Southern California Rapid Transit District
METRO RAIL PROJECT

RTD

Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A170

BY

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

OCTOBER 1983

Funding for this Project is provided by grants to the Southern California Rapid Transit District from the United States Department of Transportation, the State of California and the Los Angeles County Transportation Commission."

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

E R R A T A

GEOTECHNICAL REPORT

DESIGN UNIT A 170

1. OMIT "EARTHQUAKE" LOADING DIAGRAM IN FIGURE 6-6.

2. OMIT TEXT OF SECTION 6.11.1, PAGE 30 AND REPLACE WITH THE FOLLOWING TEXT:

   > "DESIGN PROCEDURES AND CRITERIA FOR UNDERGROUND STRUCTURES UNDER EARTH-QUAKE LOADING CONDITIONS ARE DEFINED IN THE SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT (SCRTD) REPORT ENTITLED "SUPPLEMENTAL CRITERIA FOR SESMIC DESIGN OF UNDERGROUND STRUCTURES," DATED JUNE 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."

3. OMIT TITLE AND TEXT OF SECTION 6.11.3, PAGE 32.

4. SECTION 7.1 PAGE 35, PARAGRAPH 1, LINE 2, ADD PERIOD AFTER "TUNNELLING." OMIT REMAINDER OF LINE 2, LINE 3, LINE 4 AND LINE 5, UP TO AND INCLUDING THE WORD "OPERATION."

   SECTION 7.1 PAGE 35, PARAGRAPH 3, LINE 4, ADD PERIOD AFTER "OPERATIONS." OMIT REMAINDER OF PARAGRAPH.

   SECTION 7.1 PAGE 35, PARAGRAPH 5, LINE 3, ADD PERIOD AFTER "TUNNEL" AND OMIT REMAINDER OF LINE 3 AND LINE 4 UP TO AND INCLUDING THE WORD "SHIELD."

SECTION 7.1 PAGE 36, PARAGRAPH 1, OMIT THE LAST SENTENCE OF THIS PARAGRAPH.

SECTION 7.1 PAGE 36, OMIT ALL OF PARAGRAPH 5.

SECTION 7.1 PAGE 36, OMIT LAST THREE LINES ON THIS PAGE.

SECTION 7.1 PAGE 37, OMIT FIRST TWO LINES ON THIS PAGE.

SECTION 7.3.1 PAGE 38, PARAGRAPH 1, OMIT REMAINDER OF PARAGRAPH AFTER WORD "INVERT" IN LINE 5.

SECTION 7.3.1 PAGE 38, PARAGRAPH 2, LINE 3, OMIT COMMA AFTER "CONSULTANTS," ADD APOSTROPHY TO THE "S" IN "CONSULTANTS." ADD "REPORT DATED JUNE 1983" AFTER CONSULTANTS.

SECTION 7.3.3 PAGE 39, PARAGRAPH 1, LINE 7, OMIT "CUT-AND-COVER."

SECTION 7.3.4 PAGE 40, PARAGRAPH 2, OMIT ENTIRE PARAGRAPH.

5. OMIT FIGURE 7.1.

6. OMIT TITLE AND TEXT OF SECTION 7.4 IN ITS ENTIRETY.

7. PAGE 44 - REVISE 7.5 TO READ 7.4.

8. OMIT PARAGRAPH 5, PAGE F-7, SECTION F.2 APPENDIX F.

Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

October 11, 1983 83-1101-83

Southern California Rapid Transit District
Metro Rail Project - WBS 12AAC
425 South Main Street
Los Angeles, California 90013

Attention: Mr. James Crawley
Director of Engineering-Transit Facilities

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A170 (Alvarado Station and two miles of tunnel line) prepared in accordance with the Contract No. 2256-2 agreement dated January 14, 1983 between Converse Consultants, Inc. and Southern California Rapid Transit District (SCRTD). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A170.

Our study team appreciate the assistance provided by the District's Board of Special Geotechnical Consultants and the Metro Rail Transit Consultants; namely, Dick Proctor, Jack Yaghoubian, Bud Maduke and Bill Armento. We also want to acknowledge the dedicated efforts of each member of the Converse team, especially Julio Valera, Howard Spellman, Bob Plum, and Jim Doolittle.

Respectfully submitted,

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

RMP:mro

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

PROFESSIONAL CERTIFICATION

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

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

REGISTERED PROFESSIONAL ENGINEER
ROBERT M. PRIDE
No. C 21086
CIVIL
STATE OF CALIFORNIA

Robert M. Pride
Senior Vice President

REGISTERED GEOLOGIST
HOWARD A. SPELLMAN, JR.
No. 134
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Howard A. Spellman
Principal Engineering Geologist

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for Design Unit A170. This unit includes the proposed Wilshire/Alvarado Station and Crossover Structure and about 2 miles of tunnel line which will be part of the Metro Rail Project in the Los Angeles area. The proposed cut-and-cover structures at the Station site will be about 950 feet long, 60 feet wide, and will require excavating some 43 to 63 feet below the existing ground surface at the station site. The purpose of the investigation is to provide geotechnical information and recommendations to be used by design firms in preparing designs for the project. Although this report may be used for construction purposes, it is not intended to provide all of the information that may be required to construct the project.

Subsurface conditions along the Metro Rail Design Unit A170 Tunnel Line are judged favorable for mechanical excavation tunneling, except from the Alvarado Station to the west end of MacArthur Park Lake (approximately Station 276) where cut-and-cover construction is recommended. We recommend utilizing economical pre-cast concrete segments to form both initial support and permanent lining in one construction operation. There will be significant intervals with comparable soft ground tunneling conditions, and thus comparable excavation characteristics.

The tunnel will be advanced through alluvium, bedrock and mixed face alluvium/bedrock contacts. Ground water will occur within the alluvial deposits. The primary geotechnical tunneling issues involve the type of material to be tunneled, the occurrence of mixed face conditions including boulders and occasional hard rock strata, the occurrence of groundwater and gassy ground. Squeezing ground should not be a particular stability problem in normal shielded TBM operations nor should the ground pressure exceed the capacity of normal support systems.

The crown of the tunnel coincides with the bottom of MacArthur Park Lake. Therefore, dewatering (draining) the lake, prior to cut-and-cover construction beneath the lake, should help reduce water inflows into the excavation. Even with dewatering of the lake, running ground is likely to occur within the excavation.

Preliminary plans indicate about 10 cross-passages between tunnels are proposed in Design Unit A170. These 20-foot long, 10-foot wide, 12-foot high passages, connecting the tunnels, will require mining in siltstone, claystone and sandstone of the Fernando and Puente Formations.

In general, the subsurface conditions at the structures consist of 8 to 28 feet of Old Alluvium consisting primarily of silts, clays, clayey sands and silty sands. Underlying the Old Alluvium, the explorations encountered interbedded siltstone, claystone and sandstone of the Fernando and Puente Formations. Ground water was encountered within the Old Alluvium at depths of 7 to 9 feet below the existing ground surface. This groundwater is perched above the bedrock.

Construction of the Station and Crossover will involve making a deep excavation through the Old Alluvium into predominantly siltstone and claystone bedrock. This will involve shoring and dewatering. The permanent structure will in essence be a concrete box bearing on the Puente Formation bedrock and retaining both the Old Alluvium and the Puente Formation bedrock.

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

- EXCAVATION DEWATERING: Since the excavation will extend through and below the perched water zone, a dewatering system may be required to construct the proposed excavation. However, use of sumps within the excavation may be adequate due to the relatively impermeable nature of the fine-grained Old Alluvium and Puente Formation bedrock. The contractor will be responsible for designing, installing and operating a suitable dewatering system. The report presents groundwater data and general dewatering criteria to be satisfied by the contractor.

- UNDERPINNING: Because there are no major structures located in close proximity to the proposed excavations, underpinning will probably not be required.

- TEMPORARY EXCAVATION SUPPORT: The excavation system will be chosen and designed by the contractor in accordance with specified criteria and subject to review and acceptance by the Metro Rail Transit Consultants. In our opinion the contractor will most likely propose a shoring system consisting of soldier piles placed in predrilled holes with either tiebacks or cross-lot bracing for lateral support. Accordingly, design criteria for these types of systems are presented in the report. Other systems may also be appropriate and should be considered by the contractor, such as the special cut-and-cover construction recommended beneath MacArthur Park Lake.

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

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

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

- LIQUEFACTION POTENTIAL: Based on the gradation and characteristics of the natural soils and their probable densities, it is our opinion that there is a low risk of liquefaction at the Station site.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Design Unit A170. The subject Design Unit includes the proposed Wilshire/Alvarado Station and Crossover Structure and about 2 miles of two single track tunnels. These structures will be part of the proposed 18-mile long Metro Rail Project (see Drawing 1, Vicinity Map). The purpose of the investigation is to provide geotechnical information to be used by the design firms in preparing designs for the project. Although this report may be used for construction purposes, it is not intended to provide all the geotechnical information that may be required to construct the project. The work performed for this study included field reconnaissance, borings, geologic interpretation, laboratory testing, engineering analyses, and development of recommendations.

Additional geotechnical information on the project is included in the following reports:

- "Geotechnical Investigation Report, Metro Rail Project", Volume I -Report, and Volume II - Appendices, prepared by Converse Ward Davis Dixon, Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in November 1981: This report presents general geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experiences and practices in the Los Angeles area.

- "Seismological Investigation & Design Criteria, Metro Rail Project", prepared by Converse Consultants, Lindvall, Richter & Associates, Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in May 1983: This report presents the results of a seismological investigation and establishes seismic design criteria for the project.

- "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF866, 1977), prepared by the U.S. Geological Survey in cooperation with the U.S. Department of Transportation. This publication includes a compilation of boring data in the general vicinity of the proposed Metro Rail Project.

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

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

The design concepts evaluated in this geotechnical report are based on the "Draft Report for the Development of Milestone 10: Fixed Facilities: dated March 1983 and revised plans A-21 through A-24. These documents were prepared for SCRTD by Harry Weese and Associates, Tippet-Abbett-McArthy-Stratton, Environmental Collaborative, Inc., and Gin Wong Associates.

Section 3.0

Project and Site Description

3.0 PROJECT AND SITE DESCRIPTION

3.1 SITE DESCRIPTION - STATION AND CROSSOVER

The proposed Wilshire/Alvarado Station and Crossover Structure, as shown in Drawings 1 and 2, trends west-northwest to east-southeast and will be located off-street at mid-block between 7th Street and Wilshire running from Alvarado to Bonnie Brae. The station vicinity is known as the Westlake Area. Across Alvarado from the station is MacArthur Park, which is a heavily used public space during daytime hours. The immediate area is moderately to densely developed with an even split of both commercial and residential buildings which are typically 1 to 4 stories high. Several buildings located along Wilshire Boulevard are mid-rise office buildings which are on the order of 10 stories high. Currently at the central portion of the proposed station there are parking lots, garages and 2 level parking structures. Several 1 to 3 story retail structures exist along the east side of Alvarado Street at the west end of the proposed station and on the east side of Bonnie Brae Street.

The topography generally slopes gently to the south and west. There is a N-S trending vertical drop of approximately five feet supported by a brick retaining wall located at the alley between Westlake Avenue and Alvarado Street. West of Alvarado, the grade becomes steeper down to the lake in MacArthur Park. The bottom of the lake is at an elevation that coincides with the crown of the tunnel. Significant vegetation in the vicinity is primarily located in MacArthur Park where there are large palms and deciduous trees. A few small trees exist at the corner of Bonnie Brae and the proposed alignment.

Pedestrian and vehicle traffic is extremely heavy on Alvarado Street, 7th Street and Wilshire Boulevard from early morning until well into the evening. Bonnie Brae Street and Westlake Avenue have light to moderate foot and vehicle traffic.

3.2 SITE DESCRIPTION - TUNNEL LINE

As shown on Drawings 2, 3 and 4 the tunnel line in Design Unit A170 is about 2 miles long, starting at approximately Station 205 and ending at approximately Station 309. The tunnel line continues in a northwesterly direction from 7th/Flower Station under 7th Street, curving northwesterly under Alvarado and MacArthur Park Lake, linking with Wilshire Boulevard. It then continues northwesterly under Wilshire, terminating at a point just north of Wilshire and east of Vermont.

3.3. PROPOSED STATION AND CROSSOVER STRUCTURE

The proposed main Station area will be about 550 feet long and 60 feet wide (outside wall dimensions). The Crossover Structure will extend approximately 400 feet from the east end of the Station, making the total length of the proposed cut-and-cover section approximately 950 feet. The proposed top of rail varies from about Elevation 228 at the west end to Elevation 231 at the east end of the Station platform. Assuming that the

Station will be supported on a 4 to 6 feet thick concrete mat, the station area will require an excavation to about Elevation 224. This is approximately 43 feet below the existing grade at Alvarado Street, 50 feet below grade at the eastern portion of the Station, and 63 feet below grade in the vicinity of the crossover track and traction power substation. After the Station is constructed, roughly 5 to 12 feet of fill will be placed above the Station structure, and about 17 to 37 feet of fill will be placed above the cross over track and power substation structure.

It is understood that the Station structure will be designed as a rigid reinforced concrete box with interior columns. At the time this report was being prepared, design loads had not been developed.

3.4 PROPOSED TUNNEL LINE

The ±20 foot diameter tunnel will exit the 7th/Flower Street Station at a depth of invert of about 50 feet below the ground suface and continue to the end of the Design Unit at depths of invert ranging from 60 to 100 feet below the ground surface; except at the Wilshire/Alvarado Station where the tunnel invert is on the order of 40 feet below the ground surface. Mixed-face conditions, i.e., alluvium at the crown and bedrock at the invert, should be anticipated approximately between Stations 205 and 214 as the tunnel leaves 7th/Flower Station. The tunnel will pass beneath MacArthur Lake with little, if any, cover between the base of the lake and the crown of the tunnel between about Stations 267 and 273, thus requiring cut-and-cover construction from the Alvarado Station to the west end of the lake.

Although the tunnel line does not pass directly beneath the Hilton Hotel, if the base of the hotel was projected into the tunnel line section, there would be 5 feet of cover over the tunnel crown between approximately Stations 207 and 211. The Hilton Hotel is set back approximately 10 feet from the edge of 7th Street, and it is not known whether tiebacks extend horizontally into the proposed tunnel line.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The factual information presented in this report is based primarily upon field and laboratory investigations carried out in 1981 and 1983. This information was derived from field reconnaissance, borings, geologic reports and maps, groundwater measurements, field gas measurements, field geophysical surveys, groundwater quality tests, and laboratory tests on soil and rock samples. The references noted on Page 3 and at the end of this report were utilized to compliment and supplement the more recent information.

4.2 BORINGS

Six borings, 11-1 through 11-6, were drilled at the station site as part of the 1983 investigation and all were 75 feet deep. In addition, one boring (CEG-11), drilled during the initial 1981 investigation, is located near the proposed Station on the southeast corner of Wilshire Boulevard and Alvarado Street. The locations of the borings are shown on Drawing 2 and the logs of the borings drilled for both the 1981 and 1983 investigations are provided in Appendix A. Groundwater observation wells were installed in Borings 11-1 and 11-5. Section 5.3 presents a summary of groundwater level measurements in these wells. Detailed descriptions of the field procedures are also presented in Appendix A.

In 1962, Kaiser Engineers drilled 20 borings (Boring Nos. 76-96, inclusive), spaced about 500 feet apart, ranging from 50 to 80 feet deep at the locations shown on Drawings 2, 3 and 4. Kaiser borings 76-86, inclusive, range from 100 to 600 feet from the center line of the present Metro Rail Project alignment, and were not used for interpretation of subsurface conditions. Kaiser Borings 88-96 are on the present Metro Rail Project alignment and were used to interpret the depth of soil overlying the bedrock but were not used for groundwater conditions. The Kaiser Boring Logs can be examined at the Southern California Rapid Transit District's 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 provided by the U.S. Geological Survey in their paper entitled, "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF-866, 1977). All of the Kaiser Engineering borings are tabulated in this report, as well as several other shallow borings performed for foundation investigations along or near the proposed alignment. The foundation investigation borings, included in the USGS report, are not shown on Drawings 2, 3 and 4, and were not used for interpretation of subsurface conditions as they were too shallow for proper interpretation of subsurface materials and groundwater conditions along the proposed grade of the Metro Rail tunnel.

4.3 GEOPHYSICAL MEASUREMENTS

Down-hole and cross-hole compression and shear wave velocity surveys were performed in Boring CEG-11 which was drilled during the initial 1981 investigation. This boring is located about 200 feet east from the proposed Station structure (see Drawing 2). Appendix C summarizes the field procedures used in making the surveys as well as the results of the velocity measurements.

4.4 OIL AND GAS ANALYSES

Gasoline, sulfur and petroleum odors were noted in Borings 11-1, 11-2, 11-4 and 11-6. Organic-sulfurous odors were also noted in Boring CEG-11 while drilling at relatively shallow depths, and large gas bubbles and tar were observed on the surface within the drilling fluid when the bottom of the hole was at depths greater than 150 feet. Bubbles of gas were not observed in the drilling fluid of the boreholes drilled as part of the 1983 investigation.

The Los Angeles City Oil Field is located about 2,000 feet north of the proposed Alvarado/Wilshire Station site; the closest approach to the tunnel is about 500 feet (near Station 194). There is no known history of regional subsidence due to this oil field. As was discussed in the 1981 Geotechnical Report, this field was discovered in 1892 and produced more than a million barrels of oil per year for a few years. The oil field contains shallow accumulations of petroleum, surface seeps, and more than 1,250 wells, only 54 of which were active in 1974. Most of the wells, drilled prior to 1900, were not surveyed or accurately located and the ground surface has since been developed. Consequently, accurate records of the well locations do not exist.

4.5 WATER QUALITY ANALYSES

Chemical analyses and selected parameters of sampled water obtained in Boring CEG-11 were performed as part of the 1981 geotechnical investigation. An artesian water condition was noted in this boring when it was advanced to a depth of 179 feet. The water that flowed out of the hole the day after its completion was sampled and subsequently analyzed. The chemical analyses and the results of these tests are summarized in Appendix E, which indicate poor water quality.

4.6 GEOTECHNICAL LABORATORY TESTING

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

Appendix E summarizes the testing procedures and presents the detailed results from the 1983 program. Appendix F also presents in summary form, the results of the 1981 laboratory program.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 TUNNEL LINE CONDITIONS

The tunnel line in Design Unit A-170 goes from Station 205± (west end of 7th/Flower Station) to Station 309± (Wilshire/Vermont Station). Generalized geological interpretations of subsurface conditions along the proposed route area presented on Drawings 2, 3 and 4.

About 90% of the tunnel line will be in weak bedrock of the Puente and Fernando Formations; 5% of the tunnel will be in old fine-grained Alluvium (Unit A_4) at the 7th/Flower Station area and Wilshire/Alvarado Station area (see Drawings 2 and 3), and another 5% of the alignment will pass beneath MacArthur Park Lake. A general description of the expected tunnel line geologic units follows.

- <u>A_4 - Old Alluvium (fine-grained)</u> - Old Alluvium, designated A_4 on Drawings 2, 3 and 4, consists of clayey silts and sandy silts but includes some clays, sandy clays and clayey sands. The materials are primarily stiff, but range from firm to hard consistency. Although boulders were not encountered in the borings, a few boulders are believed present because of the distant downstream location from the Santa Monica Mountains.

- <u>C - Fernando and Puente Formations</u> - Bedrock in both formations consist of well stratified claystone and siltstone with interbeds of sandstone, and are designated in the profile on Drawings 2, 3 and 4 as Unit C. Both materials are classified as "soft-ground" tunneling materials. The Fernando and Puente Formations are often referred to as "bedrock" or "rock" in various other publications and at places within this report, but they have engineering properties of hard, dense soils with significant cohesive strength. Hence, these formations are classified as "soil-like" bedrock or "soft-ground." The tunnel line will pass through the Puente/Fernando Formation contact at about Station 256 (just before entering the Wilshire/Alvarado Station open excavation). However, composition of the materials are so similar that their effects will be very similar in either formation.

  Locally, both formations contain very hard sandstone beds ranging from less than 1 inch up to 3 feet in thickness, with an estimated unconfined compressive strength ranging from 5,000 to 15,000 psi. These hard beds are estimated to comprise less than 1% of the Fernando Formation and less than 2% of the Puente Formation.

  The edge of MacArthur Park Lake, which is located in MacArthur Park, is approximately 230 feet west of the western portion of the proposed station excavation (see Drawing 3). According to information about MacArthur Park Lake obtained from the Los Angeles City Department of Parks and Recreation:

1) The lake is unlined and the bottom is in bedrock siltstone.

2) There is no evidence of water lost to vertical percolation; water losses are believed to be strictly evaporation.

3) The lake was drained about 3 years ago and the east half of the lake was cleaned out. The lowest elevation contour on the lake bottom, prior to cleanout, was 234 feet.

4) During the cleanout process, channels, up to 11 feet deep, filled with very soft, saturated muck were measured with a probe.

5) Based on this information the bottom of the deepest channel is about Elevation 223 feet.

Based on the MacArthur Lake information, the tunnel crown in the vicinity of Stations 262 to 273 is likely to encounter these channels, resulting in flowing ground at the face and caving at the surface. Therefore, we recommend cut-and-cover construction from the Alvarado Station to the west end of MacArthur Park Lake. The lake water surface is at about Elevation 255. As was previously noted, the water level recorded in the well installed in Boring 11-1 was 7 feet below the ground surface or at Elevation 258 (approximately). It is possible that the groundwater which is perched on top of the Puente Formation bedrock may be in hydraulic contact with the water in the MacArthur Park Lake. The City of Los Angeles, Department of Parks and Recreation (personal communication, September 1983) reported that basements adjacent to MacArthur Lake require pumping when the lake is full. However, pumping is not required when the lake is drained (empty).

Gasoline, sulfur and petroleum odors were detected in bedrock samples from Boring CEG-10, 11 and 12 as well as Alvarado Station Borings 11-1, 11-2, 11-4 and 11-6. No free flowing petroleum was observed. The likelihood of gas issuing from the bedrock formation is a distinct likelihood throughout Design Unit A170.

Mixed faced conditions, that is Old Alluvium and soft bedrock contacts, should be anticipated roughly between Stations 205 and 214 (near 7th/Flower Station), and again between Stations 263 and 275 (Wilshire/Alvarado Station to MacArthur Lake).

MacArthur Park fault intersects the tunnel line at approximately Station 269; it is not known to be active and will probably pose no particular tunneling problem. The east-west trending Los Angeles City Oil Field is located about 3,000 feet north of the tunnel line near 7th/Flower Station and is only 600 feet north of the tunnel line near the Vermont Station. The approximate areal distribution of the Los Angeles City Oil Field is shown on Drawing 1, "Geologic Map" in Volume 1, Geotechnical Investigation Report, November 1981.

5.2 STATION AND CROSSOVER CONDITIONS

Drawing 3 shows generalized subsurface cross sections through the proposed Wilshire/Alvarado Station. In general, the subsurface conditions at the site consist of 8 to 28 feet of Old Alluvium overlying claystones, siltstones, and interbedded sandstones of the Puente Formation. Groundwater levels measured approximately 1½ months following the completion of drilling indicated water at a depth of approximately 7 feet at the west end, and at 9 feet at the east end of proposed Station structure. An artesian water condition was noted in boring CEG-11 which was drilled at the corner of Wilshire and Alvarado during the 1981 investigation. As noted in Section 5.6, chemical analysis of the sampled water suggest that it is probably oil field brine which has infiltrated the Puente Formation bedrock at depth. The groundwater that was encountered in the Old Alluvium, however, is believed to be perched over the Puente Formation bedrock. This water may be in hydraulic contact with the water present in MacArthur Park Lake. Based on a bottom excavation elevation of about 224, the proposed excavation would extend through the Old Alluvium and approximately 10 to 40 feet of the Puente Formation bedrock.

5.2.1 Old Alluvium (A_4)

Based on the logs of the boreholes drilled at the station site, the Old Alluvium consists primarily of silts, clays, clayey sands and silty sands. Standard penetration resistances recorded through this unit range from 3 to 60 blows per foot with typical values being about 10 to 15 blows per foot. The consistency of the cohesive materials varies from soft to firm, and the relative compaction of the cohesionless materials from loose to medium dense (compact). Bedding is dominantly massive with some thickly-bedded zones being approximately 1 to 3 feet thick.

5.2.2 Bedrock (C)

Bedrock in the vicinity of the station varies from laminated to thinly-bedded siltstone, claystone, and sandstone to massive siltstone/claystone, both of the Puente Formation. Bedding generally dips southerly from 50-55 degrees, with the strike generally parallel to the station alignment. Occasionally thick, weakly cemented zones were encountered as well as smaller 2 to 3 inches thick well cemented zones or concretions. A weathered zone ranging from 5 to 8 feet thick was encountered in the borings near the center of the station (Borings 11-3 and 11-4). However, this zone was not encountered to the east, away from the lake.

Fracturing in the interbedded materials occurred primarily along sand lenses. The more thickly bedded materials generally appeared to be massive to little fractured.

