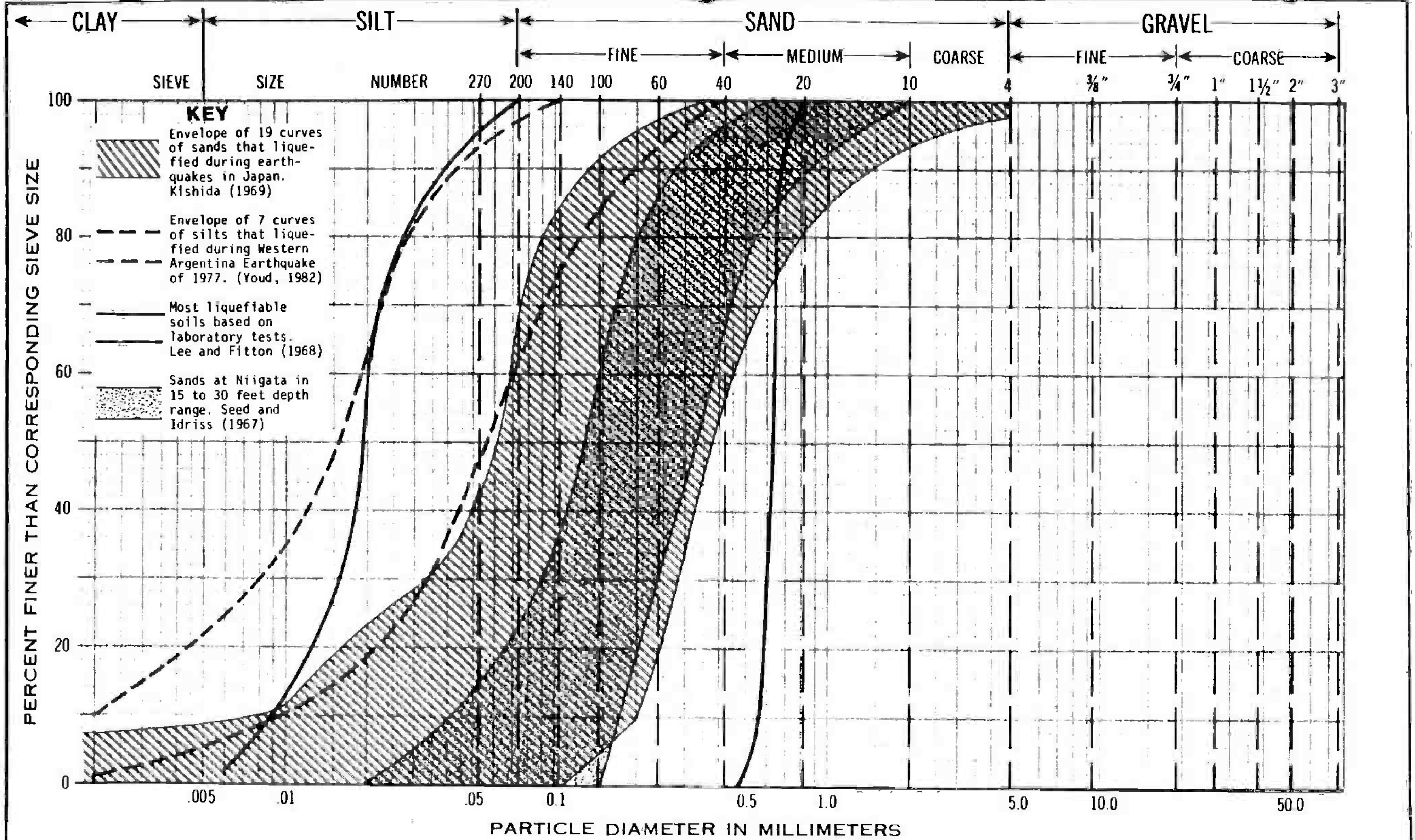
Project No. 83-1140

Figure No E-1

Converse Consultants
Geotechnical Engineering and Applied Sciences

Approved for publication ________ by ________

FORM NO. D-3-10/8 Approved for publication ________ by ________



GRADATIONS OF SOILS CONSIDERED SUSCEPTIBLE TO LIQUEFICATION

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1140

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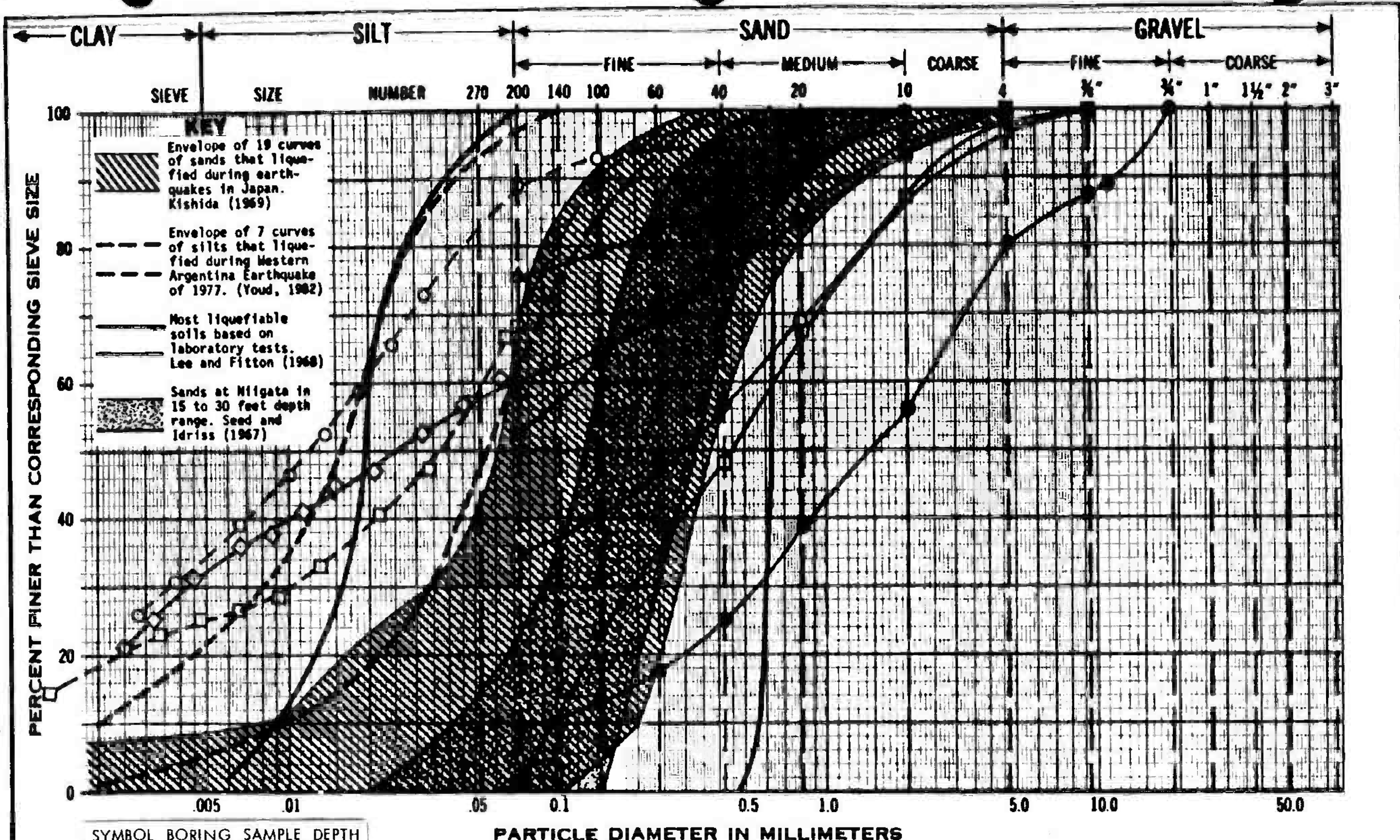
Figure No.
E-2



SYMBOL	BORING	SAMPLE	DEPTH
—○—	14-1	C-11	46.0'
—△—	14-1	C-12	52.0'
—■—	14-1	C-14	62.0'
—●—	14-2	C-9	42.5'
—□—	14-3	C-9	42.0'
- ○ -	15	C-3	60.0'
—◇—	15-1	C-10	51.5'
- □ -	15-4	C-6	27.5'

GRAIN-SIZE DISTRIBUTION CHART

DESIGN UNIT A220
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140-26

Converse Consultants
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Figure No. E-3