The bedrock surface under the eastern half of the station appears to undulate with a general slope to the southwest, vaguely following the ground surface. At the northwest end of the Station, bedrock appears to be nearer the surface and drops fairly steeply to the southwest. The two borings at the northwest end (Borings 11-1 and 11-2) exhibit both the shallowest and deepest bedrock contact encountered.

5.3 GROUNDWATER

Groundwater levels measured in piezometers installed within the proposed Wilshire/Alvarado Station and Design Unit A170 tunnel line are tabulated below.

GROUNDWATER ELEVATION (ft)

Boring	3/07/81	6/17/81	4/28/82	4/04/83	4/27/83	6/08/83	9/02/83
CEG-9	214	213	212				216
*9-1				245	245	245	244
*9-4				204	202	202	205
CEG-10	293	295	296				294
*11-1				259	258	257	257
*11-5				277	278	278	277
CEG-12	230	229	228				228
*13-1				234	235	236	232
*13-6				241	241	241	241

*Borings not drilled until after February 1983.

As a subnote, Kaiser Engineers Boring Nos. 76-96, inclusive, drilled by Raymond Concrete Pile (see Drawings 2, 3 and 4), contained water when measured two months after drilling in 1962, but

> "were actually only damp to moist when drilled and were completed in the <u>dry</u>. The water found in the hole was the result of slow percolation through the sidewalls" (Kaiser, 1962).

It appears that the groundwater measured in Borings 11-1 and 11-5 originates within the Old Alluvium and is perched above the Puente Formation bedrock. The perched groundwater roughly follows the ground surface and is at a depth of 7 feet at the west end of the proposed station structure and about 9 feet deep at the east end. The regional groundwater table probably lies within the Puente Formation at depths in excess of 100 feet.

As was previously noted, an artesian water condition was encountered in Boring CEG-11 which was drilled as part of the 1981 geotechnical investigation. Chemical analyses of the water sampled from Boring CEG-11 suggest that the source of the artesian water was probably oil field brine originating in the depth interval of 62-179 feet. Since artesian water conditions were not observed in any of the borings drilled as part of this investigation, it is likely that the brine has probably infiltrated the Puente Formation bedrock at depths greater than 75 feet, which is the maximum depth explored by the boreholes drilled as part of the 1983 investigation.

In general, the quality of the water samples taken from Boring CEG-11 was very poor and contained total dissolved solids (TDS) exceeding 19,000 parts per million. The sample also had a high sodium chloride, or salt, content of 18,000 ppm. A sodium chloride content exceeding 1,000 ppm is considered high and water having this level can easily corrode metals used in construction. This "salty", sodium chloride-type water is judged to be oil field brine which has infiltrated upward through the Puente Formation bedrock at depth, and probably originates from the Los Angeles City Oil Field. Sulfate content of the water sample was about 5 ppm.

Water samples have not been taken from the piezometers installed during the 1983 investigation. It should be noted, however, that artesian water conditions were not encountered in any of the borings drilled at the proposed station location during this investigation. It is our judgment that the quality of the water present in these boreholes is probably much better than that of the water samples taken from Boring CEG-11 (i.e., TDS is closer to 1000 ppm than 19,000 ppm).

5.4 GAS

Attempts were made to obtain a sample of the gases that were bubbling out of Boring CEG-11 during the 1981 geotechnical investigation. Sampling, however, was complicated by the artesian water condition present in this hole and gas chromatographic analysis performed on the sample that was obtained was inconclusive as to the amounts of hazardous gases present. Appendix C describes the sampling and testing procedures and results of the analysis. A gas "sniffer" was used during the drilling operation to measure the amount of combustible gas present in the bubbles coming out of the drilling fluid. Measurements from this device indicated that the gases contained about 2 to 15% combustible gas, however, this should be considered only a "rough" measurement.

The strong odors present in the soil samples obtained during this investigation, and the bubbles of gas observed in Boring CEG-11 suggest that hazardous gases may be encountered during the excavation of the station site and proper precautions should be taken to avoid the potential hazards.

5.5 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.5.1 General

For purposes of the engineering evaluations, we have grouped the subsurface materials into two primary units which are fine-grained Old Alluvium, and bedrock. Pertinent laboratory test data for these units are presented in Appendix E. The laboratory data were evaluated along with field data (Appendices A and B), data from other investigations, and published data to establish the recommended material properties for use in analyses and design. The following presents a brief description of each geologic unit and discussions of the recommended material properties which are summarized in Table 5-1. Engineering judgment was also exercised in selection of the recommended material properties.

5.5.2 Old Alluvium (A_4)

The Old Alluvium encountered consists primarily of silts, clays, clayey sands and silty sands. The materials vary in compaction between loose and soft to medium dense and stiff. Effective and total strength parameters were established from the results of direct shear tests and triaxial compression tests with pore pressure measurements, together with consideration of published data for similar materials and experience with similar soils on other projects in the Los Angeles area.

Elastic properties were based on the laboratory triaxial test data, published data and engineering judgment. The initial tangent modulus data summarized in Drawing No. E-2 of Appendix E indicate that the modulus is essentially constant over the range of applied stresses.

5.5.3 Puente Formation Bedrock (C)

The Wilshire/Alvarado Station and Crossover Structure will be founded in the Puente Formation bedrock. This material consists primarily of interbedded siltstones, claystones, and sandstones with occasional thick, weakly-cemented zones as well as smaller 2 to 3 inches thick well-cemented zones or concretions. A weathered zone ranging from 5 to 8 feet thick was also encountered in two of the boreholes drilled at the Station site.

The Puente Formation claystone and siltstone were considered to behave as a very stiff highly overconsolidated fine-grained soil for the purpose of our engineering evaluations. Based on the results of a limited number of high stress consolidation tests, performed on bedrock samples from one of the other station sites, the maximum past pressure of these materials may be on the order of 100 ksf.

It should be noted that the value listed in Table 5-1 for the average unconfined compressive strength of the Puente Formation bedrock corresponds to unweathered (fresh) bedrock. A significantly lower average value of about 1,500 psf was obtained from tests performed on 2 samples of weathered bedrock from Borings 11-3 and 11-4 and an average value of about 2,000 psf was obtained from tests performed on 3 unweathered bedrock samples from Boring 11-1 (see Appendix E). The lower unconfined strength exhibited by these 5 samples are probably the result of the moderately to intensely-fractured nature of the samples obtained and/or the presence of weak silt/siltstone bedding. Due to the nature of these materials, the unconfined strength is probably not a good parameter by which to judge the in situ strength of these materials, and are reported here for completeness only (see Appendix E, Table E-1).

Bedrock elastic properties were selected based on consideration of field performance data, laboratory test data and published information combined with engineering judgment. For this study, the highly overconsolidated bedrock material was considered to have no significant modulus increase

Table 5-1

Recommended Material Properties for Static Design

Soil Property		Old Alluvium	Puente Formation Bedrock
Unit Weight (pcf)	(moist)	125	120
	(saturated)	132	120
Effective Strength Parameters:			
ϕ'(degrees)		30	35
c'(psf)		0	0
Total Strength* Parameters:			
ϕ(degrees)		15	10
c(psf)		1000	5000
Unconfined Compressive Strength, q_u (psf)		3000	10,000**
Permeability (cm/sec),k		1×10^{-6} – 1×10^{-7} (clays)	1×10^{-7} – 1×10^{-8} (intact)
		1×10^{-4} – 1×10^{-5} (silty sands)	1×10^{-6} (including joints and pervious beds)
Poisson's Ratio, ν		0.40	0.35
Initial Tangent Modulus, E_i (psf)		5×10^5	2×10^6

*Total stress parameter should be used to determine the in situ undrained strength of the soil for use in a $\phi = 0$ type of analysis.

**Value based on tests performed on unweathered samples which were not fractured. Much lower values were obtained from tests performed on weathered bedrock samples or on samples which were intensely fractured or which contained weak silt/siltstone bedding (see Page 14, Section 5.5.3).

within the range of depth affected by the proposed station. The apparent variation of modulus values at low confining pressures indicated by the laboratory data may be due to several factors, including the effects of sample disturbance and sample expansion after in situ stresses were removed. Very little data on in situ modulus of the Puente/Fernando Formation bedrock are available. Heave monitoring data for an excavation on the order of 50 feet deep at the Equitable Life Building, 3435 Wilshire Boulevard (Evans, 1968) were obtained and evaluated to determine the average bedrock modulus consistent with the observed heave. The selected constant modulus value presented in Table 5-1 is consistent with the observed bedrock heave and laboratory measurements at higher confining pressures.

Section 6.0

Geotechnical Evaluations and Design Criteria

6.0 GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA

6.1 GENERAL

Geotechnical design criteria for design and construction of the Wilshire/Alvarado Station and Crossover Structure are provided in this section of the report. Design criteria for the tunnel line of Design Unit A170 are presented in Section 7.0. To the extent practical, the criteria have been generalized to consider various potential design and construction concepts. As the design is finalized and specific details are formulated, these geotechnical criteria may be subject to some revision.

In general, construction of the Station and Crossover will involve making a 43 to 63 foot excavation. The excavation will be through 8 to 28 feet of Old Alluvium deposits which consist of predominantly silts, clays and clayey sands, and up to 34 feet of weathered and/or unweathered interbedded siltstones, claystones and sandstones of the Puente Formation bedrock. The bottom of the excavation will be at about Elevation 224.

As was previously stated, groundwater appears to be perched on the Puente Formation bedrock, and is at a depth of 7 feet at the west end of the station and 9 feet at the east end. Based on information provided by the City of Los Angeles Department of Parks and Recreation (see Section 5.3), perched water is in hydraulic contact with the water in the MacArthur Park Lake. Therefore, for the purpose of design, it would be prudent to assume that the water levels measured in Boring 11-1, which is adjacent to MacArthur Park, may be influenced by the lake. Since the water in the lake is at approximately Elevation 255, the bottom of station excavation will be some 31 feet below the lake elevation at the west end of the station. At the east end of the Crossover Structure, the bottom of the excavation will be about 54 feet below the perched water level measured in Boring 11-5.

The primary geotechnical considerations at the Wilshire/Alvarado Station site include:

- o Construction dewatering and subsidence considerations.
- o Designing for permanent groundwater levels; i.e., completely water tight structure or provisions for an underdrain system.
- o Design and construction of the temporary shoring system and the permanent wall system.
- o Establishing magnitude and distribution of soil and water pressures acting on the permanent structure.

6.2 EXCAVATION DEWATERING

6.2.1 General

As discussed in Section 6.1, a zone of perched water exists within the Old Alluvium at the Station site. Assuming that the bottom of the excavation is at Elevation 224, the proposed excavation will extend through the perched water zone (as measured in 1981 and 1983). The temporary shoring system should be designed to minimize the problems associated with the perched groundwater.

The Old Alluvium at the Station site consists mainly of clayey material with some pockets and lenses of silt and sand. Thus, these materials should be relatively impermeable and should result in only minor groundwater inflows into the excavation during construction. We understand that contractors excavating deep shored basement excavations in the area have generally controlled drainage with sumps within the excavation supplemented, in some cases with wells or wellpoints. We believe that at this site the excavation can be adequately dewatered using sumps within the excavation. However, provisions should be made for handling temporary large inflows from local sandy zones within the alluvium which could contain trapped perched groundwater.

6.2.2 Criteria for Dewatering Systems

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

- The dewatering system should be installed and in operation for a sufficient period prior to the excavation reaching the level of static groundwater level to adequately drawdown the groundwater table.
- The system should adequately control groundwater inflow within the excavation so as to maintain a dry excavation.
- The contractor should be made responsible for disposing of well discharge. He should be made aware of the potential environmental and operational problems caused by noise, poor quality groundwater, strong gas odors from dissolved gases, and the possibility of pumping oil. He should be made responsible for resolving these potential problems. Alternatively, the contract could include provisions for payment if special procedures are required.
- The system must be designed to eliminate loss of ground from piping. The dewatering operations should be constantly monitored for evidence of piping.
- The system should be capable of continuous operation. Emergency power and backup pumps should be required to ensure continual excavation dewatering.

6.3 UNDERPINNING

The need to underpin existing structures adjacent to deep excavations depends on many factors including soil conditions, depth of excavation, type of structures and proximity to the excavation, type of shoring, and consequence of potential ground movements. Figure 6-1 presents general guidelines for assessing when underpinning should be considered.

Based on Figure 6-1 and the proximity of existing structures as shown on Drawings 2 and 3, underpinning generally does not appear to be required for the proposed excavation.

6.4 TEMPORARY SLOPED EXCAVATIONS AND SHORING

6.4.1 General

The required excavation for the Wilshire/Alvarado Station will extend 43-63 feet below the adjacent street level. Because there are no major structures located in close proximity to the proposed excavations, it is our judgment that underpinning will not be required. However, the excavation will require shoring due to the space restrictions, and to protect any existing adjacent structure. We understand that the shoring system will be chosen and designed by the contractor, in accordance with specified criteria and subject to the review and acceptance by the Metro Rail Transit Consultants.

In our opinion, the contractor will most likely propose a system consisting of drilled soldier piles with tiebacks or cross-lot bracing for lateral shoring support. In some areas he may propose partial sloped cuts. Thus, the design criteria presented in this section pertain to these specific shoring systems. Other systems may also be appropriate and should be considered by the contractor.

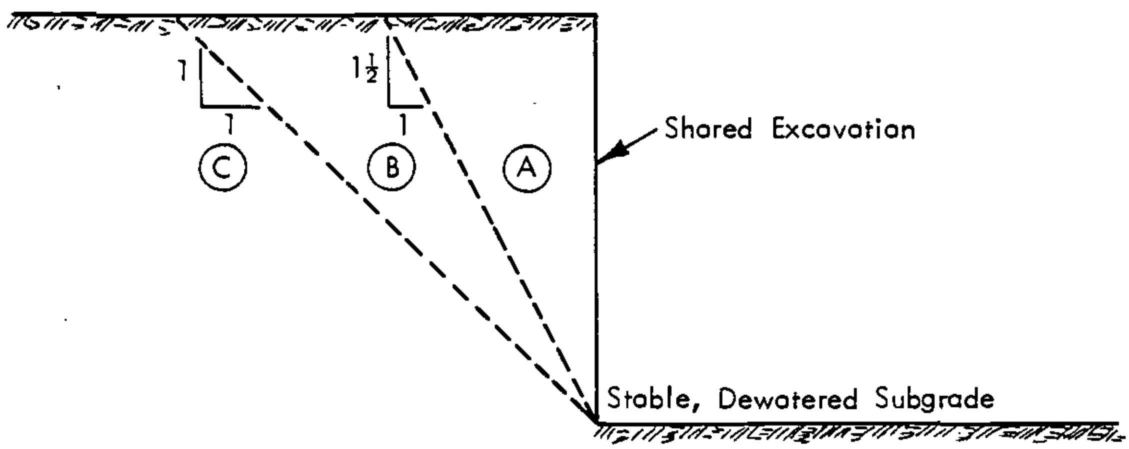
We believe that the contractor will not propose slurry walls or sheet piles. In our opinion slurry walls would be considerably more costly than a soldier pile and lagging system, and do not appear to offer any advantage at this site. Driven sheet piles are not considered feasible since the composition of the subsoils should make driving difficult if not impossible.

6.4.2 Sloped Excavations

Portions of the required excavation could be made with a sloped excavation, particularly the shallower cuts around the entry structures, and the cut-and-cover section through the lake. To minimize slope instability and potential loss of ground due to occurrences of perched water zones which may exist after dewatering the Station site, sloped excavations should be limited to depths above the static groundwater table. Sloped excavations extending to the static groundwater table at the Station site would reduce the height of the temporary shoring by about 8 feet.

Safe, stable construction slopes are normally the responsibility of the contractor and must be established in the field based on actual construction conditions. Factors which will influence determination of a safe, stable slope include soil conditions, groundwater conditions, the weather (i.e., dry or heavy rain), construction procedures and scheduling. Applicable governmental safety codes must also be complied with.

Based on previous experience in similar soils, temporary construction slopes of 1.5H:1V through the Old Alluvium above the groundwater table would probably be suitable. This assumes suitable site dewatering, no



NOTES: 1.) These guidelines are applicable only for stiff or dense stable ground conditions.
2.) 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

heavy loads at the top of the slope, protection of the slope surface and some slope maintenance. Flatter slopes may be required within the cut-and-cover section through the lake because of the overly saturated nature and lower strength of the near-surface materials. These observations should not be construed by the contractor to be a guaranteed permissible slope. The actual slope used by the contractor may be different.

6.4.3 Soldier Pile Shoring System

A soldier pile shoring system consisting of soldier piles installed in pre-drilled holes and braced with tiebacks is a common method of shoring deep excavations in the Los Angeles area. Appendix F.1 summarizes several case studies in the Los Angeles area involving soldier pile and tieback 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 site. Within the Old Alluvium, particularly below the groundwater table, caving and squeezing may be a problem. These conditions have been successfully resolved by maintaining a head of water or slurry in the hole.

The alluvial soils to be supported at the Alvarado/Wilshire Station 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. Gunite has also been used in areas where some soil arching between soldier piles allows time for guniting before soil sloughing occurs. The bedrock may not have to be lagged, although some surface treatment may be advisable to protect workers from spalling soil blocks.

6.4.4 Tiebacks and Internal Bracing

Tiebacks and/or internal struts are both suitable to support the temporary shoring wall for the proposed excavation. Tiebacks have the advantage of producing a clean, open excavation which can significantly simplify the excavation procedure and construction of the permanent structure. We believe that when the stability factor is large there is not a significant difference between the maximum movement of properly designed and constructed cross-lot braced walls and tieback walls. There may, however, be a difference in the distribution of the ground and wall movements. Generally, the maximum lateral movement of a tieback wall occurs near the top while with a cross-lot braced wall it occurs near the base of the excavation. These differences do not always occur as soil type, prestress loads and construction details can alter the distribution. With tiebacks, there is less incentive for the contractor to excavate a significant distance below the designated support level prior to installing the supports. Thus the tieback wall is less prone to the contractor excavating too far prior to installing supports (which can result in significant ground movement). This potential problem can be minimized with appropriate specifications and construction inspection.

Tiebacks may be subject to creep which could result in additional movements particularly if the excavation is to be open for an extended period. In our opinion, provided the anchors are conservatively designed, creep will not be a problem in the soils at the Station site. Struts are subject to stress variations due to temperature changes. This effect could be significant at the site due to the large temperature variations possible.

There are numerous types of tieback anchors available including large diameter straight shaft friction anchors, belled anchors, driven high pressure grouted anchors, driven high pressure regroutable anchors, and others. Generally in the Los Angeles area, high capacity straight shaft or belled anchors have been used where stable ground conditions prevail. Use of casing, slurry, or hollow stem auger may be required to maintain hole stability within portions of the Old Alluvium at this site. Driven anchors may be a feasible alternative in caving ground; however, this type of anchor is not normally used in the Los Angeles area and capacities would need to be verified by field testing. Performance criteria, particularly the need to minimize ground movement (due to either wall movement or loss of ground associated with anchor installation), can dictate the choice of anchor type.

6.4.5 Design Criteria

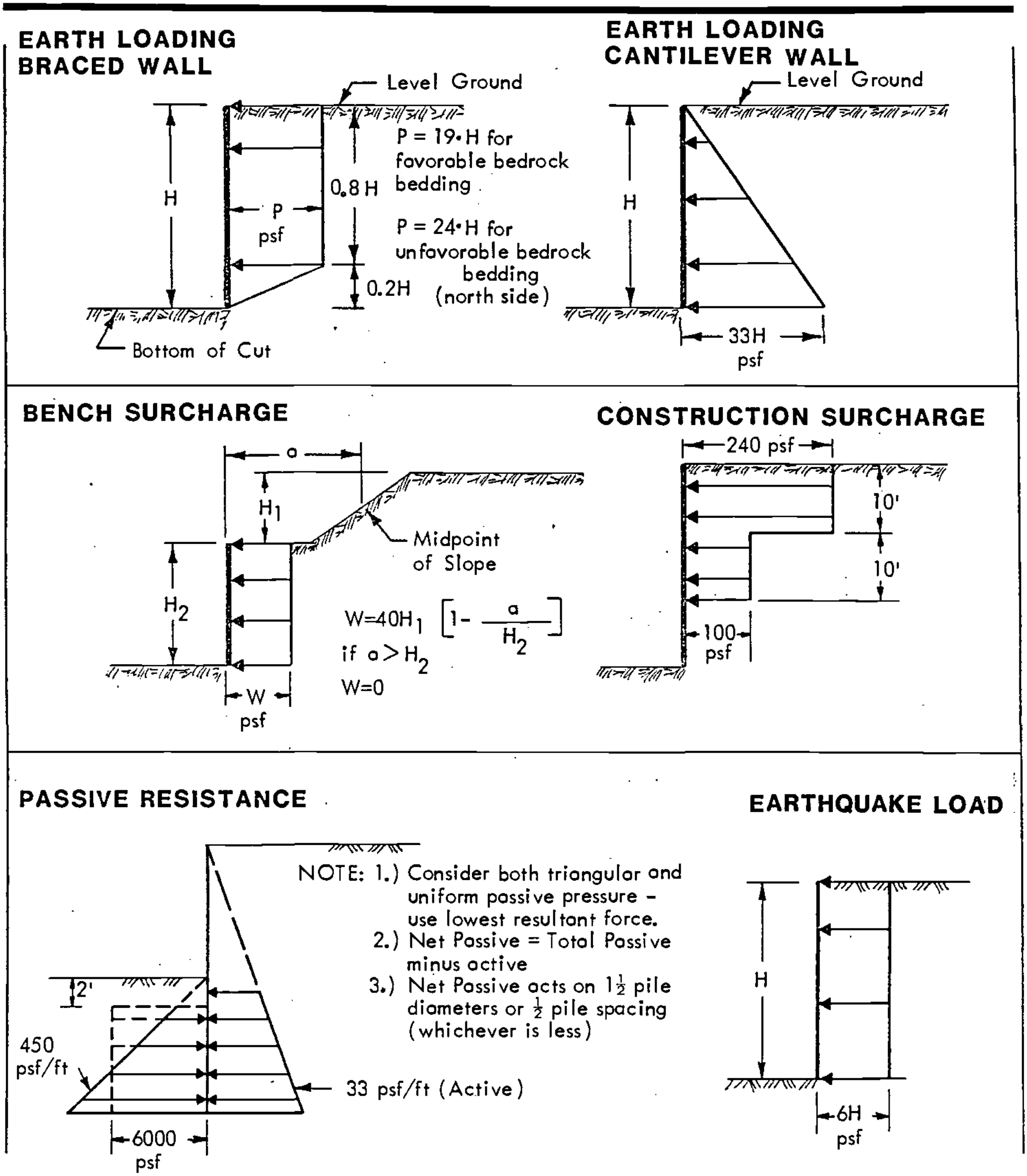
This section provides design criteria for a shoring system consisting of soldier piles, wooden lagging, tiebacks, and/or internal bracing. The soldier piles are assumed to consist of steel W or H-sections installed in predrilled circular holes. It is also the practice to fill the entire height of the predrilled hole with lean mix concrete. Thus, for computing the allowable soil support capacity, a circular concrete section has been assumed for the piles.

Specific shoring design criteria include:

- DESIGN WALL PRESSURE: Figure 6-2 presents the recommended lateral earth pressure on the temporary walls. It should be noted that increased shoring pressures are recommended for unfavorable bedrock bedding conditions. Unfavorable bedding is expected to occur primarily on the north side of the excavation due to generally south-dipping bedrock. 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. These reductions are based on consideration of arching action of the soil, and on experience and judgement.

  The recommended design static lateral earth pressures are based on standard design practices and experience in the Los Angeles area. The increase for adverse bedding conditions was based on consideration of the observed high angle bedding dip combined with our own experience and judgement. Appendix F.1 summarizes the design shoring pressures used on eight typical projects in the Los Angeles area. However, to our knowledge no actual field measurements have been made to ascertain lateral pressures acting on a shoring system in the Los Angeles area.

  Appendix F.2 and Section 6.11.3 provide technical support for the recommended design seismic pressures.



LATERAL LOADS ON TEMPORARY WALLS (WITH DEWATERING)

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Figure No. 6-2

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- DEPTH OF SOLDIER PILES: The depth of the soldier pile below the lowest anticipated excavation level must be sufficient to satisfy both the lateral and vertical loads. The vertical loads on tieback supported soldier piles can be substantial. In general the vertical load is equal to the total tieback tension on the pile times the sine of the anchor incline. Thus anchors installed at 20 degrees will develop a vertical load on the soldier pile equal to some 35% of the total anchor tension.

  The required depth of embedment to satisfy vertical loading should be computed based on both end bearing resistance and shaft friction of the soldier pile penetrating the bedrock formation. Allowable vertical loads for several typical pile diameters are given in Figure 6-3.

  The required depth of embedment to satisfy lateral loads should be computed based on the 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 times the actual diameter or half of the pile spacing (which ever is less). Figure 6-2 presents the recommended method to compute passive resistance.