(i.e., particles less than 0.005 mm), suggesting that clayey (cohesive) soils have a low liquefaction potential. Seed and Idriss (1983) stated that clayey soils are not vulnerable to significant strength loss during earthquakes if the percentage of particles finer than 0.005 mm is greater than 20 or if the water content is less than nine-tenths of the Liquid Limit. Gradation characteristics typical of gravels and gravelly soils are also absent from Figure E-2 suggesting, in part, that these types of soils may not be capable of developing high excess pore pressure because they are either capable of draining rapidly during the cyclic loading or they are usually more efficiently packed (i.e., denser) in situ than soils that consist of uniformly-sized particles. While the liquefaction potential of a soil is dependent on many factors other than gradation (such as the relative density of the soil, the intensity and duration of cyclic loading, among others), comparisons of the gradational characteristics of a soil with those ranges shown in Figure E-2 provides a useful guide in establishing the liquefaction potential of a soil.

The gradational characteristics of the various soils which comprise the onsite Alluvium were compiled from laboratory tests performed during this and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" sandy soils shown in Figure E-2 are presented in Figure E-3.

Figure E-3 indicates that none of the samples tested falls entire within the range of gradations of soils considered "susceptible" to liquefaction. On the basis of gradation alone, there appear to be few alluvial soils at the site which are susceptible to liquefaction. The clayey alluvial soils satisfy the criteria described in Seed and Idriss (1983) for non-liquefiable clayey soils.

E.3.4 Conclusions

Based on the above considerations and comparisons, it is our judgement that the alluvial soil deposits would have low liquefaction potential during ground shaking from the maximum design earthquake. The low liquefaction potential of the alluvial soils is anticipated due to sufficiently high SPT blow counts or due to sufficiently high clay content and clay characteristics. The San Pedro Sands would have low liquefaction potential for similar ground shaking due to sufficiently high SPT blow counts.

E.4 PREVIOUS TUNNELING EXPERIENCE - LACFCD SACATELLA TUNNEL

E.4.1 Facts and Figures

The following tunneling data were received in an oral communication in June 1981 with the contractor, Donald Glanville of Glanville Construction Company, and John E.Witte, Tunnel Consultant, as well as LACFCD Pre-construction "Geologic Report", dated December 26, 1973; and Victor L. Wright's "Pre-Bid Geologic Appraisal" report, dated July 1975.

Tunnel Length	0.6 miles
Tunnel Diameter	18 ft O.D. excavated; 14.5 ft I.D.
Initial Support	Precast concrete liner (3 segments/ring)
Excavation Method	Digger Gradall & shield
Advance Rate	Maximum 32 ft/8-hr shift; average 15 ft
Geology	Claystone, siltstone & occasional interbeds of very hard "calcareous" cemented sandstone
Eventual Use	Storm drain, LACFCD
Contractor	Glanville Construction Co.
Bid Price	±$4,000,000
Extras Awarded	±$500,000
Tunnelling Period	1975-77

E.4.2 Relation to Metro Rail Alignment

The Los Angeles County Flood Control District's (LACFCD) Sacatella Tunnel is in litigation for "changed (geologic) conditions" in the tunnel (settled) and at both portals (unsettled). For this reason, the LACFCD was reluctant to release information.

Geologic conditions and tunneling methods in this tunnel are very important to the Metro Rail alignment because:

- Tunnel was excavated in a "gassy" reach under Hoover Street, north of Wilshire Boulevard, in claystone, siltstone and sandstone of the Puente Formation (Unit C).
- Formation is similar to the material anticipated in Metro Rail alignment Reaches 1 to 5 (Design Units A140 to A310).
- Total cover above tunnel crown ranges from 22 to 25 feet.
- Total bedrock cover above tunnel crown ranges from 2 to 25 feet.
- Old Alluvium cover above the tunnel crown ranges from 5 to 32 feet.

E.4.3 Peak Unconfined Compressive Strength

LACFCD test results of peak unconfined compressive strength, from six core samples obtained in the Puente Formation, are tabulated as follows:

LACFCD BORING	UNCONFINED COMPRESSIVE STRENGTH, Qu (psi)
1	401
1	603
2	441
2	384
2	377
7	172
Average	396

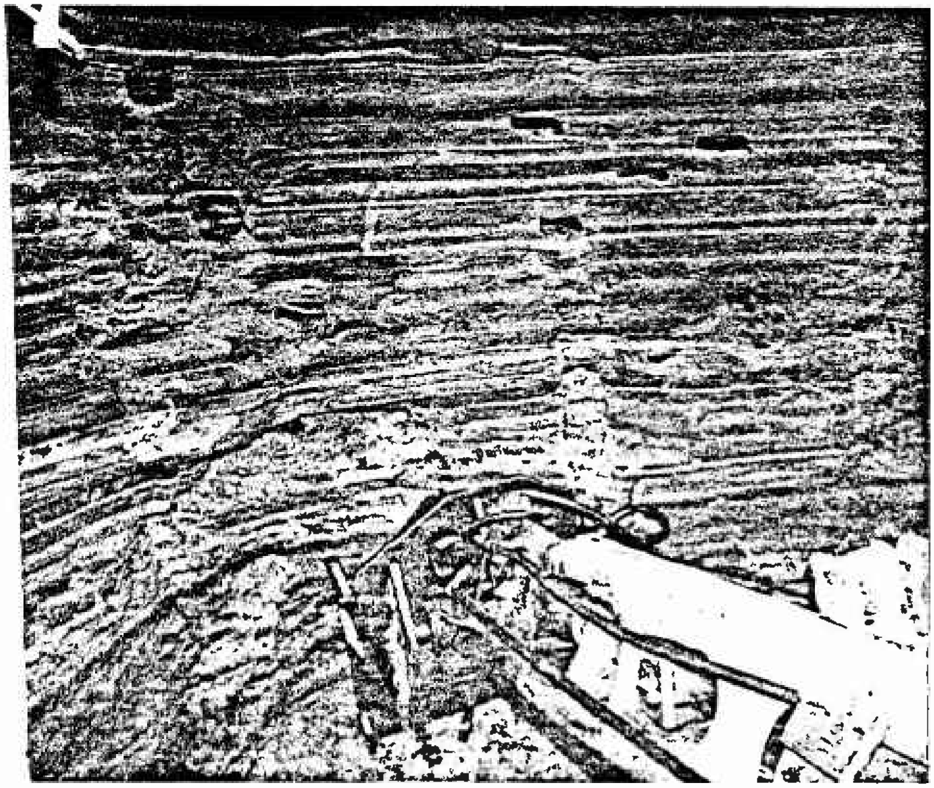
Core samples from Borings 1 and 2 were taken essentially normal to the bedding, while the bedding at Boring 7 was inclined at about 45 degrees from the long axis of the core. This probably accounts for the considerably lower compressive strength test value for the sample from Boring 7. All core segments tested were selected for cross-sectional uniformity and freedom from cracking or damage and, as such, are considerably more competent than the average grade of rock encountered during drilling. Therefore, the values obtained for the compressive strength are probably greater than the average values which would be found during tunnel excavation (LACFCD, 1973).

E.4.4. Digger Excavator and Shield

The tunnel excavation was performed with a small (Model No. 2403) Gradall excavator. The rotating, telescoping boom was connected to a flat plate that had a single ripper tooth on one edge and several digger teeth on the other edge (Figure E-4). Also note in Figure E-4 Puente Formation bedding (Unit C) and lack of ground water inflow.

E.4.5 Geology

Puente Formation: Thin bedded, soft claystone and siltstone. The formation contained occasional interbeds of very hard "calcareous" cemented sandstone from 2 to 12 inches in thickness with unconfined compressive strength of 5,000 to 15,000 psi. These interbeds caused the "changed conditions", according to Donald Glanville, as they were not mentioned in the pre-construction reports. Some very hard interbeds were nearly horizontal and followed the face for several hundred feet; some were at a 45° angle to the tunnel alignment and followed the face for several tends of feet. This resulted in the following actions:



Digger Excavator
(LACFCD Sacatella Tunnel)

Figure E-4

- replaced single-tooth ripper with hydraulic jackhammer to break up hard layer (removed jackhammer in weak ground)

- bent leading edge of shield, forcing contractor to stop and repair often; i.e., spent 8-hour shift digging and balance of day repairing shield

- difficult to maintain line and grade in hard rock layers (These hard layers, although 12 inches or less in thickness, made drilling of 5-foot diameter man-way shafts very difficult also.)

- advance rate cut drastically; i.e., often reduced advance rate to 1 to 5 feet daily.

E.4.6 Tunnel Gas Classification

The tunnel was classified "gassy" because it traversed the Los Angeles City Oil Field. However, no fire or explosion occurred during the project.