- PILE SPACING AND LAGGING: The optimum pile spacing depends on several factors including soil loads, member sizes and costs, ability for soils to arch, and squeezing ground. At the Station site the upper soils will be primarily fine-grained but may also include some layers and/or pockets of sandy soils, which may contain zones not fully drained by the construction dewatering system. In addition, some localized zones of squeezing ground may be encountered within the Old Alluvium. For these reasons, it is recommended that the pile spacing be limited to about 8 feet, and that continuous lagging be placed through the alluvium to minimize ravelling and squeezing of soils, and loss of ground between soldier piles.

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

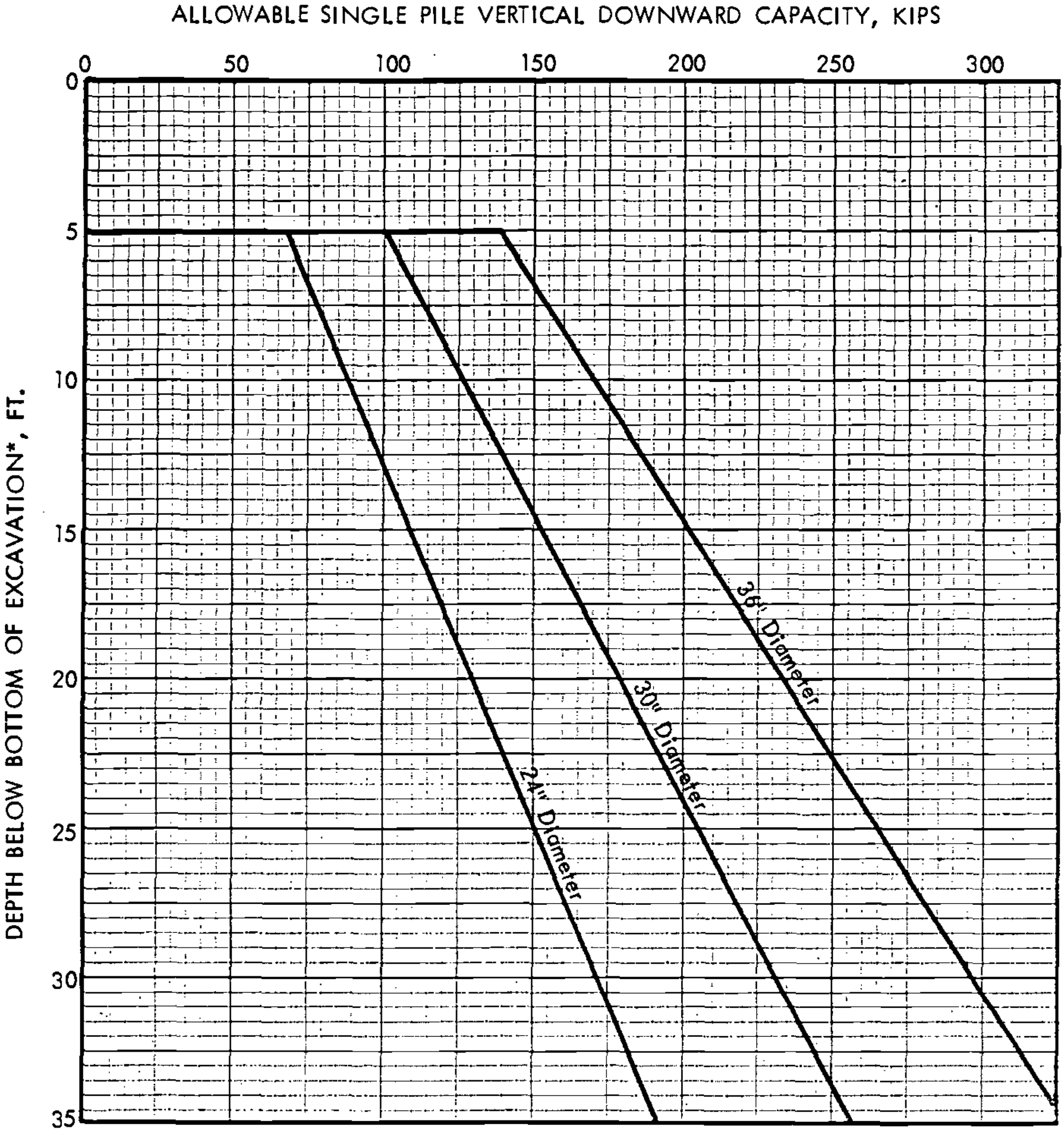
  $$P = \pi DLq \qquad \text{(anchor capacity)}$$

  where:

  P = allowable anchor load in pounds
  D = anchor diameter in feet
  L = anchor length beyond the no-load zone in feet
  q = average soil adhesion in psf

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

  $20d \leq 500$ psf, in Old Alluvium
  $20d \leq 1000$ psf, in bedrock



*Bottom of Excavation assumed to be within Puente Formation Bedrock

NOTE: 1) For seismic design, capacities may be increased 33% .

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VERTICAL CAPACITY OF PILES FOR SHORING & DECKING

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Project No.
83-1101

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Figure No.
6-3

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

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

The anchors may be installed at angles between 20 to 50 degrees below the horizontal. Based on specific site conditions, these limits could be expanded to avoid underground obstructions.

Structural concrete should be placed in the lower portion of the anchor up to the limit of the no-load zone. Placement of the anchor grout should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the no-load zone and the face of the shoring may be backfilled with sand 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.

Allowable anchor capacity for tieback types other than straight shaft friction anchors cannot be generalized. Capacity of anchors such as high pressure grouted anchors and high pressure regroutable anchors can be determined only in the field based on the results of test anchors.

- TIEBACK PRESTRESSING AND TESTING: It is recommended that each tieback anchor be load tested to 150% of the design load and then locked off at the design load. Under this load the anchor creep should not exceed 0.1 inch 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 the design load, the anchor creep should not exceed 0.15 inch over a 15-minute period.

- CROSS-LOT BRACING: The contractor should not be allowed to extend the excavation an excessive distance below a strut level prior to installing the next level of struts. The maximum allowable distance depends somewhat on the tolerances for ground movements. A maximum vertical distance between an installed strut level and the base of the excavation of about 15 feet may be appropriate.

  To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable. The shoring design and preload procedures must provide for the effects of temperature changes. Several methods should be considered, including:

- o Varying the preload stress depending on the temperature at the time of installation. The preload stress could be based on developing 50% of the design load at some designated average temperature assuming a non-yielding shoring wall. The assumption of a non-yielding wall to compute temperature-induced stresses is conservative and may warrant refinement to include the estimated soil stiffness (Chapman, 1972).

- o Providing a method of minimizing temperature variations such as covering the excavation, painting the struts with a reflective paint, cooling the struts with water or ice, and/or others.

- o Providing a method of measuring and adjusting the loads on the struts. The contractor could be required to maintain the struts within a specified stress range. A maximum stress of equal to the elastic limit of the strut with a minimum stress equal to 25% of the design load may be appropriate ranges. This method, although technically feasible, may be impractical in the field.

6.4.6 Anticipated Ground Movements

Appendix F.1 presents data on the performance of shoring systems in the Los Angeles area. Based on these completed projects, we believe that the maximum horizontal wall deflection will be equal to about 0.1% to 0.2% of the excavation depth. Thus for the proposed 43 to 63-foot deep excavation, the maximum wall deflection may approach about ½ to 1½ inches, depending on the construction procedures employed.

For a tieback system, the maximum horizontal movement should occur near the top of the wall and decrease with depth. Whereas, for a cross-lot bracing system the maximum horizontal movement generally occurs near the bottom of the excavation, decreasing to about 25% of the maximum at the top. The maximum vertical settlement behind the wall should be equal to about 50% to 100% of the maximum horizontal deflection and will probably occur at a distance behind the wall equal to 25% to 50% of the excavation depth.

6.5 SUPPORT OF TEMPORARY DECKING

We understand that temporary decking may be required within the portions of the excavation crossing Westlake Avenue and Bonnie Brae Street. At these locations it may be desirable to install piles in the center of the excavation to minimize the deck span. Piles would need to extend below the maximum proposed excavation level for support. At these depths, the piles would all be founded within the claystone and siltstone bedrock. These materials are suitable for supporting the required pile loads.

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that a structural steel pile installed in a predrilled hole to support the center decking would most likely be used. Accordingly, we have evaluated the allowable loads on these types of piles for several typical diameters. The recommended allowable design loads are presented in Figure 6-3. These values include both end bearing and shaft friction. The end bearing component includes a higher factor of safety due to the greater movement required to develop end bearing.

Due to the nature of the bedrock materials, we believe that driven piles may be difficult to install and will probably need to be predrilled to at least the bottom of the proposed excavation. In addition, driven piles may induce undesirable noise and vibration due to pile driving operations. Thus we believe that driven piles would probably not be used. Accordingly, we have not developed design loads for driven piles.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed excavation at the Wilshire/Alvarado Station should be instrumented to minimize 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:

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

- <u>Surface Survey Control</u>: It is recommended that several locations around the excavation and on any nearby structures be surveyed prior to any construction activity and then periodically monitored to detect vertical and horizontal movement to the nearest 0.01 feet. These should include points on the adjacent buildings and on top of the shoring wall (every fourth pile or a maximum distance of about 25 feet.) The monitoring program should continue until after all construction and backfill is complete at the site.

- <u>Inclinometers</u>: It is recommended that eight inclinometers be installed and monitored around the excavation. One inclinometer should be located on each side of the excavation at four locations along the excavation. The casing could be installed within the soldier pile holes or in separate holes immediately adjacent to the shoring wall. If a slurry wall is used, the inclinometer casing should be installed in separate boreholes outside the proposed excavation prior to digging the slurry trench. This would permit the performance of the wall to be monitored throughout the installation phase. The casing should extend at least 30 feet below the final excavation level to ensure base fixity. Baseline readings of the inclinometers should be made immediately upon installation. Subsequent readings should be made at regular intervals of excavation progress.

- Vertical Settlement Profiles: We recommend that four to six devices be installed to monitor the ground settlement pattern with depth around the excavation. There are several methods to obtain these data including a multi-point inductive coil settlement gage and vertical multi-point extensometers. In addition, subsurface vertical and lateral deformation data can be obtained within a single borehole by installing a special inductive coil system around the inclinometer casing.

- 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.

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

  The points should be installed and surveyed prior to starting excavation.

  Once the excavation begins, readings should be taken at regular intervals of excavation progress until the excavation is completed, and then at about two-week intervals until all heave has stopped.

- Additional Measurements of Strut Loads: 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.

- Gas and Oil Monitoring: The occurrence and concentration of gas and oil in the excavations and dewatering discharge should be monitored. It may also be prudent to install several shallow gas monitoring wells near adjacent structures to detect any potential increase in gas levels caused by site dewatering.

- 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.

- Supplementary Instrumentation: In addition to the above preplanned program, additional instrumentation may be appropriate during construction as a tool to aid in resolving specific construction concerns.

In our opinion, it is important that the installation and monitoring 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.

6.7 EXCAVATION HEAVE AND SETTLEMENT OF STRUCTURES

The excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed maximum 63-foot excavation will decrease the vertical ground stresses by about 5000 psf which will cause the soils below the excavation to heave upwards. Since the excavation will be open for an extended period, the heave is expected to be completed prior to constructing the Station and Crossover Structure. The structures and subsequent backfilling will reload the foundation soils. We estimate that the Station and Crossover loads will be about 4000 to 5000 psf including the weight of the backfill. This load will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil is approximately the same as the weight of the final structure, the structure will settle.

We estimate that the maximum heave at the center of the excavation will be on the order of 2 to 4 inches. We also believe that the majority of this will occur while the soil is being excavated. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (consolidation heave) within the alluvium and bedrock underlying the proposed excavation. Monitoring of the actual heave is recommended as discussed in Section 6.6.

Settlement on the order of 2 to 3 inches were computed due to the imposed loads from the structure and backfill. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/2 inch over half the structure width. These calculations are based on a uniform foundation bearing pressure which could only result from a uniformly loaded and perfectly flexible structure. We understand that the Station and Crossover will actually be quite rigid. Thus, the actual differential settlement will be less than the theoretical flexible foundation case.

As discussed above, we believe that the majority of the heave/settlement will be elastic. Thus the magnitude of the heave/settlement at any specific time during construction can be reasonably estimated by linear interpolation using the maximum heave/settlement presented above. As an example, the base slab, which is estimated to be about 6 feet thick, represents about 20% of the final soil loading. This load will cause about 20% of the maximum anticipated settlement to occur.

6.8 PERMANENT FOUNDATION SYSTEMS

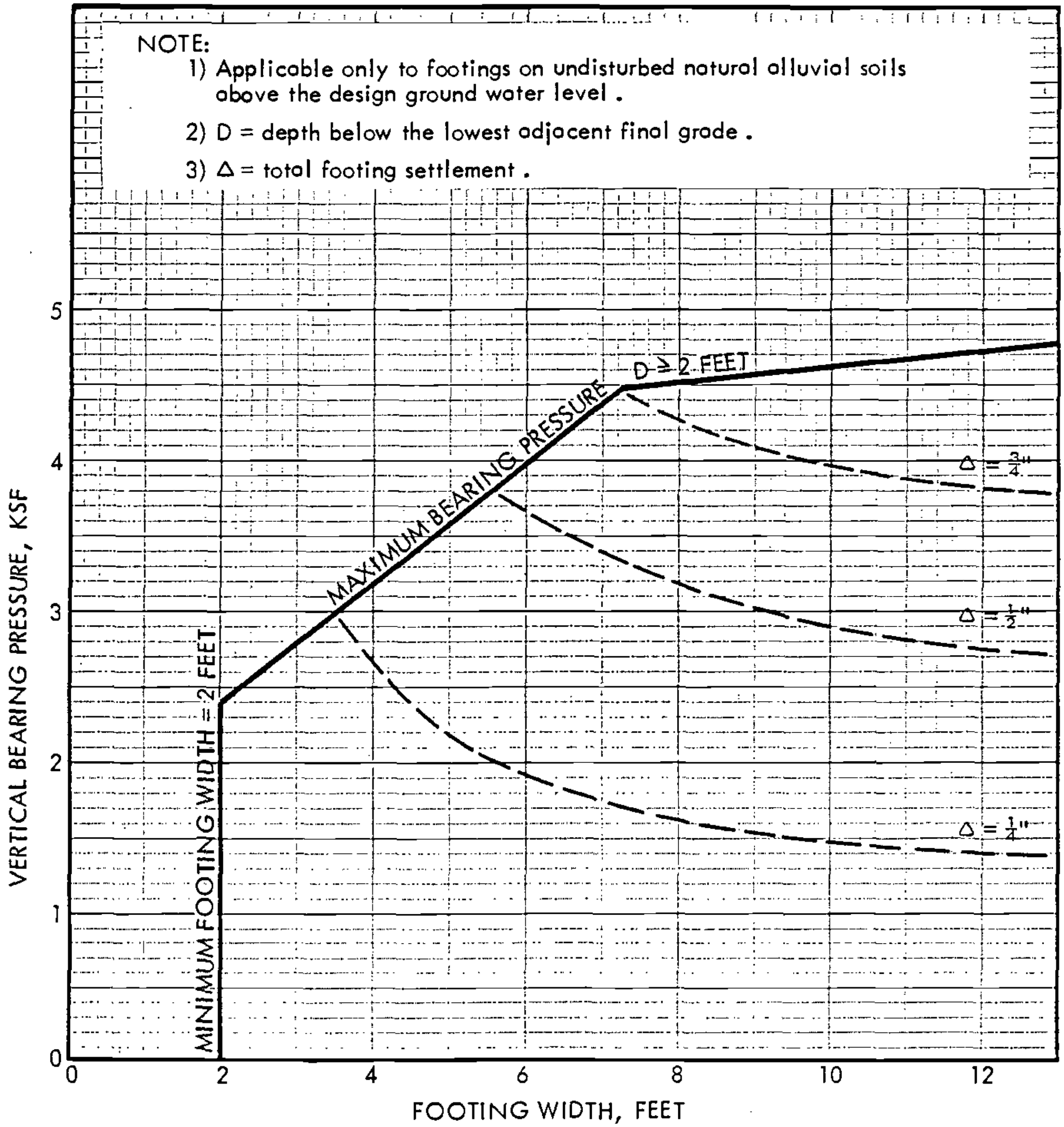
6.8.1 Main Station and Crossover Structures

It is understood that the proposed Wilshire/Alvarado Station and Crossover Structure will be supported on the lower slab which will function as a massive mat foundation. At the proposed foundation level, the mat will be bearing on the Puente Formation bedrock. We understand that the average foundation bearing pressure will be about 4000 to 5000 psf for both the Station and the Crossover Structure. After construction, the groundwater table will return to its original static level. Permanent hydrostatic uplift pressures acting on the mat foundation will range from about 2000 to 3000 psf. Thus, the permanent average bearing pressures on the soil may vary from 2000 to 3000 psf. In our opinion the Station and Crossover Structure can be adequately supported on the Puente Formation bedrock as indicated in the previous section.

6.8.2 Support of Surface Structures

Major surface structures may be supported on the firm natural Old Alluvium or the Puente Formation bedrock. However, where the Old Alluvium is loose or soft, and the bedrock is highly weathered, it may be necessary to excavate this material from below the structure and replace it with compacted engineered fill for structural support. Fill should be excavated from below the structures and backfilled with compacted structural fill for structural support. Figure 6-4 presents recommended maximum bearing pressures and anticipated settlements for footings bearing on either medium dense or firm alluvium or properly compacted structural fill. Figure 6-5 presents similar curves for footings bearing on the Puente Formation bedrock. These figures are based on analytical procedures and experience in the Los Angeles area. The values shown are for full dead load and frequently applied live load. For transient loads, including seismic and wind loads, the bearing values can be increased by 33% (one-third).

Minor lightly loaded structures (such as light standards, signs, etc.) may be supported on the natural Old Alluvium. The footing bearing area should



SPREAD FOOTING DESIGN ON FINE-GRAINED SOILS

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Figure No. 6-4

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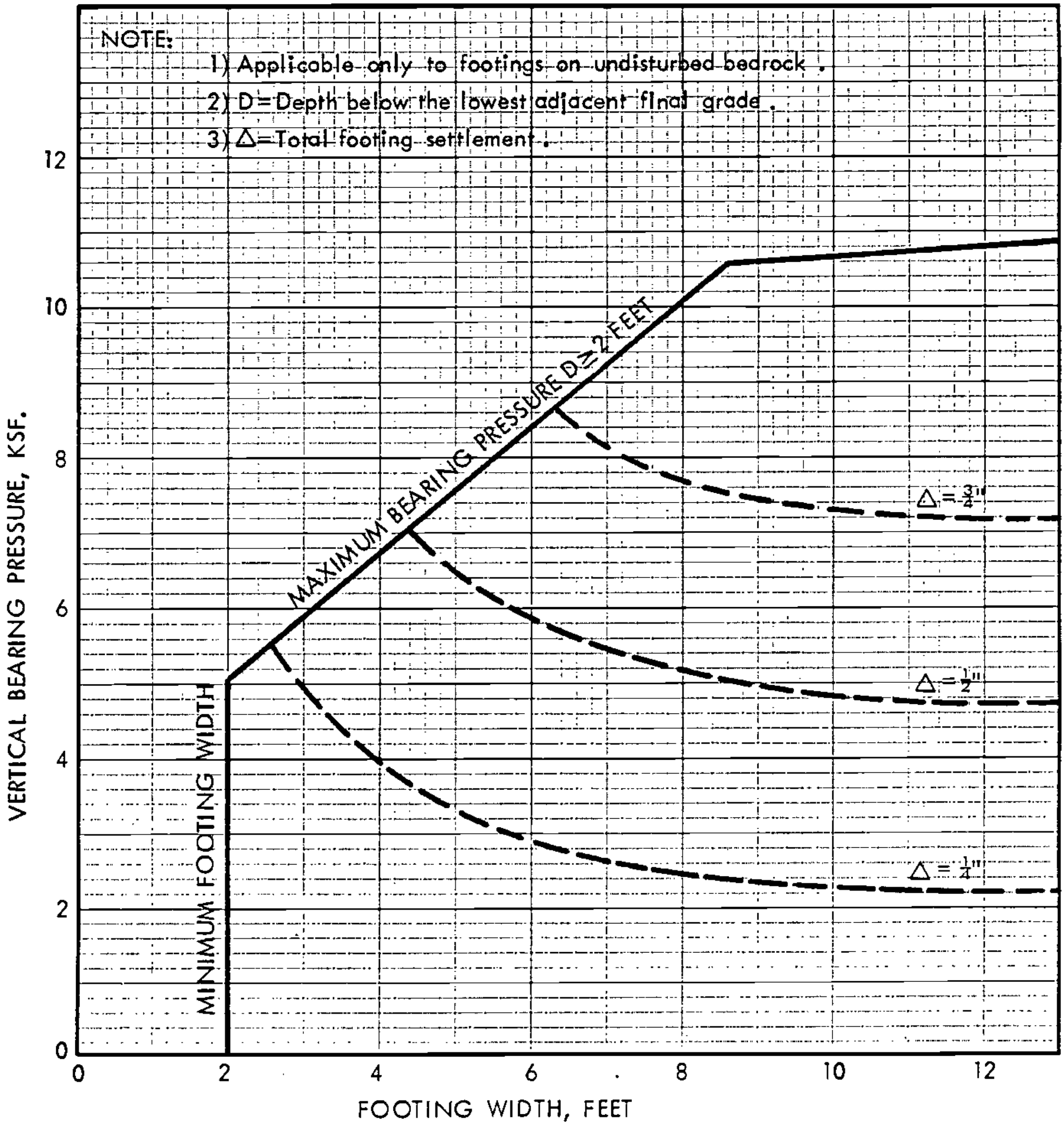

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SPREAD FOOTING DESIGN ON BEDROCK

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83-1101

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Figure No.
6-5

be overexcavated by at least 6 inches, thoroughly compacted and then backfilled to the footing level with compacted structural fill. These footings can be designed on the basis of 1500 psf vertical bearing pressure and should be at least 18 inches wide and 18 inches below the lowest adjacent finished grade.

For computing allowable lateral loads on foundation elements bearing on unsaturated alluvium or bedrock, passive pressure can be computed based on an equivalent fluid density of 450 pcf and a base friction value of 0.4.

6.9 PERMANENT GROUNDWATER CONSIDERATIONS

6.9.1 General

As discussed in Section 5, a perched groundwater feet condition was encountered in the Old Alluvium at a depth of approximately 8 feet below the ground surface. However, we believe that this does not represent a continuous condition although it is possible that the perched groundwater may be in hydraulic contact with the water in the McArthur Park Lake.

Once the Station is constructed and the excavation backfilled, the natural groundwater levels will be re-established. Even though the bedrock has a very low permeability, it is possible that eventually the groundwater levels around the excavation could rise to near the top of the original bedrock surface. In fact the Station could act as a "bathtub" and tend to collect groundwater. This could occur if the shoring wall provides a perimeter zone of higher permeability due to voids behind the shoring and/or placement of sand filler material. For design purposes, we estimate that the groundwater table could rise to about Elevation 260 at the West end of the Station, and about Elevation 280 at the east end of the Crossover Structure.

The permanent groundwater condition could be resolved by designing a water tight Station or by providing for a permanent drainage system. Conventionally, the deep basements in the area have been provided with permanent slab and wall drains draining to sumps. The pumping rates have been small due to the small permanent inflow rates. However, conventional practice for subway stations may be to provide complete water tight construction and design for the maximum hydrostatic pressures, since there is no guarantee that a permanent drainage system will be operational for the life of the facility.

General design criteria for a complete watertight system are presented below.

6.9.2 Complete Watertight System

The Station should be designed to be water tight below the level of the maximum anticipated groundwater elevation. Thus the permanent structure below this level will have to be designed to resist hydrostatic pressures.

Above the level of the maximum anticipated static groundwater level, it is recommended that some underdrainage be provided to drain potential accumulation of rainfall infiltration behind the walls. The details of this depend on the construction of the permanent wall. If the permanent wall is formed directly against the temporary wall, then a system including special fabric (such as Alidrains or equivalent) tied into a drain line inside the structure should be used. The fabric drains are attached directly to the wooden lagging and covered with plastic sheets to eliminate plugging with concrete. If the permanent wall is formed away from the shored wall, a drain should be installed at about the elevation of the static groundwater level and drained by gravity. The space between the permanent and temporary walls would then be backfilled with free draining granular backfill.

We also recommend that full waterproofing be carried at least 5 feet above the anticipated maximum groundwater levels.

6.10 STATIC LOADS ON PERMANENT SLAB AND WALLS

6.10.1 Hydrostatic Pressures

As discussed in Section 6.9, the station structures will likely be designed as a water tight section. It is recommended that for design the maximum groundwater levels be assumed equal to Elevation 260 at the west end of the Station, and Elevation 280 at the east end of the Crossover Structure. These elevations are based on judgement and are intended to provide for some increase in groundwater levels above the existing levels.

Maximum water level elevations for areas between the ends of the structures may be linearly interpolated.