- The greatest apparent risk is where folding and a suspected fault may form significant traps (Wright, 1975, p. 8). Explosive-proof equipment was installed (although arc welding was permitted in the tunnel).

- The face was continuously monitored by a gas "sniffer" that automatically set off an alarm if high LEL readings were recorded. (Note: Alarm was never activated because ventilation was so effective.)

- Installed 4-foot-diameter ventilation duct and pumped air at 400 cfpm through the vent pipe.

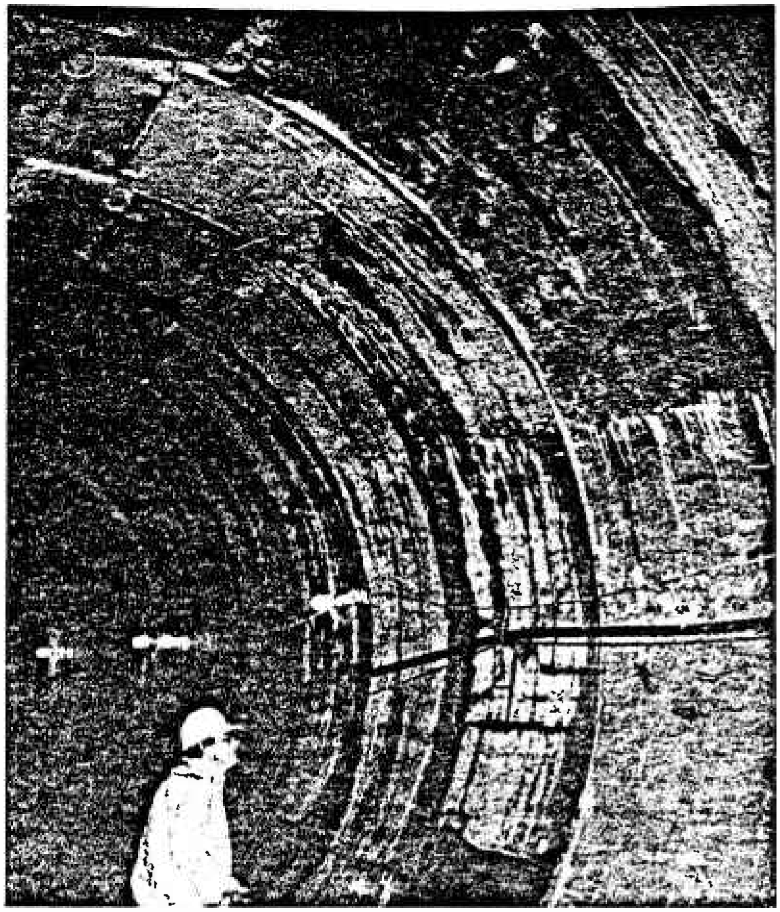
- Oil, seeping down the sides of the supports, was skimmed off the discharge water at the portal and hauled away by tank truck (personal communication, R.J. Proctor,1981). Oil seeps are shown on Figure E-5.

E.4.7 Abandoned Oil Wells

The tunnel encountered several uncharted, uncased, abandoned oil wells. Although oil was not encountered in these holes, several hundred gallons of water gushed into the tunnel for a few seconds, alarming the miners each time.

E.4.8 Ground Water

The tunnel was below a "permanent" water table. The water table was in the Puente Formation and the overlying Old alluvium. The contractor drilled 12 dewatering wells at selected locations along the alignment prior to excavating the tunnel. This dewatering of twelve 24-inch-diameter wells, recommended by Vic Wright, Tunnel Consultant, appears to have successfully kept tunnelling conditions in the "dry". According to Wright, 1975, "... ground water problems in the [Puente] formations are expected to be related more to softening and weakening, especially in the sticky shale zones, rather than to water volume."



Oil Seeps
(LACFCD Sacatella Tunnel)
Figure E-5

The wells pumped about 20 gpm each from about 25 feet of overlying Old alluvium and 20 feet of Puente Formation. The water was pumped to the surface, and the contractor believes this kept tunnel inflow to a minimum, i.e., "dripping" condition rather than 10 to 100 gpm local inflows.