6.10.2 Permanent Earth Pressures

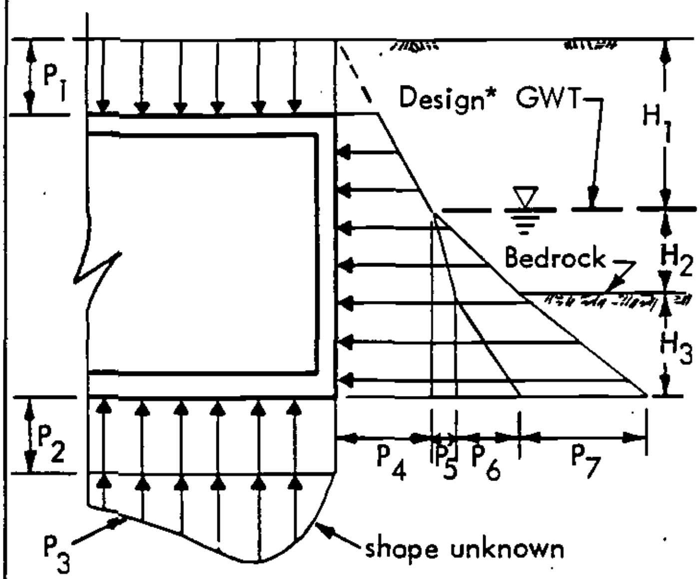
We recommend that the permanent static lateral earth pressures be based on the anticipated at-rest condition. For this condition, we recommend that the pressure be computed on the basis of an equivalent fluid with a density of 65 pcf above the groundwater table and 35 pcf below the groundwater table within the Old Alluvium. In the Puente Formation bedrock an equivalent fluid density of 55 pcf should be used above the groundwater and 30 pcf below the groundwater. Recommended soil and water pressures are presented in Figure 6-6.

The pressures on the roof should be assumed equal to the full weight of the overburden soil plus surcharge.

6.10.3 Surcharge Loads

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.

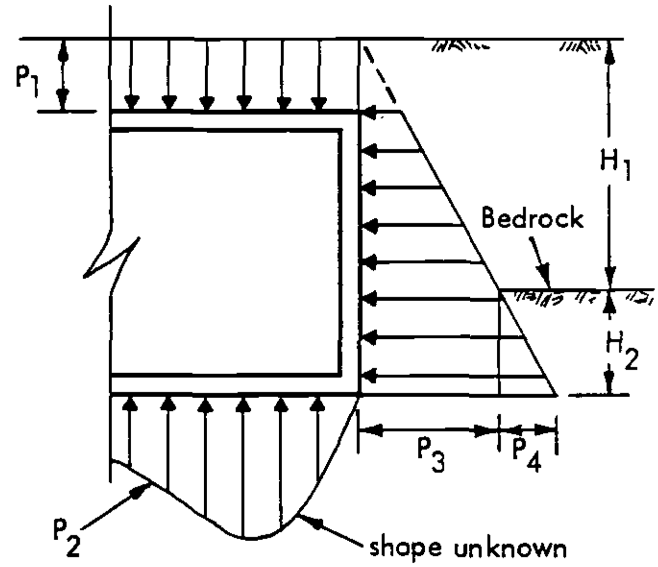
STATIC LOADS WITHOUT DRAINAGE



PRESSURE	MAGNITUDE
P_1	Overburden + Surcharge
P_2	$62.4\,(H_2 + H_3)$(psf)
P_3	Net Bearing Pressure
P_4	$65\,H_1$ (psf)
P_5	$35\,H_2$ (psf)
P_6	$30\,H_3$ (psf)
P_7	$62.4\,(H_2+H_3)$

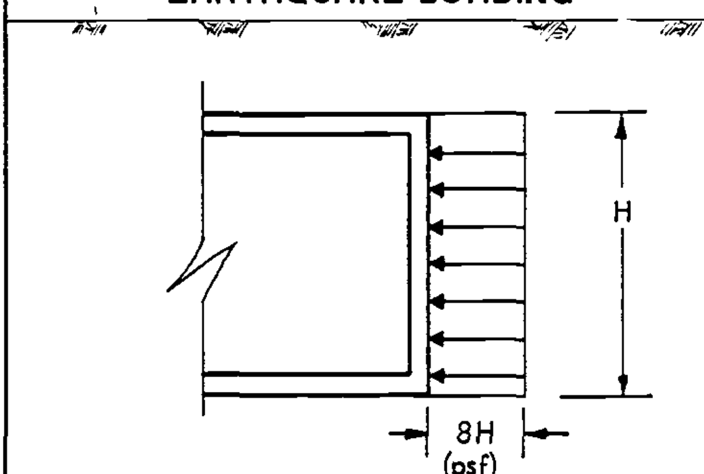
* Design ground water table (GWT) should be at elev. 260 feet at the west end of the station and 280 feet at the east end of the crossover structure.
Design GWT for intermediate points may be linearly interpolated.

STATIC LOADS WITH DRAINAGE



PRESSURE	MAGNITUDE
P_1	Overburden + Surcharge
P_2	Bearing Pressure
P_3	$65\,H_1$
P_4	$55\,H_2$

EARTHQUAKE LOADING



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LOADS ON PERMANENT WALLS AND SLABS

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Figure No. 6-6

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6.11 SEISMIC DESIGN CRITERIA

6.11.1 General

Detailed seismic criteria for design of the Southern Califonria Rapid Transit Metro Rail Project have been previously developed and are presented in the "Seismological Investigation and Design Criteria" report dated May 1983. The Part I investigation of this report contains an evaluation of the seismological conditions which may affect the project, and selection of a 100-year probable and maximum credible earthquake ground motions and response spectra for the project. The Part II investigation provides geotechnical and structural seismic design criteria to be used for design of both underground and above ground structures.

For design purposes two levels of earthquake ground shaking have been designated. The Operating Design Earthquake (ODE) corresponds to the level of ground shaking at which critical items maintain function so that the overall system will continue to operate normally. The Maximum Design Earthquake (MDE) defines the level of ground shaking at which critical items continue the function required to maintain public safety, preventing catastrophic failure and loss of life. Design ground motion parameters for these two earthquake levels are presented on Tables A-2 and A-3 of Part II, Appendix A, of the aforementioned report. Table A-3 gives values of displacements due to fault slip which must be accounted for in design at fault crossings. Design for fault displacement is required only for MDE conditions.

Elastic free field design response spectra for use as input in seismic analysis of structural response are given in Figures A-2 and A-3 of Part II, Appendix A of the seismic design criteria report.

Where time-history type of analysis is to be used the District will provide appropriate digitized records in the form of computer tapes or decks for ODE and MDE level events.

6.11.2 Dynamic Material Properties

Values of apparent wave propagation velocities for use in travelling wave analyses have been presented in Table B-2 of Part II, Appendix B of the seismic design criteria report. Other dynamic soil parameters will also be required for input into the various types of analyses recommended in the seismic design criteria report. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels. In addition, certain types of equivalent linear analyses require that the variation of dynamic shear modulus and soil hysteretic damping with the level of shear strain be known.

Average values of compression and shear wave velocities based on interpretation of limited downhole and crosshole geophysical surveys performed in Boring CEG-11 and other borings in similar materials during the 1981 investigation (see Appendix B) are presented at the top of Table 6-1. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish appropriate modulus values at low

Table 6-1

Recommended Dynamic Material Properties
for Seismic Design

	Old Alluvium	Puente Bedrock
Average Compression Wave Velocity, V_c (ft/sec)	5000	5700
Average Shear Wave Velocity, V_s (ft/sec)	1000	1300
*Poisson's Ratio	0.40	0.35
**Young's Modulus, E, (psi)	315,000 (moist) 185,000 (sat.)	530,000
**Constrained Modulus, E_c, (psi)	675,000 (moist) 710,000 (sat.)	850,000
**Shear Modulus, G_{max}, (psi)	27,600 (moist) 28,500 (sat.)	45,000

*For saturated Old Alluvium use value of 0.45.

**Saturated values of modulus should be used for undrained loading conditions in saturated Old Alluvium.

strain levels. Computed moduli values for the Old Alluvium and Puente bedrock are 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 6-7 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 6-8.

6.11.3 Seismic Lateral Earth Pressures

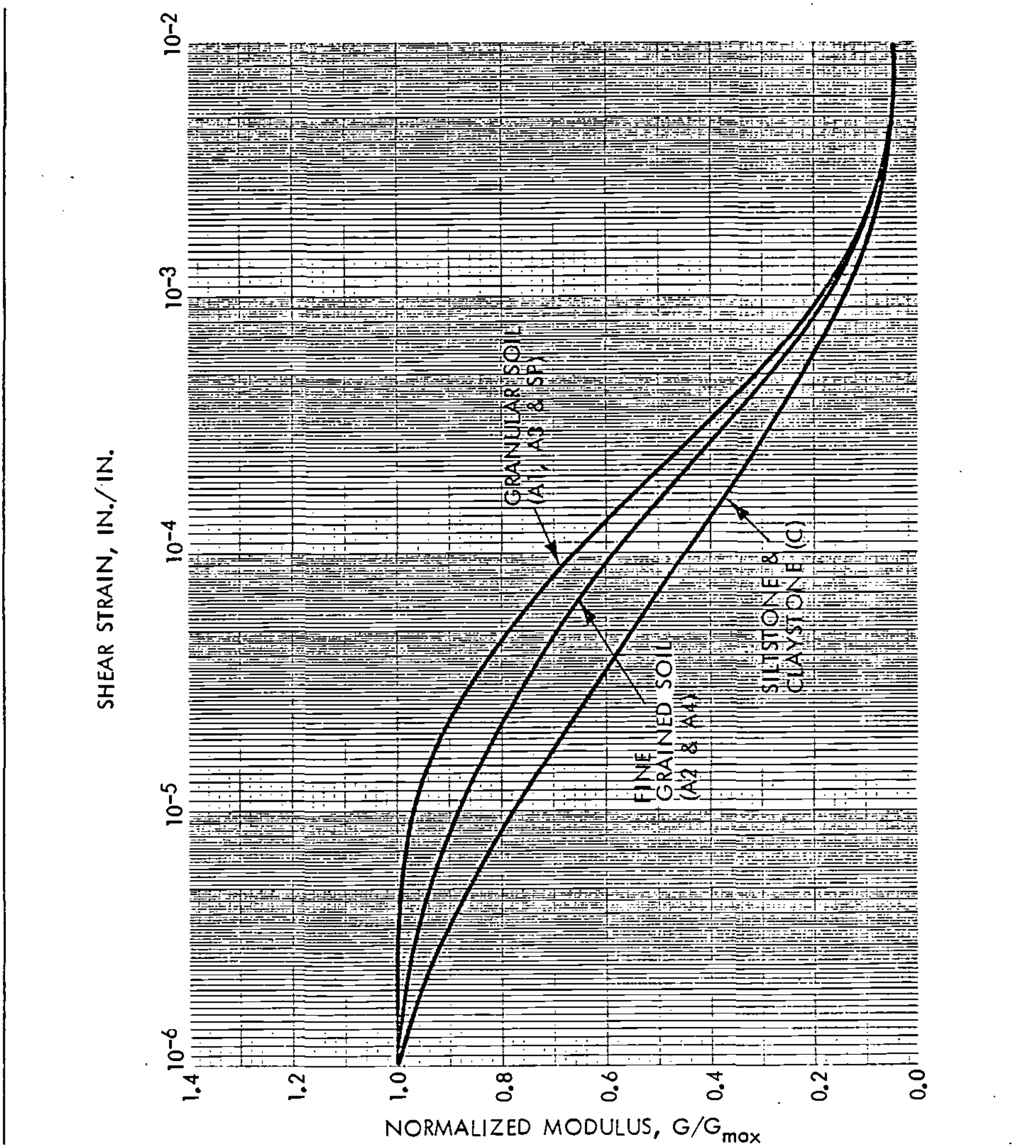
In Section 4.5.4.5 of Part II, Appendix A of the seismic design criteria report a discussion of the static and dynamic soil pressures which should be considered in the design of below grade walls to resist lateral earth pressures is presented. The procedure used to calculate seismic loadings on walls is based on the well known Monobe-Okabe formulation. For this study the analysis presented in Appendix F.2 should be used. This analysis is also based on the Monobe-Okabe formulation but includes various other assumptions based on previous experience and engineering judgment (see Appendix F.2).

Based on the analysis presented in Appendix F.2 the temporary shoring system should be designed for a uniform seismic lateral earth pressure equal to 6H as shown in Figure 6-2. The permanent wall should be designed for an equivalent uniform lateral earth pressure equal to 8H (see Figure 6-6). This value is based on a peak ground acceleration of 0.30 g corresponding to the Operating Design Earthquake (ODE).

6.11.4 Liquefaction Potential

As previously discussed, the Wilshire/Alvarado Station wil be founded on the Puente Formation bedrock. The excavation for the Station structure will extend into this formation to depths ranging from 15 to 38 feet. The overburden soils present at the site are approximately 8 to 28 feet thick and consist primarily of clays and clayey sands. Some pockets and lenses of silty sand appear to be interbedded within the clayey soils as indicated by the logs of the boreholes drilled at the Station site. Piezometers installed at the site indicate that water levels are approximately 8 feet below the ground surface. Therefore, it is likely that some of the silty sand lenses are saturated. Standard Penetration Tests (SPT) performed in the overburden soils are quite low and range from typically 3 to 17 blows per foot with most values in the range of 10 to 15 blows per foot. While most of the blow counts were recorded in the clayey soil deposits, it is prudent to assume that the silty sands present at the site probably also possess low SPT blow counts.

While liquefaction of some of the silty sand lenses present at the site is possible during the postulated earthquake, it is our judgment that because of their discontinuous nature and their limited occurrence, the hazard which would be posed by their liquefaction would be minimal and should be discounted for this study.



RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
6-7

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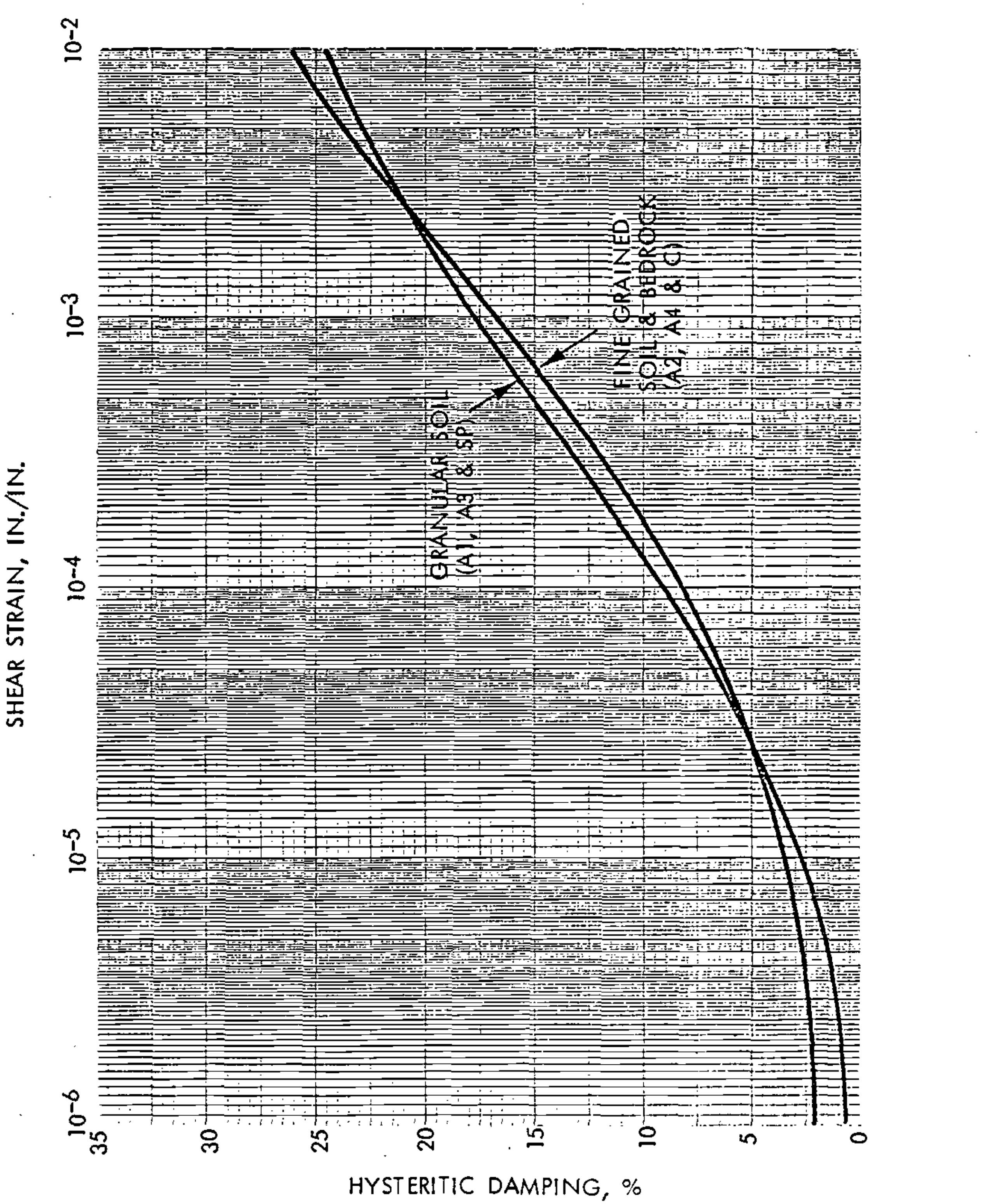

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No. 6-8

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6.12 EARTHWORK CRITERIA

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

Only excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. The excavated fine-grained and bedrock materials are not considered suitable due to their fine-grained nature which will make compaction difficult and could lead to fill settlement problems after construction. If the granular alluvium materials cannot be obtained in sufficient amounts, imported granular soils could be used for fill, subject to approval by the soils engineer.

6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- <u>Observation Well Monitoring</u>: The ground water observation wells, already installed in this study, should be read several times a year prior to construction and more frequently during construction if the wells can be maintained. These data will aid in confirming the maximum design ground water levels. It will also provide valuable data to the contractor in determining his construction schedule and procedures prior to construction and evaluating dewatering during construction.

- <u>Additional Oil and Gas Explorations</u>: The available data may be insufficient to evaluate adequately the anticipated problems associated with gas release during dewatering, occurrence of gas in the permanent structure, and the possibility of oil migrating to the pumps during dewatering.

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

- <u>Shoring Review</u>: 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 is not a certification of the contractor's plan but rather an independent review made with respect to the owner's interests.

- <u>Construction Observations</u>: A qualified geotechnical engineer should be on site full time during installation of the 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 recommended instrumentation data and respond to any specific geotechnical problems that may occur.

Section 7.0

Tunnel Alignment

Geotechnical Considerations and Recommendations

7.0 TUNNEL ALIGNMENT GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

7.1 GENERAL

Subsurface conditions along the Metro Rail Design Unit A170 Tunnel Line are judged favorable for mechanical excavation tunneling and at relatively high rates of advance, except in the MacArthur Park Lake area. We recommend utilizing economical pre-cast concrete segments to form both initial support and permanent lining in one construction operation. There will be significant intervals with comparable ground conditions and thus comparable excavation characteristics. The tunnel line will pass through two soft-ground tunneling units. These are:

- Unit A_4 - Old Alluvium: Medium dense to very dense, unsaturated, primarily silt with silty clay, silty sand.
- Unit C - Fernando and Puente Formation: Weak to moderately strong rocks of low hardness but locally will contain very hard cemented sandsone interbeds from 1 inch to 3 feet in thickness (see Appendix F.3).

Based on the proposed line and grade, there will be unfavorable tunneling conditions in Unit A_4 beneath MacArthur Park Lake between about Stations 267 and 275. This is due to the shallow ground cover over the tunnel crown (see Drawing 3).

Squeezing of Unit C is not anticipated at the planned 40 to 100 foot tunnel invert depths. Unit C materials are hard enough (average unconfined compressive strength of about 10,000 psf) that squeezing should not be a particular stability problem in normal shielded TBM operations, nor should the ground pressures exceed the capacity of normal support systems.

Because of shifts in the strike and dip of the formation, it is possible to encounter a section of tunnel wherein a hard interbed may persist in the tunnel face for a considerable distance. In this respect, bedding plane dip information from borings are as follows: CEG-9 (Station 205) -10° to 30°; CEG-10 (Station 239) - 55°; CEG-11 (Station 265) - 40° to 50°; CEG-12 (Station 294) - 40° to 50° and CEG-13 (Station 321) - 25° to 45°. Thus, bedding in the claystone unit dip at angles below the horizontal between 10° and 55°, with average dips of 40° from the horizontal. Although the direction of dip cannot be determined from the borings, projections from surface outcrops suggest that the bedding is inclined to the south or southwest. Therefore, the strike of beds are likely to parallel the entire A170 tunnel alignment. Thus, hard, cemented beds that are encountered should incline from the upper right to the lower left side of the face and could persist for hundreds of feet.

Units A_4 and C are conducive to soft-ground tunneling methods, that is, a method wherein ground conditions are such that mechanical excavation methods can be used within a shield to advance the tunnel, and wherein a support system is required immediately behind the shield. Mechanical (TBM) methods include spade and claw backhoe-type diggers, roadheaders and wheel type (dragpick) machines. The density of the Old Alluvium lends itself very well to shield tunneling methods with mechanical excavators. Because of their density and strength, face stability is expected to be good. However, there may be local areas where groundwater control and face support will be needed for satisfactory ground control. The entire tunnel in Design Unit A170 is considered gassy.

Previous tunneling experience in the Los Angeles area, especially the Sacatella and Tanner tunnels, is presented in Appendix F.3 (1981 Geotechnical Study, Volume 1, Section 6.0).

Dewatering of Unit A_4 materials from the surface by pumping wells should be coordinated with dewatering open-excavations for the 7th/Flower and Wilshire/Alvarado Stations. Dewatering is not likely to create serious ground settlement because the materials are dense, fine-grained and the groundwater levels are being lowered no more than 30 feet below their present level.

Water seepage into the tunnel from fresh, unfaulted, slightly fractured, fine-grained Unit C would likely be of small amount, i.e., dripping conditions.

A support system will be required throughout most of the tunnel. We recommend installing an initial support system which will serve also as the permanent lining. A shield operation will require a strong initial support system that can serve as a buttress for jacking the shield forward. The most common and appropriate supports are pre-cast concrete segments. If proper seals are used to minimize water inflows, concrete segments could be both the initial support and permanent lining. Concrete segments would not, however, keep gas from entering, since concrete is approximately 80 times more pervious to gas than it is to water. A lining system to seal out gas will have to be designed for tunnel line Design Unit A170 which is considered gassy based on boring data. Expansion of the supports or timely and thorough backpacking and grouting of the annular void between the support segments and tunnel wall is recommended to prevent surface settlement, particularly where the tunnel is driven at a shallow depth. A shallow depth is considered 40 feet (two tunnel diameters) below the original ground surface.

The quality of groundwater in the tunnel line may be poor and could be corrosive, requiring special type of cement. For example, the total dissolved solids (TDS) in Boring CEG-11 is 19,670 ppm, of which 19,000 ppm was sodium chloride (NaCl) which apparently originated from the nearby Los Angeles Oil Field as brine. Calcium sulfate ($CaSO_4$) made up over 2,000 ppm of Boring CEG-10 water samples and the TDS was 4460 ppm. Conversely, water quality in Boring CEG-9 contained total dissolved solids of 485 ppm and was considered only slightly corrosive. The source of the poor quality water in Borings CEG 9, 10, and 11 may be from depths of 200 feet which is 100 feet deeper than the proposed depth of construction. Therefore, the quality of water at the proposed depth of construction may be much better than indicated by the reported chemical analyses.

Our recommendation for determining the magnitude of the loads acting on the tunnel lining is that the procedures presented by Peck (1969) and Deere and others (1969) be adopted for current design purposes to estimate the forces on the lining. These methods are based on field observations and basic geotechnical principles.

Tunneling conditions beneath MacArthur Park Lake may be impractical due to little, if any, ground cover above the crown (see Drawing 3). Therefore, cut-and-cover construction may be more practical and economical beneath the lake.

7.2 DEWATERING

Perched groundwater will be a consideration for tunnel line construction between Stations 207 to approximately 213 (exiting from the 7th/Flower Street Station), and exiting from the Wilshire/Alvarado Station between tunnel line Stations 263 to approximately 275 (just beyond MacArthur Park Lake). The lake water surface is at Elevation 255 feet and the lake bottom coincides with the tunnel crown. We recommend draining (dewatering) the lake in order to perform cut-and-cover construction between approximately Stations 263 and 275.

The dewatering program associated with the 7th/Flower Station and the Wilshire/Alvarado Station should ameliorate much if not all of the perched groundwater problems in the tunnel line sections adjacent to the Stations.

As shown on Drawing 2, the perched water is approximately a few feet above the proposed tunnel at the 7th/Flower Station. Between the 7th/Flower Station and the Wilshire/Alvarado Station the perched groundwater table lies 40 to 50 feet above the crown of the tunnel.

As shown on Drawings 3 and 4, the perched groundwater table is on the order of 30 to 60 feet above the crown of the tunnel west of the MacArthur Park Lake between approximately Stations 275 to 310 (entering the Wilshire/Vermont Station).

The permanent groundwater table, as shown on Drawings 2, 3 and 4, lies 30 to 100 feet below the tunnel invert, in the siltstone bedrock, for the full tunnel line section.

There appears to be hydraulic continuity between the bottom of the MacArthur Park Lake and the crown of the tunnel (see Section 5.3 and Drawing 3). Dewatering (draining) the lake prior to construction of the tunnel beneath it should help reduce seepage and running ground into the tunnels during excavation. In addition, the MacArthur Park fault underlies this area, and could possibly provide broken rock conditions several feet to tens of feet on either side of the fault which would permit surface water to migrate into the tunnel (see Drawing 3). This fault is not known to be active or potentially active. Neither the physical condition nor the width of the fault is known. Since the fault trace crosses the alignment at nearly a right angle, it would not follow excavations any great distance. Upon completion of Boring CEG-11 (Drawing 3) artesian flow of water at the surface occurred for about 48 hours ranging from 1 gallon a minute to less than 1/2 a gallon a minute and then stopped flowing. The water originated somewhere below a depth of 67 feet. This artesian condition may indicate the fault is a barrier to groundwater, as well as a trap for gas and oil.

The water was highly saline, containing 19,570 ppm total dissolved solids, suggesting an origin deep in the oil bearing Puente formation.

In summary, except for minor dewatering associated with the 7th/Flower Street and Wilshire/Alvarado Street Stations, as well as running ground and water inflows beneath MacArthur Park Lake, in our judgment, little more than nuisance seeps and dripping is anticipated from the Puente and Fernando bedrock formations between Stations 205 to 309 of the tunnel line except that section passing beneath MacArthur Lake.

7.3 TUNNEL CONSIDERATIONS AND RECOMMENDATIONS BY STATIONINGS

7.3.1 Station 205 (7th/Flower Station) to 214 - This should be a mixed-faced excavation with invert at Elevation 220 feet or about 45 to 60 feet below the original ground surface. Materials will consist of Unit A_4 -Old Alluvium near the crown and bedrock (Unit C) at the invert. The materials are saturated and the perched groundwater table is at approximately Elevation 225 feet. This means about 45 to 60 feet of saturated alluvial thickness overlies the tunnel crown, and we recommend dewatering to get below the invert elevation.

Water quality is judged to be poor and corrosive to cement and metal materials. More definitive information can be obtained from the MRTC(s) corrosion consultants, Water Consultants, located in San Diego, California.

7.3.2 Station 214 to 253 (Wilshire/Alvarado Station) - Materials will consist of Unit C - Puente and Fernando Formations. Unit C, basically claystone/siltstone, can be excavated by TBM.

Hard cemented sandstone beds should be anticipated occasionally between Stations 214 and 253 ranging from 2 inches to 3 feet in thickness. These conditions should be considered when selecting excavation equipment for Unit C. The hard, cemented sandstone interbeds have a compressive strength ranging from 5,000 to 15,000 psi. These beds were of sufficient hardness to cause bending of the shield cutting edge in the Sacatella Tunnel. The Sacatella Tunnel penetrated materials similar to Unit C and is located 1000 feet north of the intersection of Wilshire Blvd. and Hoover Street (see Appendix F.3).

Gas should be anticipated throughout the Design Unit A170. Gas (sulfur and petroleum odors) were detected in Boring CEG-10, 11 and 12 as well as Borings 11-1, 11-2, 11-4 and 11-6 at the Wilshire/Alvarado Station. As noted in Section 5.1 gas bubbled out of the Boring CEG-11 and a gas "sniffer" detected 2% to 15% combustible gas. The Los Angeles City Oil Field is about 2000 feet north of the tunnel and is another reason for anticipating gas.

Groundwater in Unit C is not judged to be a dewatering problem and should be limited to dripping conditions and local nuisance seepage.

Between Stations 214 and 253 no mapped faults are known to cross the tunnel alignment in Unit C. However, Unit C has been folded and faulted, as evidenced in surface outcrops, so it is likely that many small faults will be encountered in the tunnel. Ground conditions at small fault crossings are probably not significantly different from the conditions in the unfaulted sedimentary rocks of Unit C. Some faults may, however, provide localized sources of groundwater seepage, probable no more than a few gallons per minute, but provisions should be available to handle occasional small water flows and ravelling ground at the face. In addition, fault zones may contain altered materials with physical properties different from that of the wall rock unit, such as swelling and/or slaking characteristics.

The quality of water in Unit C is probably poorer than the quality of the groundwater in the overlying alluvium and bedrock. Therefore, the water could be corrosive to improperly designed concrete and metal lining systems.

7.3.3 Station 263 (Alvarado Station) to 309 (Vermont Station) -The tunnel will pass through Unit C, the entire length. Tunnel invert ranges from about 40 to 60 feet below the original ground surface (see Drawings 3 and 4). An exception is at MacArthur Park Lake (approximately Stations 265 to 273) where the crown of the tunnel is approximately at the bottom of the lake (mudline). Dewatering of the lake prior to cut-and-cover construction should reduce water inflows.

The depth to the permanent ground water table is on the order of 100 to 150 feet below the ground surface.

Unit C and Unit A_4 tunneling standup time is judged good. Settlement of the surface should not be a problem except beneath MacArthur Lake. Cut-and-cover construction beneath MacArthur Park Lake would mitigate settlement.

This tunnel line is judged to be gassy. This opinion is based on the aforementioned nearby oil fields, sulfur odors detected in most borings and gasoline/petroleum odors in Borings 11-4, 11-6, and 13-5.

Somewhere between Stations 267 and 270 the MacArthur Park fault will be traversed as discussed earlier in Section 7.2. Little is known about this inactive fault, nor the physical conditions of the bedrock on each side. Therefore, poorer bedrock conditions, poorer standup time and ground water inflows of 10 to 20 gpm, for a distance of 10 to few 10's of feet on each side of the fault zone represent our best judgment of conditions within this reach of the tunnel line.

7.3.4 Cross-Passages Between Tunnels - Preliminary plans indicate about 10 cross-passages between tunnels are proposed in Design Unit A170. Based on RTD Tunnel Standard Drawing Nos. SD-053 and SD-054, the cross-passage dimensions are about 20 feet long, 10 feet wide, and 12 feet high. The plans indicate the finished opening will be supported by a 2-foot thick concrete liner.

The passages will require mining between twin-bore tunnels. Based on tunnel lines and grades shown on Drawings 2, 3, and 4, mining will be in siltstone, claystone, and sandstone of the Fernando and Puente formations (Unit C). This is "soft-ground" tunneling material, as described in Section 5.1. Except for the cross-passage beneath MacArthur Lake, Unit C should stand well with little, if any, caving or slabbing that would require bracing, timbers, or rock bolts. Digger-type equipment can excavate this material, possibly assisted by jack-hammers if very hard 1- to 3-foot thick cemented interbeds are encountered. Bedding will likely trend in an east-west direction and dip southerly at angles of 10° to 55° from the horizontal.

The first cross-passage planned just west of the Wilshire/Alvarado Station, at about Station 269, is located beneath MacArthur Park Lake. Cut-and-cover construction is recommended for this cross-passage because available information indicates that there is no cover above the crown of the passage (see Drawing 3).

7.4 DESIGN CRITERIA

7.4.1 Ring Loads

Designing for full overburden pressure is normally acceptable for tunnels at shallow depths in soil or soft ground. For Unit A_4 - Old Alluvium and Unit C - Puente and Fernando Formation, arching of load over the tunnel will be significant at greater depths, and design for full overburden would be overly conservative. This will become particularly significant if something other than a concrete lining is selected. It is recommended that the following criteria, patterned to some extent after BART, be utilized for current design:

$$P_r = p \times R$$

Where: P_r = ring load (lbs/lin. ft)
p = earth and water pressure on lining, assumed uniform all around (psf)
R = radius of tunnel (ft).

Recommended values of design pressure "p" are as follows (see Figure 7-1 for term definition):

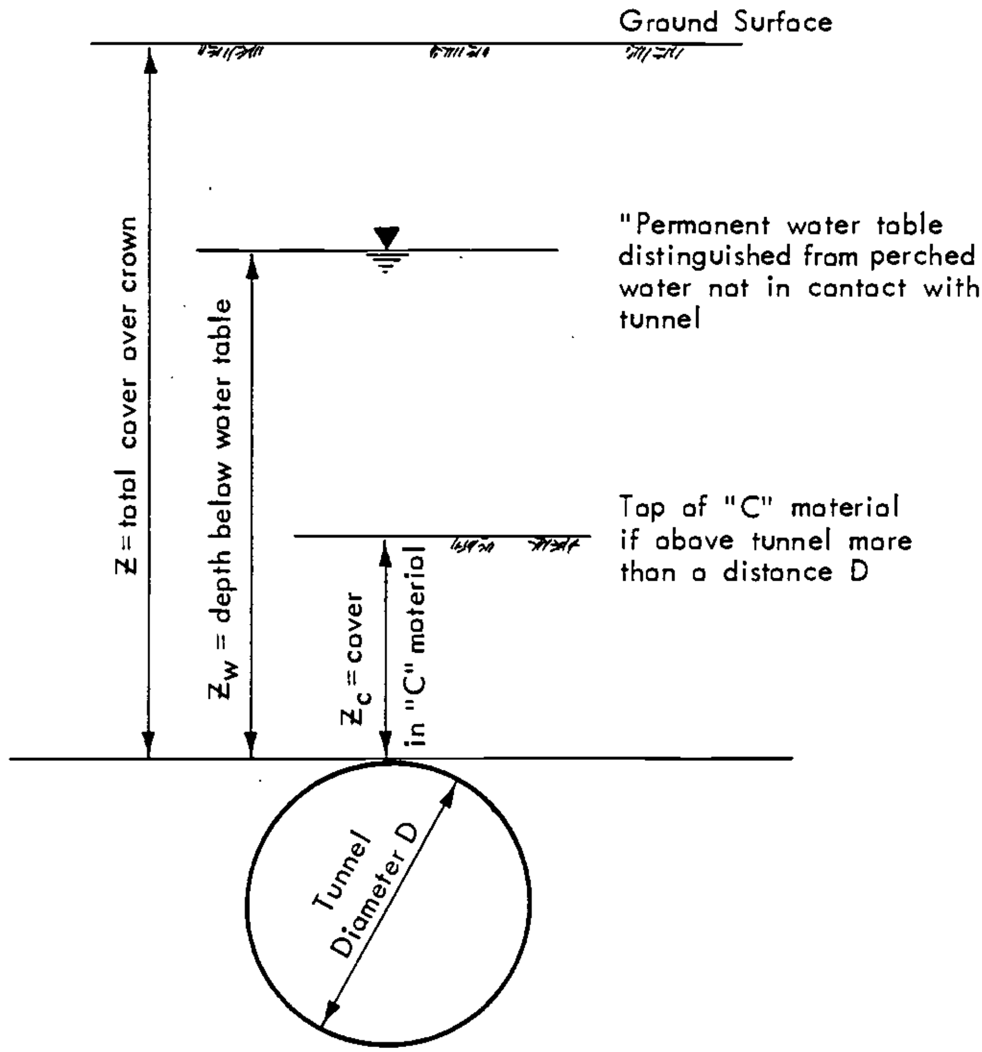
a. Minimum for all cases:

$$p = \gamma D$$

b. Alluvium (A_4):

$$p = \gamma(2D + \frac{Z-2D}{2}) + \gamma_w Z_w$$

(Note: This provides full support of everything in the first 2 diameters over the tunnel, and 1/2 of everything above that.)



LEGEND:

γ = total unit weight of earth, above water table.
Use buoyant unit weight of earth below water table (pcf).

γ_w = unit weight of water (pcf).

q_u = unconfined compressive strength (psf).

TERM DEFINITION FOR TUNNEL RING LOADS

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Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 7-1

c. Puente and Fernando Formations (C):

I. For $Z_c < D$:

$$p = \gamma(2D + \frac{Z-2D}{2})$$

II. For $D < Z_c < 2D$:

$$p = \gamma Z - q_u/2$$

III. For $Z_c > 2D$:

$$p = \gamma Z - q_u$$

(Note: Maximum p should not exceed that calculated for case of $Z_c < D$.)

All the above relationships consider full external water pressure on the lining. It is recommended that full hydrostatic pressures be used in Unit A_4 and C materials where the tunnel is in relatively "impermeable" ground.

7.4.2 Bending Moment

Due to the flexural rigidity of the lining, some bending moments in the linings will be induced. These bending moments can be estimated as follows:

$$M_{max} = \frac{3EI}{R_m} (\Delta R/R) \quad \text{(After Golder, 1976)}$$

where:

M_{max} = maximum bending moment (lb-ft)

E = modulus of elasticity of lining (psf)

I = moment of inertia of lining (ft^4)

R_m = average radius of lining (ft)

R = inner radius of lining (ft)

ΔR = change of radius of lining (ft).

Peck (1969) provides typical values of $\Delta R/R$ for various types of tunnel linings and ground conditions (typically 0.1 percent to 0.6 percent). A value of 0.5 percent is recommended for design.

7.4.3 Buckling

Experience has shown that failure of tunnel linings by buckling normally occurs only where the lining is not in continuous contact with the surrounding ground due to poor installation and grouting procedures. Such conditions cannot effectively be taken into account in design, but must be avoided by proper construction and inspection (after Golder, 1976).

7.4.4 Parallel Tunnels

When a second tunnel is driven parallel and close to an existing tunnel, the resulting stresses on both tunnels can be greater than for a single tunnel. We recommend a minimum pillar width of one tunnel diameter except in special cases such as at portals and transition into station. In such special cases, a minimum of one radius pillar width is recommended. It is recommended also that ground loading "p" (Section 7.4.1) for design should be increased by 20 percent where pillar width is less than one tunnel diameter. This increase applies only to the earth component of "p", not to the water component.

7.4.5 Surcharge from Adjacent/Overhead Buildings

According to Peck (1969), bored tunnels adjacent to our underlying existing buildings may experience marginally more radial deformation than normal during excavation and prior to installation of lining. This increased deformation apparently causes increased earth arching, with the net result that little if any increased ring load has been observed.

7.4.6 Jacking Forces

The tunnel will probably be advanced by jacking the shield against the lining. These induced jacking stresses can be substantial and often control the lining design. Consideration should be given to this factor if the temporary support is also to be used as the permanent lining, as with precast concrete segments.

7.4.7 Shafts

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 degrees from the vertical, can be estimated as follows:

- o Fine-Grained Alluvium (A_4) and Claystone (C)

  Radial pressure 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

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

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

μ = anticipated groundwater pressure at designated location (psf)

o <u>Granular Alluvium (A_4) and Siltstone/Sandstone (C)</u>

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 at great depths. Radial pressures on shafts can be estimated as:

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

where:

σ_r = total radial pressure (psf)

K_a = active lateral earth pressure coefficient

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

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

μ = anticipated groundwater 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

7.4.8 Earthquake Effects

Effects of earthquake loading or permanent underground structures are addressed in our separate report entitled, "Seismological Investigation and Design Criteria," dated May 1983.

7.5 SUPPLEMENTARY GEOTECHNICAL INVESTIGATION--MACARTHUR PARK LAKE CROSSING

Based on the expected absence of ground cover above the tunnel beneath MacArthur Park Lake, we recommend a supplementary geotechnical investigation be performed to provide site specific subsurface data for the design of cut-and-cover construction from the Alvarado Station to the west end of MacArthur Park Lake.

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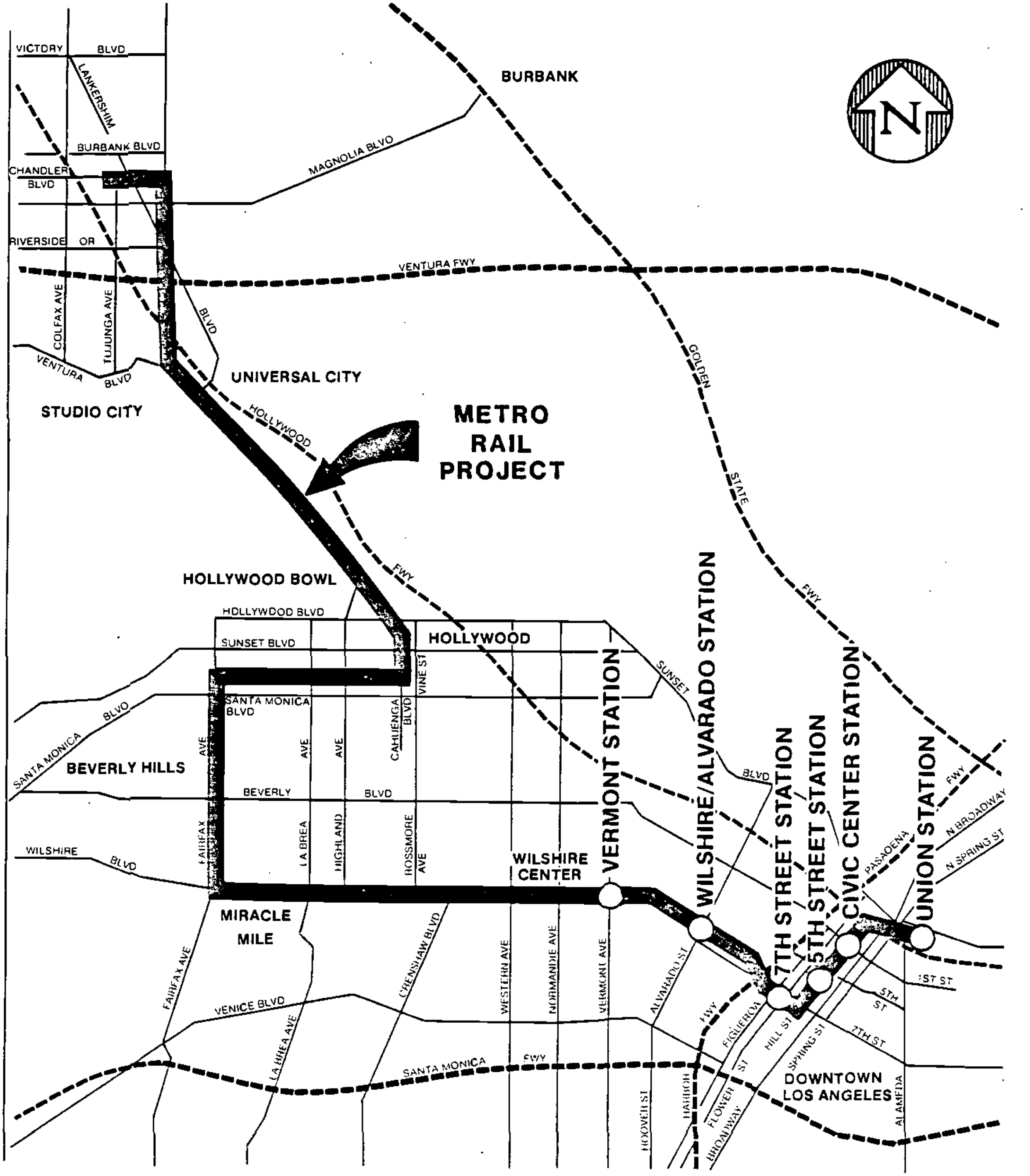

VICINITY MAP

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1101

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Geotechnical Engineering
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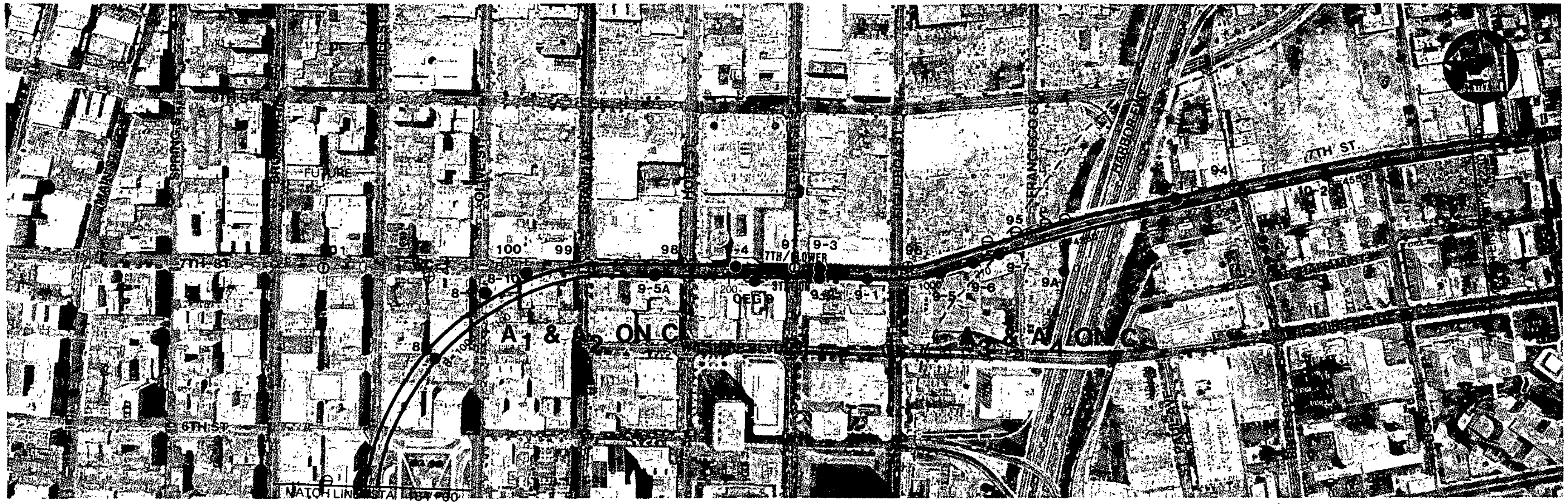
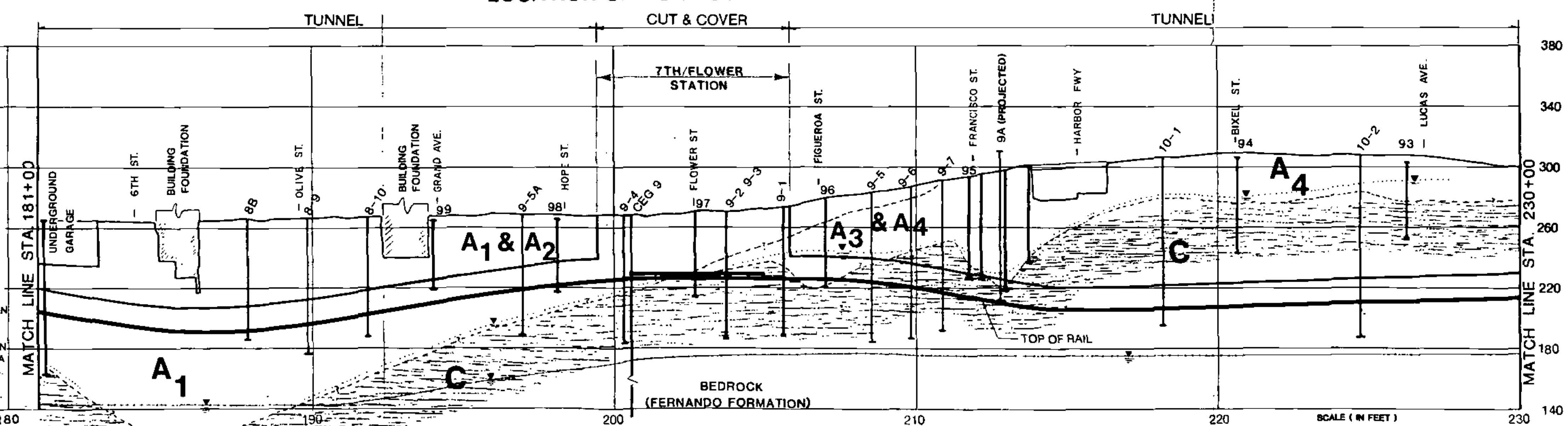
Drawing No
1

Approved for publication 10/11/83 by JAD



LOCATION OF BORINGS



NOTES:

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

2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 9

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

REFERENCE:
MILESTONE 10 SHEET 4 OF 21 ALIGNMENT PLAN AND PROFILE STATION 181+OO TO STATION 230+00 DATEO MARCH 1983.