The following ground water information on transmissibility, permeability and artesian conditions in Old alluvium and Puente Formation at the Sacatella tunnel is not a substitute for dewatering pump tests for the Metro Rail Project. However, the data do provide some relative measure of inflow rates that could be locally applicable to the Metro Rail alignment. The following is excerpted from the LACFCD Geologic Report, pages 7 and 8:

> Ground water was found in all [LACFCD] borings. However, due to drilling fluid in the boring, it was not possible to accurately determine the depth at which ground water was first encountered or if there were artesian or perched water table conditions. The initial soils investigations were conducted by the City of Los Angeles between 1967 and 1972, using augers which did not require drilling fluid. Logs of these borings indicated, at least in several locations, that water is perched in the unconsolidated sediments [Old alluvium] overlying the bedrock and is also found within the bedrock [Puente formation, Unit Cw], often under minor artesian head. Artesian head in the vicinity of Boring No. 3 was noted previously by the City as being particularly high with water rising from a depth of 33 feet to 13 feet overnight. Other borings in the vicinity had artesian heads of only 1 to 2 feet (City of Los Angeles Soils Investigation report, Test Boring Nos. 48, 48A and 48B). Static water levels in all borings were well above the top of the proposed tunnel, indicating that the excavation will probably be conducted under saturated conditions. The measurements for individual borings are listed in Table E-2.

Core samples [LACFCD] of the bedrock appeared to have extremely low permeabilities; hence it is presumed that ground water movement occurs through bedding planes, fracture fissures, rather than through pores in the rock. Estimates of bedrock transmissibility and permeability were made using the recovery time of the water surface in the borings after air jetting. The results are listed in Table E-2. The Coefficient of Transmissibility "T" ranges from 0.41 to 7.66 and is defined as the rate of flow in gallons per day through a vertical section of the water-bearing material, in which the width is 1 feet and the height is the measured thickness. The Coefficient of Permeability "p" is the flow in gallons-per-day through a cross-sectional area of 1 square foot of saturated material. The average coefficient of permeability was calculated from the coefficient of transmissibility by dividing this value by the footage thickness of the saturated material.

E.4.9 Stand-up Time, Slabbing, Overbreak

Stand-up time was more than 2 to 3 hours prior to placing liner. Slabbing of flat-lying or steeply dipping beds did not occur. No overbreak was recorded, but minor air slaking developed due to the high air ventilation. Mr. Glanville called this "ideal" tunneling formation, except for the hard cemented layers.

TABLE E-2

COEFFICIENTS OF TRANSMISSIBILITY AND PERMEABILITY
LACFCD (1973)

LACFCD BORING No.	APPROXIMATE[a] LOCATION	DEPTH (ft)	DISTANCE TO WATER (ft)	SATURATED MATERIAL OVERLYING TUNNEL (ft)	ESTIMATED YIELD (gpm)	ESTIMATED[b] T	ESTIMATED[c] P	MAXIMUM[d] GAS READING	GROUND WATER SULFATE CONTENT (ppm)
1	292+70	47	10.5	9.5	0.15	2.43	0.07	17	66
2	287+58	52.25	7.2	19.0	0.49	4.96	0.02	12	778
3	282+46	53	10.0	15.0	1.23	7.66	0.18	0	928
4	277+00	47.17	9.2	10.0	0.09	0.63	0.02	7	1,350
5	272+60	50.75	10.6	12.2	0.10	1.13	0.03	20	154
6	229+94	54.08	17.6	5.5	0.03	0.41	0.01	0	252
7	227+27	59	23.7	4.3	0.17	2.28	0.06	0	182

[a] Refer to LACFCD Dwg. Nos. 364-1102-D7.6 and D8.4-8-7.

[b] T = Coefficient of Transmissibility in gallons per day per foot.

[c] p = Coefficient of Permeability in gallons per day per square foot.

[d] In percent of Lower Explosive Limit (LEL).

E.4.10 Ground Settlement Above Tunnel

The tunnel was excavated within 40 feet below the street surface in a residential area with one hotel. No settlement was noted, or reported, by the residents. No known complaints of noise, except at portals, were registered by the residents living above the tunnel during construction.

E.4.11 Local Caving Problem

An abandoned 2-foot-diameter auger hole was penetrated. The hole caved upward to within 6 feet of the ground surface. The contractor drilled a hole from the surface into the cavern and filled the cavern with pea gravel prior to advancing the tunnel. The cave did not "daylight" to the surface.