GEOLOGIC SECTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

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

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

DESIGNED BY
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Submitted R.M. Pride Date 10/21/83

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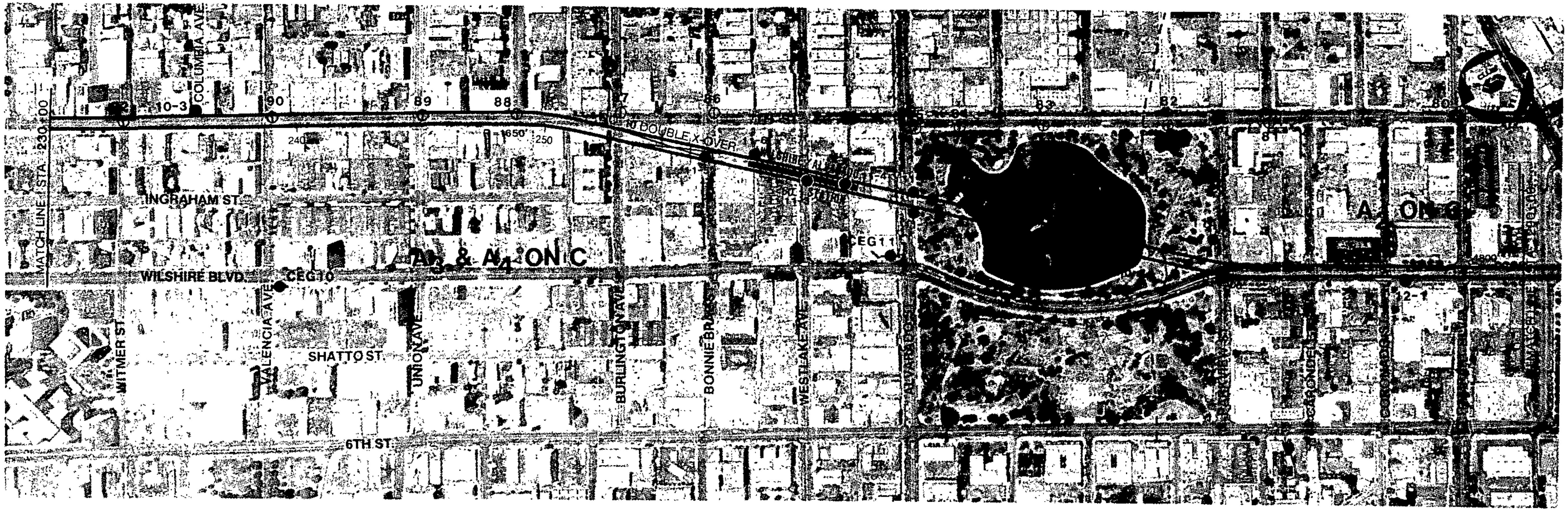
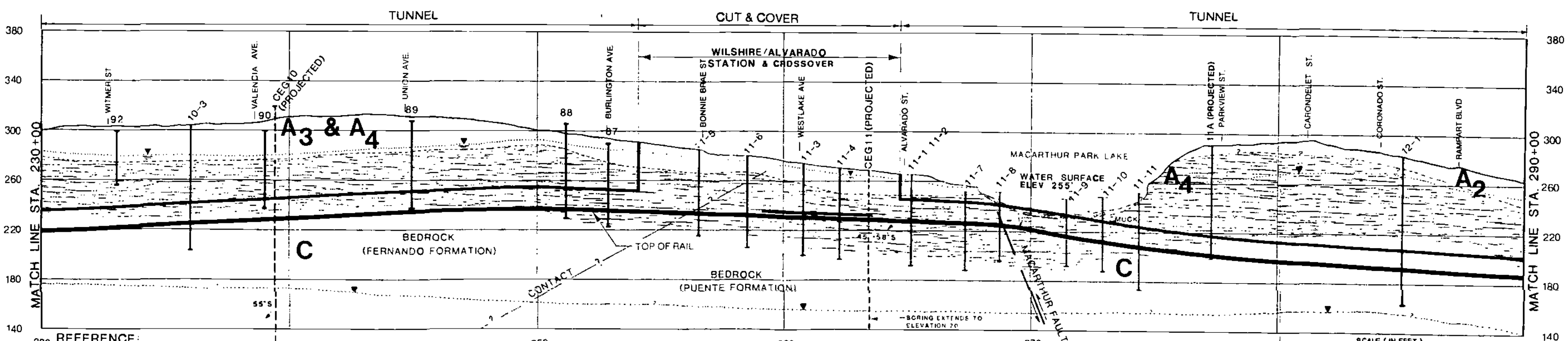
LOCATION OF BORINGS
AND GEOLOGIC SECTION

PROJECT NO. 83-1101
DRAWING NO. 2
REV.
SCALE AS SHOWN
SHEET NO.



LOCATION OF BORINGS



REFERENCE:
MILESTONE 10 SHEET 5 OF 21 ALIGNMENT PLAN AND PROFILE STATION 230+00 TO STATION 290+00 DATED MARCH 1983.

FOR NOTES SEE DRAWING 2

GEOLOGIC SECTION

REV.	DATE	BY	SUB	APP.	DESCRIPTION

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DESIGN UNIT A170 LOCATION OF BORINGS AND GEOLOGIC SECTION

PROJECT NO 83-1101
DRAWING NO. 3
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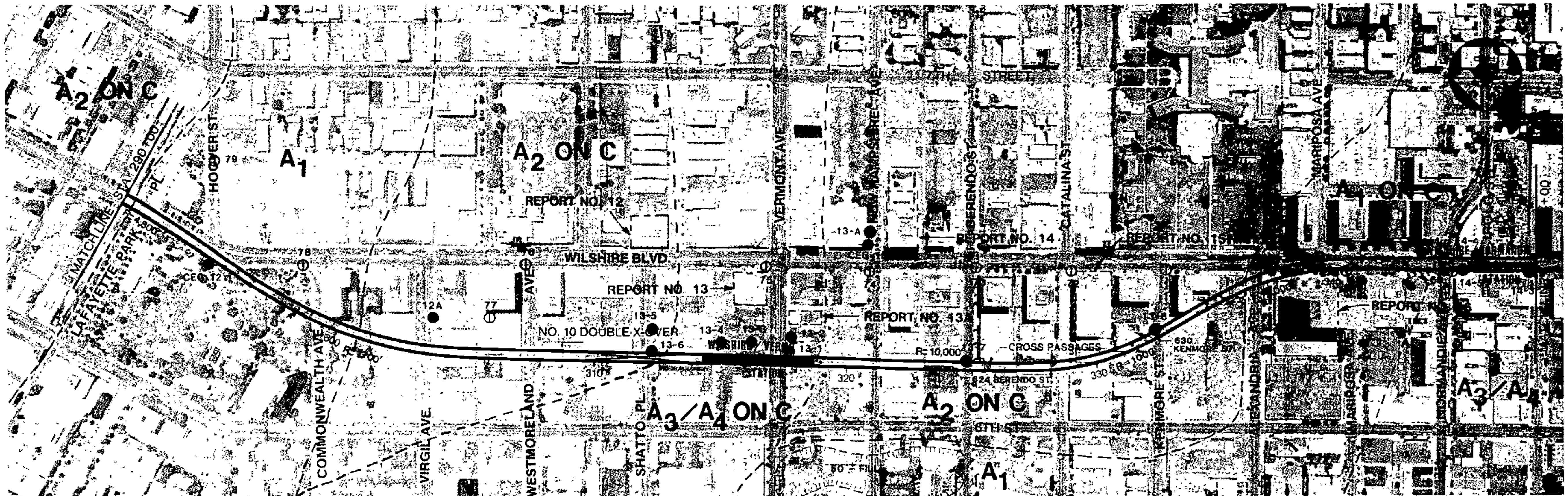
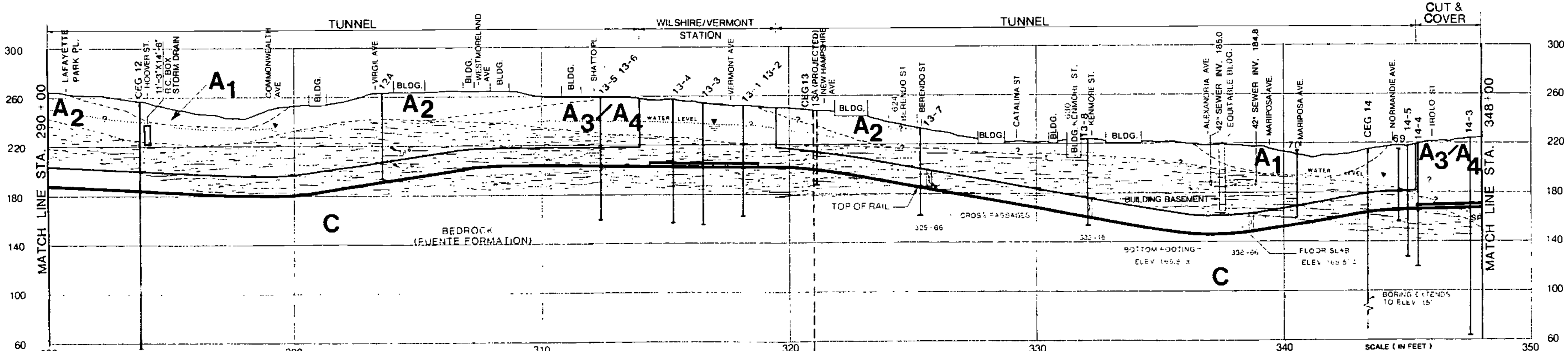


LOCATION OF BORINGS



GEOLOGIC SECTION

REFERENCE.
MILESTONE 10 SHEET 6 OF 21 ALIGNMENT PLAN AND PROFILE STATION 290+00 TO STATION 348+00 DATED MARCH 1983

NOTES:
1. LOCATION AND GRADE OF TUNNEL AND STATION SUBJECT TO CHANGE.
2. FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 12.
3. THIS DRAWING WAS PREPARED AS AN AID IN DEVELOPING DESIGN RECOMMENDATIONS. SUBSURFACE INFORMATION PRESENTED ON THIS DRAWING IS BASED ON ... SUBSURFACE DATA BETWEEN AND BEYOND BORING ... CONDITIONS ENCOUNTERED DURING CONSTRUCTION MAY BE DIFFERENT.

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LOCATION OF BORINGS AND GEOLOGIC SECTION

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DRAWING NO. 4
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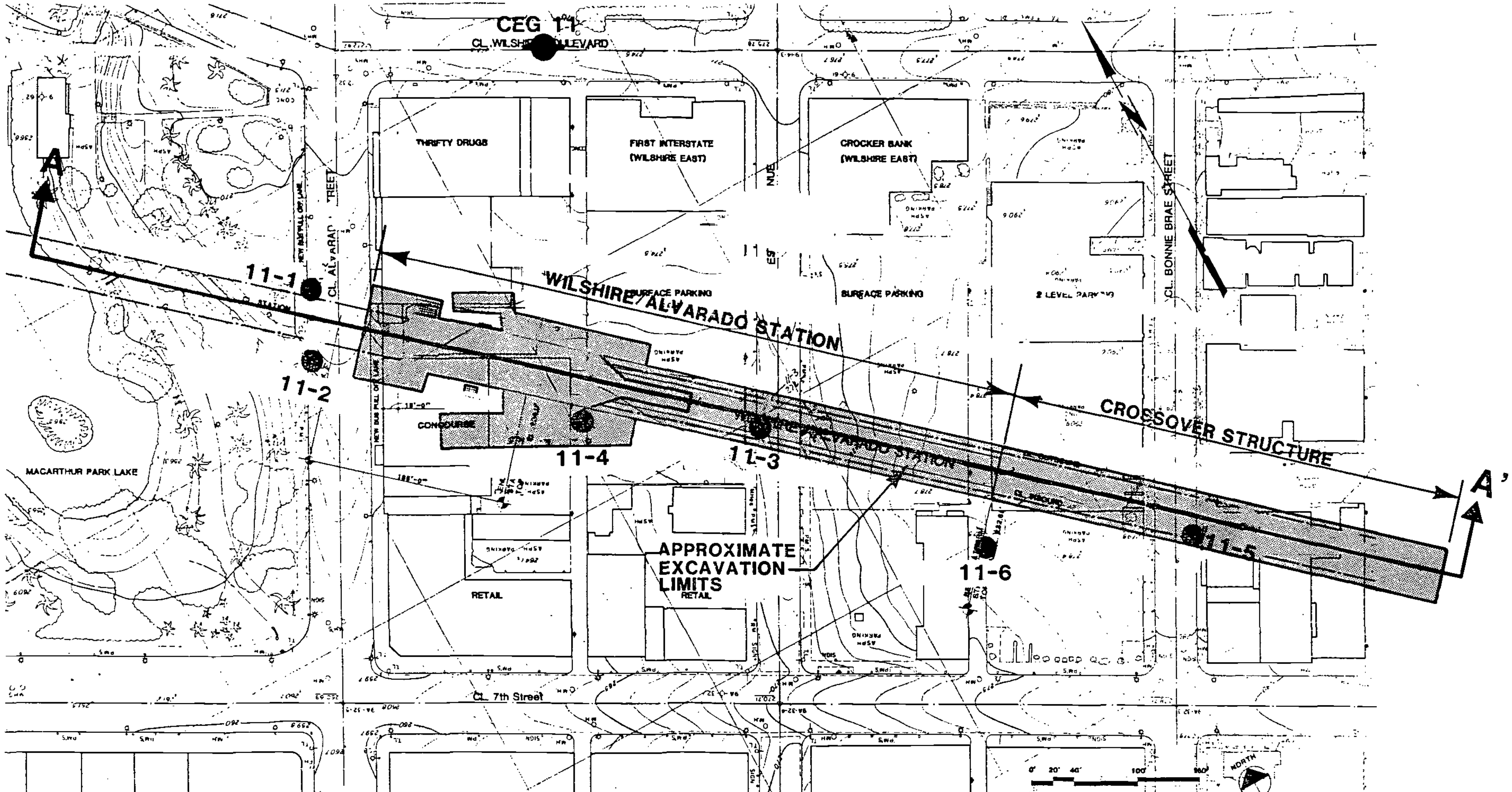

NOTE: GRATES, SHAFTS, AND COOLING TOWER LOCATIONS TO BE CONFIRMED BY FINAL DESIGNER

REF:
"PRELIMINARY WILSHIRE/ALVARADO STATION SITE PLAN" PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" 40' REDUCED TO 1"= 100'.

NOTES:

1.) FOR SECTION A-A' SEE DRAWING NO. 6
2.) FOR EXPLANATION SEE DRAWING NO. 7

LOCATIONS OF BORINGS & CROSS SECTION

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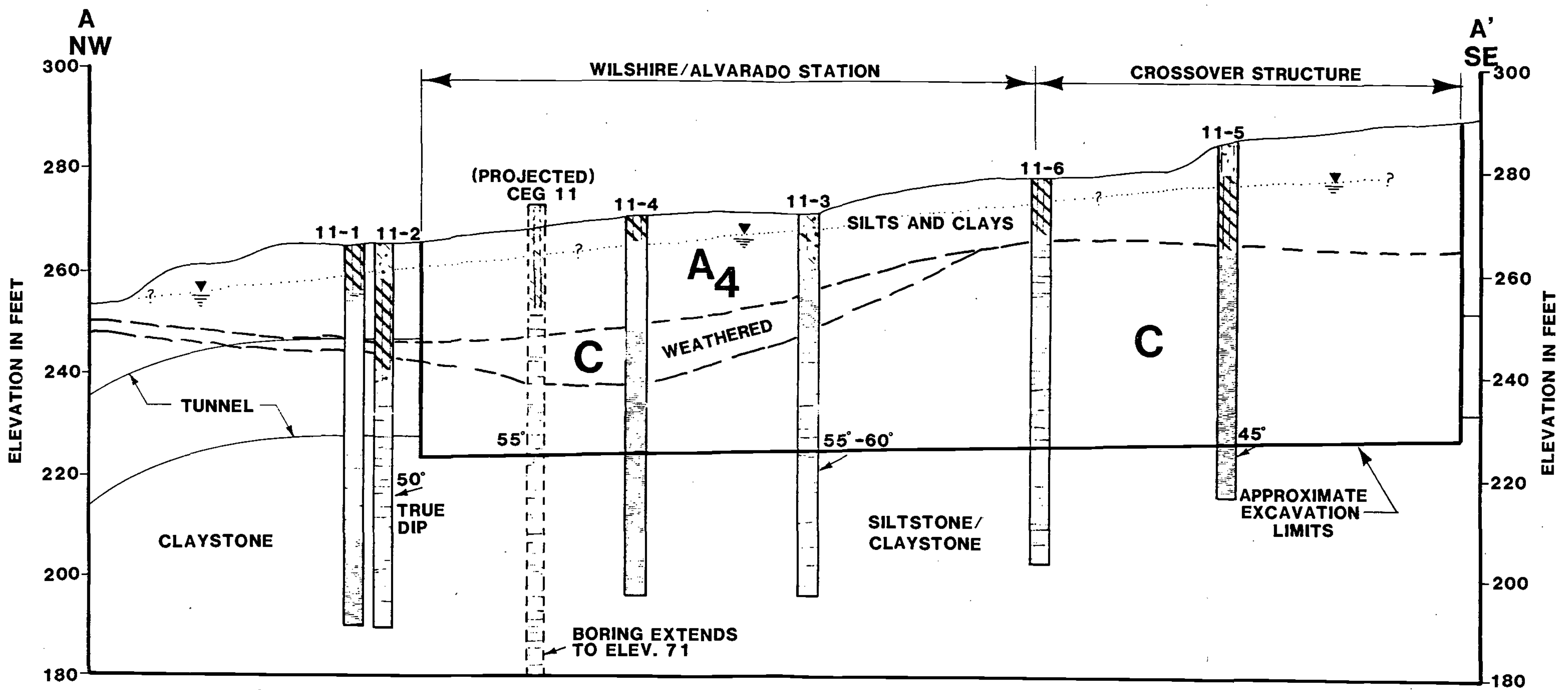
Scale As Shown	Project No 83-1101
Date 10-11-83	
Prepared by RG	Drawing No 5
Checked by JAD	
Approved By RMP	



SCALE

HOR. 0 50 100 200 feet

VER. 0 5 10 20 40 feet

NOTES:

1.) FOR SECTION LOCATION SEE DRAWING NO. 5

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

GEOLOGIC SECTION A-A'

DESIGN UNIT A170
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	As Shown	Project No	83-1101
Date	10-11-83		
Prepared by	RG	Drawing No	
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GEOLOGIC UNITS

SOFT GROUND TUNNELLING

QUATERNARY — HOLOCENE

A1 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.

A2 YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.

QUATERNARY — PLEISTOCENE

A3 OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.

A4 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
- Fault: 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
- Perched water level: approximately located; queried where inferred
- Permanent 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)

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

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) The locations of the tunnel line and stations are based on the Metro Rail Project, Milestone 10 alignment as of February, 1983.
3) Borings projected more than 200' 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.
4) Displacements shown along faults are graphic representations. Actual vertical offsets are unknown.

GEOLOGIC EXPLANATION

DESIGN UNIT A170
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	N/A
Date	10-11-83
Prepared by	RG
Checked by	JAD
Approved By	HAS
Project No	83-1101
Drawing No	7

Appendix A

Field Exploration

APPENDIX A-FIELD EXPLORATION

A.1 SUMMARY

Six borings (11-1 TO 11-6) were drilled in 1983 and three borings (CEG 10, 11 and 12) were drilled in 1981 near Wilshire/Alvarado Station, Design Unit A170 (See Drawing 2). The field boring logs are appended. In addition, two borings, 100 feet deep, were drilled for a cross-hole survey at Boring CEG 11 near the Wilshire/Alvarado Station location. A summary of 1981 boring data is presented in Table A-1.

Sample and core recovery from the borings was good. Sample recovery from the Converse ring, Split Spoon and Pitcher Barrel samplers averaged 84%. This appendix includes:

- o Field Boring Logs,
- o Summary of Boring Data - 1981 (Table A-1),
- o Correlation of Soil Data Obtained in the Field (Table A-2),
- o Bedrock Description Terms (Table A-3),
- o Water Pressure Test Results (A-4),
- o Ground Water Monitoring - 1981 (Table A-5).

A.2 ROTARY WASH AND WIRE LINE CORING

A.2.1 Technical Staff

Members of three firms (CWDD/ESA/GRC) participated in the drilling exploration program. The field geologist was responsible for preparing a detailed lithologic log of the rotary wash cuttings, sample/core identification, labeling and storage of samples, water pressure tests, installation of piezometer pipe, gravel pack and bentonite seals.

A.2.2 Drilling Contractor and Equipment

Drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 1500 rotary wash rigs.

A.3 SOFT-GROUND LOGGING AND SAMPLING

Rotary drill/sampling was performed over the length of Design Unit A170. Station locations were investigated with additional borings during the course of this investigation, but no additional borings were drilled along the proposed tunnel alignment. A summary of the drilling and sampling procedure follows:

TABLE A-1

SUMMARY OF 1981 BORING DATA

Hole No.	Depth of Geology Boring *	Depth of Geology Boring **	Feet in Soil	Feet in Rock	Station* Depth (ft)	Tunnel* Depth (ft)	Sampling/Core Recovery Number of Samples/% Recovery SS	DS	PS	NX	Down Hole	Cross Hole	Piezometer	Piezometer Installed Depth (ft)	Water Pressure Test Depth Intervals (ft)	Natural Gas	Amount of Casings Installed (ft) 4 in.	3 in.†	2 in.	1 in.	Hole Completion Cased and Grouted	Hole Completion Piezometer
10	200	202.0	22.5	179.5	-	165	03/88	2/00	62/72	0/-	no	no	yes	0-42 0-200	Not performed	Trace	-	-	200.0	42	-	Double
11	200	201.1	20.0	180.2	60	-	04/100	1/100	63/92	0/-	yes	yes	no	-	63-83 35-55 55-75	Yes	100	200	-	-	✓	-
12	200	200.1	32.0	168.1	-	95	03/62	1/80	60/95	0/-	no	no	yes	0-200	Not performed	Trace	-	-	200	-	-	Single

- In overburden, standard penetration tests (SPT), using a conventional split spoon sampler driven with a 30-inch stroke and 140-pound hammer, were performed at 10-foot intervals. In bedrock Converse ring samples, using a 450-pound slip-jar hammer, or pitcher-Barrel samples were taken at 20-foot vertical intervals; followed by the split spoon.

- Geophysical downhole and crosshole surveys were performed in Boring CEG 11. The crosshole procedure involved drilling two additional 100-foot deep borings in line with, and spaced 15 to 30 feet from, the boring. It was not possible to complete cross- hole borings as ground water observation holes because PVC casing was grouted in place.

A.4 LITHOLOGIC LOG

A.4.1 Field Classification of Soils

All soil types were classified in the field by the site geologist using the "Unified Soil Classification System." Based on the characteristics of the soil, this system indicates the behavior of the soil as an engineering construction material.* 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-2 shows the correlation of N-values from standard penetration tests as compared to the physical description of the consistency of clays (hand-specimen and the compactness of sands described by the field geologists).

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

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

*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.

The following symbols were used 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.4.2 Field Description of the Formations

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

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

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

Physical condition:____________fractured, minimum__________, maximum,__________, mostly__________; __________hardness; __________strength; __________weathered.

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

A.5 WATER PRESSURE TESTING

Water pressure tests were performed in Boring CEG 11 during the 1981 field investigation. Results of these tests are summarized in Table A-4. The artesian flows that were observed in this hole made it difficult to determine whether or not leakage of the packer was occurring. However, because of the massive and clayey nature of the Puente Formation, good data was gathered. The data demonstrates that the formation is tight and, in all cases, takes less than about 1 gpm over a 20-foot test interval at 40 psi.

It is anticipated that the greatest water inflow from this soft claystone/siltstone formation will occur along fault planes and along the hard-cemented concretionary beds occasionally encountered in the borings. Flows from these zones are anticipated to be on the order of 5 gpm. Marginal success was achieved when attempts were made to double pack across some hard-cemented zones as was previously mentioned. Packer leakage during these selective tests allowed only estimates of actual water loss to the formation.

A.6 GROUND WATER MONITORING

Two-inch diameter plastic ABS pipe was installed in Borings CEG 10 and 12. The annulus of the boring around the pipe was backfilled with a coarse sand/pea. gravel aggregate. A 5-foot thick surface bentonite seal was placed on all monitoring holes 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 an using air lift provided by a trailer-mounted air compressor. The boring 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 are presented in Section 5.3.