E.4.12 Portal Excavation Problems

Both portal excavations encountered local, very hard sandstone interbeds which could not be excavated by small equipment. Therefore, heavy equipment (D-9 Caterpillar) was required. These are part of the "changed conditions" (as yet unsettled), according to Mr. Glanville.

E.4.13 Ground Loading and Estimated Support Requirements

The following ground loading and estimated support requirements were reported (Wright, 1975, p.5 and 6):

> Continuous light tunnel support will be necessary whether the tunnels are driven by boring machine or by drilling and blasting. The need for immediate support may often be marginal if the tunnel is machinebored. However, the shales will need support eventually because of stress relief fracturing and slaking. Slaking was evident in a small percentage of the cores. The generally short core lengths are probably due to stress relief. Ground loading assumptions in the specifications seem unreasonably high at 3370 psf. Maximum estimated loads for this study are 2400 psf, where the ground is wet and highly unstable. Most loads should be on the order of only 800 to 1600 psf. Lateral loading up to possibly 800 psf may build up in the wet unstable reaches.
>
> Six-inch, 15.5# steel horseshoe sets spaced 3 to 5 feet apart will hold the estimated loads. A few invert struts may be necessary where the formation is extensively softened by ground water, especially through the low bedrock cover reaches.

Appendix F

Earthwork Recommendations

APPENDIX F EARTHWORK RECOMMENDATIONS

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

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

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

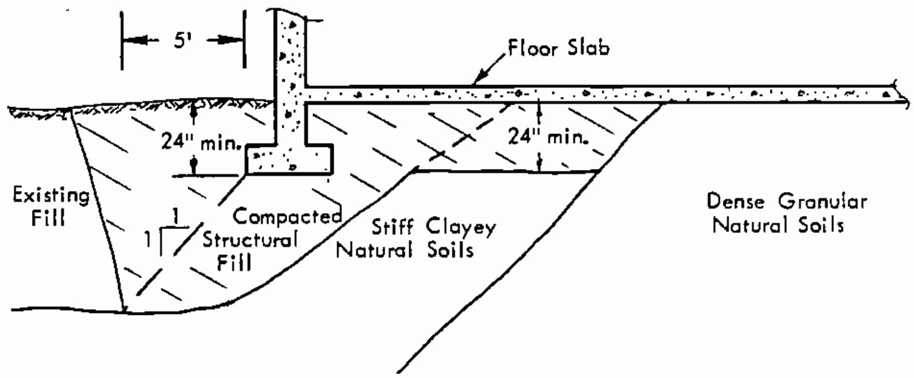
- Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed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

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

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

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

FOUNDATION/SUBGRADE PREPARATION



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

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

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

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

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

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

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

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

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

Appendix G

Geotechnical Reports References

APPENDIX G GEOTECHNICAL REPORTS REFERENCES

REPORT No.	REPORT OATE	LOCATION	CONSULTANT
12	05/23/67	3130 Wilshire	LeRoy Crandall & Associates
13	08/28/50	Northeast corner Vermont & Wilshire	L.T. Evans
13a	02/09/59	North of northwest corner Vermont & Wilshire	L.T. Evans
14	07/15/69	Southwest corner New Hampshire & Wilshire	LeRoy Crandall & Associates
15	08/27/79	Corner of Wilshire & Catalina	LeRoy Crandall & Associates
16	09/21/66	South of Wilshire at Mariposa	L.T. Evans
17	12/29/67	Block bounded by Kingsley, Wilshire, Ardmore and Seventh	LeRoy Crandall & Associates
18	09/11/68	North of Wilshire between Oxford & Serrano	LeRoy Crandall & Associates
19	06/10/69	North of Wilshire between Oxford & Serrano	P&C Drilling Company
20	02/16/54	Northeast corner Norton & Wilshire	L.T. Evans
21	11/15/81	4200 Wilshire Boulevard at Lorraine	LeRoy Crandall & Associates
22	03/21/67	4311 Wilshire at Windsor	LeRoy Crandall & Associates
23	04/14/47	Block bounded by Wilshire, Mansfield, Carling and Citrus	L.T. Evans
24	03/04/47	Northeast corner Wilshire & Curson	L.T. Evans
25	04/22/47	Northeast corner Wilshire & Sierra Bonita	L.T. Evans
26	10/27/69	Block bounded by Wilshire, Masselin, Eighth and Curson	L.T. Evans