TABLE A-3 Bedrock Description Terms

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

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

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

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

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

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

TABLE A-4

WATER PRESSURE TEST - 1981

Boring No.	Depth of Section Tested (ft)		Pressure (psi)	Duration of Test (min)	Average Flow (gpm)	Remarks
	Top	Bottom				
11	63.0	83.0	20	5	1.0	Natural artesian flow made packer leakage estimates difficult
	63.0	83.0	40	5	1.0	
	63.0	83.0	60	5	1.6	
	63.0	83.0	80	5	1.6	
	55.0	75.0	20	5	0.6	
	55.0	75.0	40	5	0.7	
	55.0	75.0	60	5	0.9	
	55.0	75.0	80	5	0.8	
	35.0	55.0	20	5	0.8	
	35.0	55.0	40	5	0.9	
	35.0	55.0	60	5	1.3	
	35.0	55.0	80	5	1.6	

TABLE A-5

GROUNDWATER MONITORING SUMMARY - 1981

Hole No.	Pipe Diameter (in.)	Pipe Depth (ft)	Date Depth Measured	Depth to Water (ft)	Depth to Water (ft) 06/17/81	Date of Water Quality Sample	Remarks
10	2	200.0	03-22-81	27.1	25.1	*	Bentonite separating plug ineffective; reads perched water
10	1	42.0	03-22-81	22.7	22.9	02-23-81	Reads perched water
11	crosshole	-	--------	-----	--------	01-31-81	Artesian
12	2	200.0	03-22-81	14.5	16.1	02-18-81	Reads perched water

Y321 X036

RECEIVED BY MRTC

DEC 7 1983

DOCUMENT CONTROL

A170

Converse Consultants **MEMORANDUM**

Date	December 2, 1983	Project No.	83-1140-71, 83-1101-83
To	File	Subject	Record of MRTC
From	Howard A. Spellman		Boring 9A, Francisco/7th Street

Man-sized auger Boring 9A, drilled today, 12/2/83, is the 4th of 5 borings drilled under the Amendment No. 1 to the MRTC Contract No. 503. These are the notes from John Stellar's downhole observations and logging. The boring is located in Design Unit A170, on the west side of Francisco Street (25 feet east of the Harbor Freeway) and 180 feet north of 7th Street. The purpose of the boring was to determine bedrock and groundwater conditions at the tunnel grade. The boring is at approximately tunnel line Station 213, collar Elevation 315 feet, tunnel invert grade at this location is about Elevation 205 feet and total depth of the hole is 100 feet. (305

Type Material

0' to 12' - fill, mixed sandy silt and clayey sand, 3 inch thick old concrete floor slab at about 12 feet

12' to 18' - sand, buff to orange brown, slightly moist to dry, medium dense, clean fine sand

18' to 100' - Bedrock (Fernando Formation) - primarily sandy siltstone, with occasional very hard nodules of cemented shells up to 8 inches diameter, light greenish grey to orangish brown (weathered) from 18 to 44 feet, blue grey (fresh) from 44 to 100 feet, closely fractured slightly moist, friable to low hardness, and weak strength (18 to 44 feet), moderately hard, weak to moderately strong 18 to 100 feet, generally massive containing very few bedding planes, bedding plane attitude at 44 feet is N72°W, dipping 24° South; nodules to 8 inches in diameter composed of shells and calcareous cement from 52 to 54 feet; bag sample at 100 feet.

Bucket auger drilled the "soft" bedrock easily, which should translate into good excavation for tunneling equipment.

Caving: The hole stood very well for the entire 100-foot depth. Bedrock at tunnel grade appears to be excellent tunneling material.

Groundwater: No groundwater was encountered in this boring; a very minor seep (droplets) issued from a siltstone strata from 40 to 41 feet and 52.5 to 52.7 feet; the latter had an oder of sulfur (H_2S) gas bubbling out of the seep at 52.5 feet; another very minor seep occured at 73 feet; total inflow of seeps is less than 0.1 gpm.

Gas and Oil: No gas was detected by the meter and no petroleum products were encountered in the boring. However, a sulfurous (H_2S) gas odor was detected at 52.5 feet and a light sulfurous (H_2S) odor at 62 feet.

Downhole Observers: John Stellar (CCI) observed downhole conditions. Interested observers were Bud Maduke and Frank McLean (MRTC), John Campbell and Buzz Spellman (CCI).

HAS:dmG

cc: B.I. Maduke & R.J. Proctor

Converse Consultants

Boring Log 9-5

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME.
CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12/13-84 HOLE NO. 9-5
LOCATION 7th STREET, 300' EAST OF FRANCISCO ST. ON 7th STREET GROUND ELEV. 284
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Mark Schluter DEPTH TO GROUND WATER ____
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 325#@18"/140#@30"
SURFACE CONDITIONS A.C. STREET SURFACE TOTAL DEPTH 100.0' NO. CORE BOXES ____

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0-0.8 CONCRETE GUTTER			C			STARTED DRILLING @ 1540
	SC	0.8-1.8 CLAYEY SAND (FILL) MODERATE BROWN 5YR 3/4 65%-75% SAND 35-25% FINES LOW PLASTICITY, VERY MOIST, LOOSE						
2	DRAIN S / S RPC	1.8-3.5' DRAIN PIPE AND SLURRY BACKFILL 1' DRAIN PIPE (2.25'→3.25') PARALLELS 7th STREET						DRAIN PIPE DRY, POSSIBLY ABANDONED, WILL REPAIR PIPE AT COMPLETION OF HOLE
4	SC	3.5'-11.0' CLAYEY SAND MODERATE BROWN 5YR3/4 70-80% SAND F/C 30-20% FINE LOW PLASTICITY, MOIST, MEDIUM DENSE, TRACE GRAVEL 5-35mm MOTTLED	C-1	8 / 14	DR 325			
6					RD			ROTARY WASH INSTALLED 6' OF CASING, TRI-CONE BIT, BENTONITE ADDED TO DRILLING FLUID
		TRACE MICACEOUS	C-1	7 / 14 / 22	SS 140		1.0 / 1.5	
8								DIFFICULTY EXTRACTING SPT
10								SAMPLE NOT RECOVERED
	SW	11.0-22.0 SAND / GRAVELLY SANDS DUSKY YELLOW 5Y6/4 80-100% SAND F/C 20-0% GRAVEL 5-40mm SLIGHTLY MOIST, MEDIUM DENSE TO DENSE, CLEAN SANDS, WELL GRADED, SAND GRAINS SUBANGULAR, TRACE GRAVEL						
12			C-2	13 / 24				
14		GRAVELS, GRAVEL LENSES			RD			SLIGHT-MODERATE DRILL RIG CHATTER
16	SW		C-2	21 / 50/4"	SS 140		0.8 / 0.9	SPT REFUSAL @ 10"
18					RD			
20								

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 1 OF ____

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12/13-84 HOLE NO. 9-5

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	SW	SAND - CONTINUED - 20.0' GRAVELS, GRAVEL LENSES	C-3	61 60/3	DR 325			CCI REFUSAL @ 9" GRAVELS BLOCKED AND REDUCED RECOVERY
					RD			
22	SP	22.0 – 28.0 SAND YELLOWISH GREY 5Y7/2 100% SAND F/M SLIGHTLY MOIST, DENSE, POORLY TO MODERATELY GRADE, OCCASIONAL LENSES OF SILTY FINE SAND (ML), RUSTIC BROWN OXIDATION STAINING, CLEAN SANDS, GRAINS SUBANGULAR	C-4	62 60/4	DR 325			
24					RD			
26								
				21	SS 140		1.4/1.5	
28	ML/SP	28.0 – 37.0 SILTY SAND / SAND YELLOWISH GRAY 5Y7/2 PALE YELLOWISH BROWN 10YR6/2 70-100% SAND 30-0% FINES MOIST, DENSE, POORLY GRADED, WEAKLY CEMENTED IN SPOTS, INTERBEDDED (ML) AND (SP), RUSTIC BROWN OXIDATION STAINING, (SP) ARE CLEAN SANDS.	J-3	29 34				5-12-84 SHUT DOWN @ 1836 STARTED DRILLING @ 0600 5-13-84
					RD			
30			C-5	36 89/5	DR 325			CCI REFUSAL @ 11.5"
32					RD			
		BOIL CLASSIFICATIO... DESCRIPTIONS SHOWN ON THE BORING LOG ... OUR INITIAL UNEDITED FIELD CLASSIFICATI... LOGS HAVE NOT BEEN EDITED OR MODIFIED ... THE RESULTS OF LABORATORY TESTING AN... CLASSIFICATION.	C-6	30				5 RINGS RECOVERED CCI REFUSAL @ 9"
34								
					RD			
36								
37	SM	37.0 – 41.0 SILTY SAND MODERATE BROWN 5YR4/4 DARK YELLOWISH BROWN 10YR4/2 65-75% SAND F/M 35-25% FINES (SILT) MOIST, DENSE, TRACE MICACEOUS MODERATE BROWN OXIDATION STAINING	PB 1		PB		7	SAMPLING PRESSURE 160 P.S.I. 2.1/3.0
38								
40								
41		41.0 – FERNANDO FORMATION SILTSTONE MEDIUM GRAY N5 100% FINES LOW PLASTICITY, DAMP, BEDDING INDISTINCT, VERY STIFF TO HARD (ML)			RD			
42				19 40	SS 140		1.5/1.5	
			J-4	56				
44					RD			SHEET 2 OF ___

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-13-84 HOLE NO. 9-5

DEPTH 44	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		FERNANDO FORMATION SILTSTONE -CONTINUED-			RD			
46		PHYSICAL CONDITION LITTLE FRACTURED TO MASSIVE, FRIABLE HARDNESS AND STRENGTH, LITTLE WEATHERING, SOFT ROCK	PB 2		PB			SAMPLING PRESSURE 200 P.S.I. 1.6/3.0
48		48.0 - 49.0 GRAVELS, GRAVEL LENSE 49.0 - 100.0 5-30mm SILTSTONE / CLAYSTONE			RD			SLIGHT TO MODERATE DRILL RIG CHATTER
50		DARK GREENISH GRAY 5GY 4/1 SHELL FRAGMENTS AND CALCITE INCLUSIONS 1-2 mm, MICACEOUS CLAYSTONE - MEDIUM TO HIGH PLASTICITY LITTLE FRACTURED TO MASSIVE FRIABLE HARDNESS AND STRENGTH FRESH, BEDDING INDISTINCT	C-7	28 56	DR 325			
52					RD			
54			PB 3		PB			SAMPLING PRESSURE 180 P.S.I. 2.1/3.0
56					RD			
58		- CONTINUED -	C-8	60 60/4"	DR 325			CCI REFUSAL @ 10"
60					RD			
62			PB 4		PB			SAMPLING PRESSURE 150 P.S.I. 2.8/3.0
64					RD			
66			C-9	24 49	DR 325			
68					RD			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 3 OF ____

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-13-84 HOLE NO. 9-5

DEPTH 68	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		FERNANDO FORMATION SILTSTONE / CLAYSTONE - CONTINUED -			RD			
70			PB 5		PB			SAMPLING PRESSURE 160 P.S.I. 2.55/3.0
72		- CONTINUED -			RD			
74			C-10	24 / 39	DR. 325			
76		SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.						
78		- CONTINUED - DARK GREENISH GRAY 5GY 4/1 DAMP, TRACE MICACEOUS, SHELL FRAGMENTS, MASSIVE, FRIABLE - LOW HARDNESS, FRIABLE - MODERATE STRENGTH, FRESH	C-11	59 / 75/4	DR. 325			CCI REFUSAL @ 10"
80								
82					RD			
84			C-12	39 / 50/4	DR. 325			CCI REFUSAL @ 10.5"
86					RD			
88		- CONTINUED -	C-13	35 / 50/4	DR. 325			
90					RD			
92								SHEET 4 OF ___

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-13-84 HOLE NO. 9-5

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (N)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92		FERNANDO FORMATION SILTSTONE/CLAYSTONE - CONTINUED -			RD			
94			C-14	32 50/5	DR 325			
96		- CONTINUED - DARK GREENISH GRAY 5G 4/1 DAMP, TRACE MICACEOUS, SHELL FRAGMENTS, MASSIVE, FRIABLE TO LOW HARDNESS, FRIABLE TO MODERATELY STRONG, FRESH, BEDDING INDISTINCT.			RD			
98								
100			C-15	32 52	DR 325			
		END OF BORING 100.0' TREMIED A 3 SAC/85 GALLON CEMENT SLURRY INTO BORING						FINISHED DRILLING @ 1100

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 5 OF ____

Converse Consultants

Boring Log 9-6

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12-84 HOLE NO. 9-6
LOCATION 7th STREET, 176' EAST OF FRANCISCO ST. ON 7th STREET GROUND ELEV. 289
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Mark Schluter DEPTH TO GROUND WATER ____
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 325#@18"/140#@30"
SURFACE CONDITIONS A.C. STREET SURFACE TOTAL DEPTH 100.0' NO. CORE BOXES ____

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0-0.7 CONCRETE GUTTER			C			STARTED DRILLING @ 0650
	SW	0.7-1.5 GRAVELLY SAND ASPHALTIC ROAD BASE IMPORT						
2	CH	1.5-3.5 CLAY (FILL) MODERATE BROWN 5YR3/4 100% FINES VERY MOIST, VERY SOFT - SOFT, MEDIUM - HIGH PLASTICITY, MOTTLED	C-1	1 / 2	DR 325			5 RINGS RECOVERED
4	CL	3.5-6.5 CLAYEY SILT DUSKY YELLOW 5Y6/4 DARK YELLOWISH BROWN 10YR4/2 100% FINES VERY MOIST, SOFT, MEDIUM PLASTICITY, MOTTLED, INCLUDES SILTSTONE FRAGMENTS			R			ROTARY WASH TRI CONE BIT BENTONITE ADDED TO DRILLING FLUID
6	SC	6.5-10.0 SILTY CLAYEY SAND MODERATE BROWN 5YR3/4 70-80% SAND F/C 30-20% FINES MOIST, MEDIUM DENSE, LOW-MEDIUM [illegible]	J-1	6 / 11 / 22	SS 140		1.4/1.5	
8								
10	SM	[illegible] 80-90% SAND F/C 20-10% FINES MOIST, MEDIUM DENSE - DENSE, TRACE MICACEOUS, SLIGHTLY POROUS, TRACE - LITTLE GRAVEL 5-35 mm CLAY BINDER			R			[illegible] CONE BIT [illegible] - DRAG BIT
12					R			[illegible] RIG
								TOP RINGS DISTURBED
14	SW	13.0-16.2 SAND YELLOWISH GRAY 5Y 7/2 90-100% SAND F/C 10-0% GRAVEL 5-25 mm SLIGHTLY MOIST, MEDIUM DENSE - DENSE, WELL GRADED, CLEAN SANDS, TRACE GRAVEL AND GRAVEL LENSES	C-3	13 / 25	CS 325			
					R			
16								
18	ML/SM	16.2-21.0 SANDY SILT / SILTY SAND LIGHT OLIVE GRAY 5Y5/2 65-75% SAND F/M 35-25% FINES MOIST, DENSE, CLAY BINDER MODERATE BROWN AND OLIVE BROWN STAINING	J-2	25 / 51	SS 140		0.9/1.0	SPT REFUSAL @ 12"
20					R			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 1 OF ____

PROJECT SCRTD 82-1140-21 DATE DRILLED 5-12-84 HOLE NO. 9-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (BF)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	ML/SM		C-4	31 60/5	DR 325			CCI REFUSAL @ 11"
	SW/GM	21.0 - 35.5 SAND. YELLOWISH GRAY 5Y7/2. 85-100% SAND F/C. 15-0% GRAVEL 5-40mm. MOIST, DENSE - VERY DENSE, INTERBEDDED (SW) AND (SP) LAYERS, GRAVEL LENSES, GRAYISH YELLOW STAINS AND STREAKS, WELL GRADED TO POORLY GRADED, MICACEOUS, CLEAN SAND → TRACE BINDER, SUBANGULAR SAND GRAINS & GRAVELS			RD			
22			C-5	46 60/5	DR 325			CCI REFUSAL @ 11"
24					RD			
26	(W)		J-3	31 50/3	SS 140		0.6/0.7	SPT REFUSAL @ 9"
28		GRAVELS AND GRAVEL LENSES			RD			SLIGHT - MODERATE DRILL RIG CHATTER
30		OCCASSIONAL INTERBEDDED (SM/ML) LAYERS	C-6	38	DR 325			
32		YELLOWISH GRAY 5Y7/2						
	SP	POORLY GRADED, CLEAN F/M SAND VERY DENSE			DR 325			CCI REFUSAL @ 8"
34					RD			
36	ML	35.5 - 41.0 CLAYEY SILT. YELLOWISH GRAY 5Y7/2. LIGHT OLIVE GRAY 5Y5/2. 90-100% FINES. 10-0% SAND F/M. MOIST, STIFF - VERY STIFF, LOW PLASTICITY, RUSTIC BROWN OXIDATION STREAKS,	J-4	11 21 32	SS 140		1.4/1.5	
38					RD			
40		PALE BROWN 5YR 5/2. OCCASSIONAL SAND (SP) LENSES. SAND FINE → MEDIUM	C-8	21 31	DR 325			
42	SC	41.0' - 43.0' CLAYEY SAND. YELLOWISH GRAY 5Y6/2. 80-90% SAND F/C. 20-10% FINES. MOIST, LOW-MEDIUM PLASTICITY, MEDIUM DENSE, TRACE GRAVEL	PB 1		PB			SAMPLING PRESSURE 150 P.S.I. 2.7/3.0
44								

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 2 OF ____

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12-84 HOLE NO. 9-6

DEPTH 44	CLASS	FIELD DESCRIPTION	SAMPLE	SPT (N)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		43.0 - 100.0' Fernando Formation SILTSTONE / SANDSTONE	PB 1		PB			TIP OF SAMPLE BARREL DAMAGED (GRAVELS)
					RD			
46		DARK GREENISH GRAY 5GY4/1 85-95% FINES 15-5% SAND F/M MOIST / DAMP, MASSIVE-BEDDING INDISTINCT, TRACE MICACEOUS, BLACK (N-1) SILTY INCLUSIONS 5-15mm TRACE SULFUROUS ODOR	C-9	35 75/4.5	DR 325			CCI REFUSAL @ 10.5"
48		PHYSICAL CONDITION MASSIVE, FRIABLE HARDNESS AND STRENGTH, LITTLE WEATHERING - FRESH SOFT ROCK (VERY STIFF-HARD)			RD			MODERATE - HEAVY DRILL RIG CHATTER
50		43.0'-48.0' PREDOMINANTLY SILTSTONE WITH OCCASIONAL INTERBEDDED SANDSTONE (SANDS-SP), DENSE	PB 2		PB		2.3/3.0	SAMPLING PRESSURE 150 P.S.I. TIP OF SAMPLE BARREL DAMAGED (GRAVELS)
52		48.0-51.0' GRAVELS AND GRAVEL LENSES GRAVELS SUBANGULAR, 5-35mm DENSE						
		51.0 - 100' SILTSTONE / CLAYSTONE			RD			
54		DARK GREENISH GRAY 5GY4/1 100% FINES MOIST / DAMP, BEDDING INDISTINCT, CALCITE INCLUSION 1-2mm, SHELL FRAGMENTS	C-10	21 46	DR 325			
56		PHYSICAL CONDITION LITTLE FR... - MASSIVE FRIAB... ...SS AND STRENGTH, LITTLE WEA... ... FRESH SOFT ROCK (... STIFF - HARD)			RC			
58		SOIL CL... AND FIELD DESCRIPTIONS SHOWN ON TH... REPRESENT OUR INITIAL UNEDITED FIELD ... THESE LOGS HAVE NOT BEEN EDIT... ...CLUDE THE RESULTS OF LABOR-ATOR... ...AL CLASSIFICATION.	PB 3		PB		2.4/3.0	SAMPLING PRESSURE 150 P.S.I.
60		61.5-63.5' WEAK CALCAREOUS LAYER			RC			SLIGHT DRILL RIG CHATTER
62			C-11	24 84/6.5	DR 325			CCI REFUSAL @ 8.5" TIP OF SAMPLER DAMAGED 5 RINGS RECOVERED
64					RD			
66		— CONTINUED —	PB 4		PB		2.9/3.0	SAMPLING PRESSURE 160 P.S.I.
68								

SHEET 3 OF ___

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12-84 HOLE NO. 9-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		FERNANDO FORMATION SILTSTONE/CLAYSTONE — CONTINUED —	PB 4		PB			
70					RD			MODERATE DRILL RIG CHATTER
			C-12	49 50½	DR 325			CCI REFUSAL @ 9" 5.5 RINGS RECOVERED
72					RD			
74		— CONTINUED —						
76			PB 5		PB			SAMPLING PRESSURE 220 P.S.I. 1.1/3.0 SAMPLE DISTURBED DURING SAMPLING
78					RD			
80		DARK GREENISH GRAY 5GY4/1 100% FINES DAMP, BEDDING INDISTINCT, TRACE MICACEOUS, SHELL FRAGMENTS. MASSIVE, FRIABLE — LOW HARDNESS, FRIABLE - MODERATE STRENGTH, FRESH,	C-13	33 695	DR 325			CCI REFUSAL @ 11"
					RD			
		— CONTINUED —	C-14	33 75	DR 325			
86					RD			
88								
90			C-15	33 68	DR 325			
					RD			
92								

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-12-84 HOLE NO. 9-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92		FERNANDO FORMATION SILTSTONE / CLAYSTONE — CONTINUED —			RD			
94		SHELLS AND SHELL FRAGMENTS 1-35mm — CONTINUED — DARK GREENISH GRAY 5GY4/1 DAMP, MASSIVE, LOW HARDNESS, INDISTINCT BEDDING, FRIABLE TO MODERATE STRENGTH, FRESH	C-16	61 61/4	DR 325			CCI REFUSAL @ 10"
96								
98								
100			C-17	33 61	DR 325			T1330
		END OF BORING 100.0' TREMED A 3SAC/80 GALLON CEMENT SLURRY INTO BORING						

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 5 OF

Converse Consultants

Boring Log 9-7

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT SCRTD 83-1140-21 **DATE DRILLED** 5-10 **HOLE NO.** 9-7
LOCATION 7th STREET, 66' EAST OF FRANCISCO ST. ON 7th STREET **GROUND ELEV.** 293'
DRILLING CONTRACTOR Pitcher Drilling Co **LOGGED BY** Mark Schlutter **DEPTH TO GROUND WATER** ____
TYPE OF RIG Failing 1500 **HOLE DIAMETER** 4 7/8" **HAMMER WEIGHT AND FALL** 325# @ 18" / 140# @ 30"
SURFACE CONDITIONS A.C. STREET SURFACE **TOTAL DEPTH** 100.0' **NO. CORE BOXES** ____

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0 - 0.8 CONCRETE GUTTER			C			STARTED DRILLING @ 0720
2	CL	0.8 - 6.0 SILTY CLAY (FILL) DARK YELLOWISH BROWN 10YR4/2 90-100% FINES 10-0% SAND (FINE) VERY MOIST, VERY SOFT - SOFT MEDIUM PLASTICITY, MOTTLED, INCLUDES SUBANGULAR SILTSTONE FRAGMENTS - SOFT & SLIGHTLY MOIST	C-1	1 FOR 12"	DR 325			
4					RD			ROTARY WASH TRI CONE BIT 5' OF CASING BENTONITE ADDED TO DRILLING FLUID
6	SC	6.0 - 11.0 SILTY CLAYEY SAND MODERATE BROWN 5YR3/4 65-75% SAND F/C 35-25% FINES MOIST, MEDIUM DENSE, LOW PLASTICITY, TRACE GRAVEL 5-15mm, SAND SUBANGULAR FINE - COARSE						
8			C-1	7 / 14 / 20	CC 140		1.3/1.5	
10			C-2	17	RD			Removed [illegible] REPLACE WITH [illegible]
12	SM	11.0 - 16.5 SILTY SAND MODERATE BROWN 5YR4/4 75-85% SAND F/C 25-15% FINES MOIST, MEDIUM DENSE, TRACE MICACEOUS, TRACE GRAVEL 5-15mm, SAND SUBANGULAR FINE-COARSE GRAVEL CONSISTS OF WEATHERED GRANITIC ROCK (D.G.)			RD			
14			C-3	13 / 27	CC 140			
16					RD			
18	SP/SW	16.5 - 20.0 SAND DUSKY YELLOW 5Y6/4 95-100% SAND FINE - COARSE 5-0% FINES (SILT) SLIGHTLY MOIST, MEDIUM DENSE - DENSE, TRACE MICACEOUS, SAND IN LAYERS OF (SP) AND (SW), POORLY GRADED TO WELL GRADED, CLEAN SANDS, TRACE GRAVEL WITH OCCASIONAL GRAVEL LENSES	J-2	16 / 22 / 50/5"	CC 140		1.4/1.4	SPT REFUSAL @ 17"
20					RD			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 1 OF ____

PROJECT SCRTO 83-1140-21 DATE DRILLED 5-10-84 HOLE NO. 9-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	SW	20.0' – 26.5' SAND DUSKY YELLOW 5Y6/4 90-100% SAND F/C 10-0% GRAVEL SLIGHTLY MOIST, DENSE, WELL GRADED, SUBANGULAR SAND GRAINS, LITTLE GRAVEL 5-35mm CLEAN SANDS (LITTLE TO NO BINDER)	C-4	28 75	DR 325			
22					RD			
			C-5	101 90/3	DR 325			CCI REFUSAL @ 9" DISTURBED SAMPLE LOOSE / FELL OUT OF SAMPLER 3 RINGS TOTAL
24					RD			
26	SM	26.5' – 29.0 SILTY SAND LIGHT OLIVE GRAY 5Y5/2 60-70% SAND F/m 40-30% FINES MOIST, DENSE, CLAY BINDER, TRACE GRAVEL, SAND GRAINS SUBANGULAR	J-3	18 29 52	SS 140		1.4/1.5	
28					RD			
29	SW	29.0 – 36.0 SAND YELLOWISH GRAY 5Y7/2 85-100% SAND F/C 15-0% GRAVEL 5-30mm MOIST, DENSE, WELL GRADED, SUBANGULAR GRAINS, CLEAN SANDS (LITTLE TO NO BINDER)		42 50/3	DR 325			CCI REFUSAL @ 9" SAMPLE NOT RECOVERED
32					RD			
33			C-6	60	DR 325			CCI REFUSAL @ 9" 5 RINGS RECOVERED
36		36.0 – 100.0' FERNANDO FORMATION CLAYSTONE / SILTSTONE MODERATE OLIVE BROWN 5Y4/4 (36→38') (WEATHERED) DARK GREENISH GRAY 5G4/1 (38'→) 100% FINES LOW-MEDIUM SILTSTONE PLASTICITY MEDIUM-HIGH CLAYSTONE PLASTICITY MOIST/DAMP, MASSIVE-BEDDING INDISTINCT, TRACE CALCITE INCLUSIONS PHYSICAL CONDITION MASSIVE, SOFT ROCK (VERY STIFF), FRIABLE, LITTLE WEATHERING → FRESH 36'-38' WEATHERED CONTACT ZONE	J-4	21 25 37	SS 140			
38					RD			
40			C-7	30 79	DR 325			
42					RD			
44			C-8	51 49	DR 325			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 2 OF ____

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-10-84 HOLE NO. 9-7

DEPTH 44	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		FERNANDO FORMATION - CONTINUED CLAYSTONE / SILTSTONE						SLIGHT RIG CHATTER
		45': SMALL CALCAREOUS LAYER OR NODULE, HARD			RD			
46								
				17	SS 140		1.5/1.5	
48		TRACE MICACEOUS CONTINUED DARK GREENISH GRAY 5GY4/1	J-5	30 44				
					RD			
50		OCCASSIONAL SHELL FRAGMENTS 1-2mm	C-9	23 50/1	DR 325			
52					RD			
					DR 325			
54					RD			
56								
		57.0-57.5' CALCAREOUS LAYER						SLIGHT TO MODERATE DRILL RIG CHATTER, SAMPLE 6" LOWER BELOW CEMENTED LAYER
58			J-6	10 21 31	SS 140		1.5/1.5	
					RD			
60			PB 1		PB			SAMPLING PRESSURE 120 P.S.I. 2.0/2.0
62					RD			
64		— CONTINUED —	C-11	37 50/4.5	DR 325			CCI REFUSAL @ 10.5"
66					RD			
68			PB 2		PB			SHEET 3 OF

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

PROJECT SCRTD 83-1140-21 DATE DRILLED ______ HOLE NO. 9-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68		FERNANDO FORMATION CLAYSTONE / SILTSTONE -CONTINUED-	PB 2		PB			SAMPLING PRESSURE 120 P.S.I. MODERATE CHATTER DURING SAMPLING, END OF TUBE DAMAGED, PORTION OF SAMPLE LOST.
70		69.0 - 70.0 CALCAREOUS LAYER PRODUCED DRILL RIG CHATTER AND DAMAGED PITCHER SAMPLING TUBE,			RD			
		PHYSICAL CONDITION DARK GREENISH GRAY 5GY4/1 DAMP, LOW HARDNESS, VERY STIFF-HARD, MASSIVE, BEDDING INDISTINCT, WEAK STRENGTH, FRESH	C-12	31 63	DR 325			
72		OCCASIONAL SHELL FRAGMENTS AND CALCAREOUS LAYERS AND CONCRETIONS			RD			
74			PE 3		PB			SAMPLING PRESSURE 160 P.S.I. 2.7/3.0
76								
78					RD			
			C-13	33 50	DR 325			
80		— CONTINUED —			RD			
82			PB 4		PB			SAMPLING PRESSURE 160 P.S.I. 2.2/3.0
84					RD			
			C-14	28 60/5"	DR 325			CCI REFUSAL @ 11.5"
86					RD			
88			PE 5		PE			SAMPLING PRESSURE 160 P.S.I. 2.7/3.0
90								
92					RD			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 4 OF ____

PROJECT SCRTD 83-1140-21 DATE DRILLED 5-10-84 HOLE NO. 9-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (BLOWS)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92		FERNANDO FORMATION CLAYSTONE / SILTSTONE - CONTINUED -			RD			
94		94.0 - 94.3 CALCAREOUS LAYER HARD AND STRONG						MODERATE DRILL RIG CHATTER
96		PHYSICAL CONDITION DARK GREENISH GRAY 5GY4/1 DAMP, MASSIVE, LOW HARDNESS, BEDDING INDISTINCT, WEAK TO MODERATE STRENGTH, FRESH OCCASIONAL SHELL FRAGMENTS AND CALCAREOUS LAYERS	C-15	30 54	DR 325			
98					RD			
100			C-16	60	DR 325			CCI REFUSAL @ 10.5"
		END OF BORING 100.0' 5-11-84 TREMIED APPROX. 50 GALLONS OF 3 SAC CEMENT SLURRY INTO BORING						

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 5 OF ____

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 10-1

Proj: DESIGN UNIT A170 Date Drilled 1/15-16/85 Ground Elev. ____

Drill Rig Failing 1500 Logged By M. Schluter Total Depth 111.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	BLOWS [6"]	DRILL MODE	REMARKS
0		0.0-0.5	Concrete Gutter			C	
	GP	0.5-1.0	SANDY GRAVEL: gray, 3/4" base				
	SM	1.0-6.0	FILL				
2			SILTY SAND: brown to dark brown, fine to coarse sand, moist, med. dense to dense, trace fine gravel	C-1	6 7	DR 325	
4						RD	
6	SM	6.0-16.5	ALLUVIUM SILTY SAND: brown, moist, dense, trace clay binder, trace fine gravel				
8				J-1	13 18 18	SS	0.4/1.5 Recovery
						RD	
10				C-2	16 22	DR 325	
12			medium to coarse interbedded sand lenses with some fine gravel			RD	
				PB 1		PB	2.4/2.5 Recovery
14							
16						RD	
	SW	16.5-28.5	SAND: light brown, moist, dense, little gravel				slight rig chatter
18				J-2	31 60	SS	SPT refusal @ 12" 0.9/1.0 recovery
20						RD	

Sheet 1 of 5

Project DESIGN UNIT A170 Date Drilled 1-15-85 Hole No. 10-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	16.5-28.5 SAND: (continued)			RD	
		increasing fine gravel content				
22			C-3	31 / 50	DR 325	refusal @ 9"
					RD	
24		gravel and gravel lenses				moderate to heavy chatter
26						
28			J-3	73	SS	SPT refusal @ 6" 0.2/0.5 recovery
		BEDROCK - 28.5-111.0 FERNANDO FORMATION SILTSTONE and CLAYSTONE: light olive gray, moist, friable to low hardness, friable strength, deep to moderate weathering,			RD	
30						
32		31.5-32.5' cemented layer, pale green, calcareous cemented siltstone			PB	moderate rig chatter 31.5-32.5' high drilling resistance 32.0' no recovery,
		moist	PB-2		PB	2.5/2.5 recovery
34						
36					RD	
38		olive gray; moist		19 / 26 / 34	SS	no recovery
		Physical Condition: low hardness, weak, moderate to little weathering, little fractured to massive.			RD	
40						
42		occasional light brown oxidation discoloration along fractures	C-4	16 / 31	DR 325	
44					RD	

Sheet 2 of 5

Project DESIGN UNIT A170 Date Drilled 1/15-16/85 Hole No. 10-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		28.5-111.0 SILTSTONE/CLAYSTONE: (cont.)			RD	
46						
48			J-4	14 22 31	SS	1.5/1.5 recovery
					RD	
50			C-5	36 50	DR 325	refusal @ 10"
52					RD	
		olive gray; moist	PB 3		PB	2.3/2.5 recovery
54		Physical Condition: low hardness, weak, little weathering, little fractured to massive, bedding indistinct			RD	
56						
58			J-5	13 24 40	SS	1.5/1.5 recovery 1-15-85
60					RD	1-16-85
62			C-6	27 50	DR 325	refusal at 10"
64					RD	
66						
68		small calcareous inclusions, at times forming thin hard layers	J-6	17	SS	

Project DESIGN UNIT A170 Date Drilled 1-16-85 Hole No. 10-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		28.5-111.0 SILTSTONE/CLAYSTONE: (cont.)	J-6	40 50	SS	SPT refusal @ 16"
					RD	
70			C-7	60 75	DR 325	refusal @ 9"
72		72.0-72.5' calcareous layer cemented siltstone			RD	moderate rig chatter
74			PB 4		PB	2.5/2.5 recovery
		predominantly siltstone decreasing claystone				
76					RD	
78			J-7	24 41 50	SS	SPT refusal @ 17" 1.5/1.5 recovery
80					RD	
82			C-8	48 60	DR 325	refusal @ 9"
84					RD	
86						
88			J-8	13 20 42	SS	1.5/1.5 recovery
90					RD	
		small calcareous inclusions	C-9	48 50	DR 325	refusal @ 10"
92					RD	

Project DESIGN UNIT A170 Date Drilled 1-16-85 Hole No. 10-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		28.5-111.0 SILTSTONE/CLAYSTONE: (cont.) occasional small shell fragments	PB 5		PB	2.5/2.5 recovery
94						
					RD	
96						
98			J-9	50	SS	0.5/0.5 recovery SPT refusal @ 6"
					RD	
100		olive gray to dark greenish gray; slightly moist to moist				
		Physical Condition: low hardness, weak, little weathering, massive, bedding indistinct; occasional calcareous inclusions, predominantly siltstone				
102			C-10	37 50	DR 325	refusal @ 9"
					RD	
104						
106						
108			J-10	16 25 32	SS	
					RD	
110		trace shell fragments	C-11	37 70	DR 325	refusal @ 9"
	B.H.	111.0' Terminated hole				1-16-85
112		Flushed Boring and Installed Piezometer				
114						
116						

Perforated	Non Perforated
10-30'	0-10'
50-70'	30-50'
90-111'	70-90'

Backfilled with pea gravel, installed casing and cap.

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 10-2

Proj: DESIGN UNIT A170 **Date Drilled** 1/17-18/85 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** M. Schluter **Total Depth** 120'

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

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.6	Concrete Gutter			C	
	GP	0.6-1.0	SANDY GRAVEL: gray, imported 3/4" base				
			ALLUVIUM				
2	ML	1.0-10.0	CLAYEY SILT: moderate brown, with fine to coarse sand, moist stiff to very stiff, trace fine gravel, slightly porous	C-1	7 9	DR 325	
						RD	Rotary Drill, 5' casing installed, tri cone bit
4							
6							
			decreasing sand content, light gray to moderate yellowish brown				
8				J-1	6 8 12	SS	1.2/1.5 recovery
						RD	
10	SP	10.0-15.0	SAND: yellowish brown, fine sand with trace silt, slightly moist to moist, medium dense to dense	C-2	12 18	DR 325	
						RD	
12						PB	0.0/2.5 recovery sample fell out
14						RD	
	SP	15.0-20.0	SAND: yellowish gray, fine to medium sand, slightly moist, dense, trace fine gravel, occasional fine gravel lenses with coarse sand				
16				PB 1		PB	1.5/2.0 recovery refusal @ 9"
18				J-2	44 50	SS	0.6/0.8 recovery
20							

Sheet 1 of 6

Project DESIGN UNIT A170 Date Drilled 1/17/85 Hole No. 10-2

DEPTH	USCS	MATERIAL CLASSIFICATION		SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	20.0-24.5	SAND: yellowish gray, fine to coarse sand with gravel, slightly moist, dense to very dense, with fine gravel layers and lenses			RD	occasional moderate to heavy rig chatter
22				C-3	70 74	DR 325	disturbed sample refusal @ 10"
						RD	
24							
		BEDROCK - 24.5-120.0'	FERNANDO FORMATION SILTSTONE/CLAYSTONE: light olive gray to moderate brown (oxidized), slightly moist to moist				
26							
28			Physical Condition : low hardness, deep to moderate weathering, oxidation discoloration streaks and along fractures, friable to weak	J-3	10 12 14	SS	1.5/1.5 recovery
						RD	
30			Weathered contact zone to about 45-47' depth	C-4	27 44	DR 325	
						RD	
32				PB 2		PB	2.0/2.5 recovery
34			decreasing oxidation discoloration				
						RD	
36							
38				J-4	11 14 23	SS	1.5/1.5 recovery
						RD	
40							
42			trace gravel to 3"	C-5	47 36	DR 325	
						RD	
44							

Project DESIGN UNIT A170 Date Drilled 1/17/85 Hole No. 10-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		24.5-120.0 SILTSTONE/CLAYSTONE: (cont.)			RD	
46						
48		olive gray to dark greenish gray; moist Physical Condition: low hardness, weak, little weathered to fresh, predominantly siltstone, trace shell fragments	J-5	14 28 34	SS	1.5/1.5 recovery
					RD	
50			C-6	21 37	DR 325	
52					RD	
			PB 3		PB	2.5/2.5 recovery
54						
56					RD	
		57.5-58.0' calcareous cemented layer				rig chatter
58				18 31 34	SS	sample not recovered
60					RD	
62			C-7	31 50	DR 325	refusal @ 11"
					RD	
64						
66						
68			J-6	15	SS	

Project DESIGN UNIT A170 Date Drilled 1/17-18/85 Hole No. 10-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		24.5-120.0 SILTSTONE/CLAYSTONE: (cont.)	J-6	26 39	SS	1.5/1.5 recovery
					RD	
70			C-8	31 63	DR 325	
					RD	
72			PB 4		PB	2.4/2.5 recovery
74						
76					RD	
78		predominantly siltstone, dark greenish gray, moist, low hardness, weak, little weathered to fresh, occasional calcareous inclusions	J-7	18 37 48	SS	1.5/1.5 recovery
80					RD	
82			C-9	31 60	DR 325	refusal @ 11"
84					RD	
86						
88			J-8	20 41 53	SS	1.5/1.5 recovery
					RD	
90			C-10	35 60	DR 325	refusal @ 11" 1-17-85
92					RD	1-18-85

Project DESIGN UNIT A170 Date Drilled 1-18-85 Hole No. 10-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		24.5-120.0 SILTSTONE/CLAYSTONE: (cont.)	PB 5		PB	2.5/2.5 recovery
94						
					RD	
96						
98			J-9	19 30 42	SS	1.5/1.5 recovery
100					RD	
102		predominantly siltstone, dark greenish gray, moist, low hardness, weak, fresh, trace calcareous inclusions	C-11	31 56	DR 325	refusal @ 11"
					RD	
104						
106						
108			J-10	27 51	SS	SPT refusal @ 12" 1.0/1.0 recovery
					RD	
110			C-12	34 60	DR 325	refusal @ 11"
					RD	
112			PB 6		PB	2.2/2.5 recovery
114						
					RD	
116						

Project DESIGN UNIT A170 Date Drilled 1-18-85 Hole No. 10-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		24.5-120.0 <u>SILTSTONE/CLAYSTONE</u>: (cont.)			RD	
118						
			J-11	31 51	SS	SPT refusal @ 11" 1.0/1.0 recovery
120					RD	1-18-85
	B.H.	120.0' - Terminated Hole Flushed Boring and Installed Piezometer <u>Perforated</u> / <u>Non Perforated</u> 20-40' / 0-20' 60-80' / 40-60' 100-120' / 80-100' backfilled with pea gravel, installed casing and steel cover				
122						
124						
126						
128						
130						
132						
134						
136						
138						
140						

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 10-3

Proj: DESIGN UNIT A170 **Date Drilled** 1-18/19-85 **Ground Elev.**

Drill Rig Failing 1500 **Logged By** M. Schluter **Total Depth** 101.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.3 Asphalt Pavement				
	SM	0.3-1.5 misc. debris and silty sand including brick, concrete, small cobbles			C	
2	SP	ALLUVIUM 1.5-8.5 SAND moderate brown, yellowish brown fine to medium sand, trace fines	C-1	7 16	DR 325	
4		slightly moist, medium dense to dense			A	
6						
8			J-1	12 18 23	SS	1.3/1.5 recovery
					RD	
10	SM SP	8.5-16.0 SILTY SAND/SAND yellowish brown fine sand with fines, slightly moist, dense, occasional sand interbeds	C-2	34 50	DR 325	refused at 11" 2.0 tsf. pocket pene. Rotary drill 1-18-85
					RD	
12			PB 1		PB	1-19-85 2.4/2.5 recovery
14						
					RD	
16	SP	16.0-28.0 SAND yellowish brown to yellowish gray fine to medium sand with trace of fines, moist, dense, occasional fine to coarse sand layers				
18				44 50	SS	SPT refusal @ 10"
					RD	
20						

Project DESIGN UNIT A170 Date Drilled 1-19-85 Hole No. 10-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	16.0-28.0 SAND: (cont.)			RD	
		trace coarse sand/fine gravel, predominantly yellowish gray	C-3	60 50	DR 325	refusal @ 11"
22					RD	
24						
26						
		27.6' - gravel/coarse sand	J-2	35 45 34	SS	1.5/1.5 recovery
28		BEDROCK - FERNANDO FORMATION				
		28.0-101.0 SILTSTONE with CLAYSTONE: yellowish brown (oxidized), yellowish gray and moderate brown, predominantly siltstone, moist to slightly moist,			RD	
30			C-4	14 29	DR 325	
		Physical Condition : bedding indistinct, highly weathered, oxidation discoloration and streaks, friable hardness and strength, little fractured			RD	
32			PB 2		PB	2.4/2.5 recovery
34						
		weathered contact zone to about 36 to 37' depth			RD	
36						
		dark greenish gray, little weathered, weak, low hardness	J-3	15 27 41	SS	1.5/1.5 recovery
38					RD	
40						
		trace small calcareous inclusions	C-5	32 46	DR 325	
42					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A170 Date Drilled 1-19-85 Hole No. 10-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		28.0-101.0 SILTSTONE with CLAYSTONE: cont.			RD	
46						
48			J-4	14 25 31	SS	1.5/1.5 recovery
					RD	
50			C-6	37 57	DR 325	refusal @ 11"
					RD	
52			PB 3		PB	2.4/2.5 recovery
54					RD	
56		Physical Condition: dark greenish gray, moist, low hardness, weak, little weathered to fresh, occasional calcareous inclusions				
58			J-5	14 22 34	SS	1.5/1.5 recovery
60					RD	
			C-7	31 64	DR 325	refusal @ 11"
62					RD	
64						
66						
68			J-6	16 35	SS	1.5/1.5 recovery

Project DESIGN UNIT A170 Date Drilled 1-19-85 Hole No. 10-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		28.0-101.0 SILTSTONE with CLAYSTONE: cont.	J-6	42	SS	
					RD	
70			C-8	33 60	DR 325	refusal @ 10"
					RD	
72			PB 4		PB	2.5/2.5 recovery
74						
76					RD	
78			J-7	18 33 48	SS	1.2/1.5 recovery
		predominantly siltstone, dark greenish gray, low hardness, weak, moist			RD	
80	50-55°					
82			C-9	33 60	DR 325	refusal @ 11"
					RD	
84						
86						
88			J-8	22 38 52	SS	1.3/1.5 recovery
					RD	
90		cemented, calcareous siltstone layer 90'-91', hard		100	DR 325	refusal @ 4.5", no recovery, heavy rig chatter
92					RD	

Project DESIGN UNIT A170 Date Drilled 1-19-85 Hole No. 10-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		28.0-101.0 SILTSTONE w/ CLAYSTONE (cont.)	PB 5		PB	2.4/2.5 Recovery
94–96					RD	
97–98			J-9	20 38 58	SS	Recovery 1.4/1.5
99					RD	
100			C-10	48 60	DR 325	Refusal at 10" 1-19-85
101	B.H.	101 Terminated Hole Flushed Boring and Installed Piezometer Perforated: 20-60', 80-100' Non-Perforated: 0-20', 60-80' Backfill with pea gravel, installed casing and steel cover.				

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 11A

Proj: DESIGN UNIT A170 **Date Drilled** 11-30-83 **Ground Elev.** 290'

Drill Rig BUCKET **Logged By** J. Stellar **Total Depth** 90'

Hole Diameter 33" **Hammer Weight & Fall**

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 AC 0.4-0.8 CONCRETE 0.4-3.0 FILL mixed silty sand and clayey sand; with gravel to 1"; tan to orange brown; moist; soft to medium dense				Hole stands very well 0-90' No gas registered on monitor
2						
3		3.0-7.0 GRAVELLY SAND: light greenish gray to tan; slightly moist; medium dense to dense; cobbles to 12"				
4						
6						
7		7.0-90.0 BEDROCK-PUENTE FORMATION: SILTSTONE: light orange brown to tan; slightly moist; closely fractured; friable; weak strength; deep to moderate weathering; deep and thorough discoloration				
8						
10						
12						
14						
16						
18						
20						

Sheet 1 of 4

Project DESIGN UNIT A170 Date Drilled 11-30-83 Hole No. 11A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20		7.0-90.0 BEDROCK-PUENTE FORMATION SILTSTONE: (Cont.)				
		21.0 becomes blue gray; moderately hard and moderately strong				
22						
24						
26						
28		very minor seep at 28.0				strike E-W, dip 42° S @ 30'; shear zone ½" silicified siltstone completely healed
		29-30' shear zone; deep weathering & discoloration; friable hardness and strength				
30						
32						
34						
36						
38						
40						beds @ 40' N 85° E, dip 33° S
42						
44						

Project DESIGN UNIT A170 Date Drilled 11-30-83 Hole No. 11A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		7.0-90.0 BEDROCK-PUENTE FORMATION SILTSTONE: (cont.) moderately hard; moderately strong				
46						
						weeping water droplets @ 47-47.3'
48						
		3-4" layers of hard; strong silicious siltstone 49-50'				
50						
52						
54						
56						
58						
60						beds @ 60' N 85 E dip 45° S
62						
64						
66						
68						

Project DESIGN UNIT A170 Date Drilled 11-30-83 Hole No. 11A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		7.0-90.0 BEDROCK-PUENTE FORMATION SILTSTONE: (cont.) moderately hard, moderately strong				
70						bedding attitude @ 70', N 85° E, dip 34° S
72						
74						
76						
78						
80						Beds @ 80' N 85° E dip 35° S
82						
84						
86						
88						
90	B.H.	90.0 Terminated Hole case top 20' Downhole Observer - J. R. Stellar				
92						Sheet 4 of 4

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Boring Log 11-1

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME.
CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/8-9/83 HOLE NO. 11-1
LOCATION Alvarado & Wilshire ~200' S of Wilshire GROUND ELEV. 266.2'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS Asphalt roadway TOTAL DEPTH 75.0' NO. CORE BOXES 2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC	0.0-0.4 asphalt 0.4-1.0 Concrete			GB			8:30 am moved on hole set up began drilling @ 10 am. Set 5" steel casing to 4'. Hole initiated with a 7" [illegible] barrel for [illegible] cap [illegible].
	CL	OLD ALLUVIUM 1.0-8.5 SILTY CLAY: mottled moderate yellowish brown (10YR 5/4) and dark yellowish brown (10YR 4/2); moderately plastic, trace fine sand; stiff; moist.						
2.0					RC			
4.0								
			S-1	2 2 1	SS			0.5/1.5 33% recovery
6.0					RC			
8.0								
		8.5-11.5 PUENTE FORMATION: CLAYEY SILTSTONE interbedded with SILTY SANDSTONE: mottled greenish grey (5GY 6/1) and moderate brown (5YR 4/4); non-plastic to moderately plastic, moist; contains cemented sand zones; moderate weathering						
10.0			S-2	9 6 11	SS			[illegible]/1.5 50% recovery
		11.5-39.0 INTERBEDDED SILTSTONE SANDSTONE and CLAYSTONE: sandstone - medium greenish grey (5GY 5/1) 95% very fine sand 5% silt; siltstone - dark greenish grey (5GY 4/1) 95% non plastic fines; claystone - olive black (5Y 2/1) 95% moderately plastic fines; sulfur odor present in clay with minor small wood fragments; thinly bedded 1/8" to 1/4"			RC			
12.0			C-1	8 11 19	OR			1.5/1.5 100% recovery
14.0					RO			minor circulation loss
			S-3	9 18 12	SS			1.5/1.5 100% recovery
16.0					RO			
18.0		Physical Condition: little fracturing to massive; [illegible] friable hardness; plastic to friable strength; fresh to [illegible] weathered						
20.0								

SHEET 1 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/8-9/83 HOLE NO. 11-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0		11.0-39.0 INTERBEDDED SILTSTONE, SANDSTONE, and CLAYSTONE: cont.	PB-1		PB			2.5/2.5 100% recovery 200 psi.
22.0			J-4	13 18 26	SS			1.5/1.5 100% recovery
24.0					RD			
26.0	45°							
28.0		27.0 - 39.0 grading from thinly to thickly bedded 1/2" to 1 1/2"	J-5	20 30 43	SS			1.5/1.5 100% recovery
					RD			
30.0			C-2	28 50	OR			1.0/1.0 100% recovery
					RD			
32.0			J-6	18 28 46	SS			1.5/1.5 100% recovery
34.0					RD			
36.0		sand decreasing ↓						
38.0			J-7	28 46 50-4"	SS			1.3/1.3 100% recovery
					RD			
40.0		39.0 - 57.6 SANDY SILTSTONE: olive black (5Y 2/1); 95% nonclastic fines 5% very fine sand; micaceous; contains irregular sand pockets. Physical Condition: little to massive fracturing; soft to friable hardness; plastic to friable strength; fresh to little weathered.	PB-2		PB			2.5/2.5 100% recovery 200 psi.
42.0			J-8	15 30 50-4"	SS			1.3/1.3 100% recovery
44.0								

SHEET 2 OF 5

PROJECT 82-1101-81 DATE DRILLED 2/8-9/83 HOLE NO. 11-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		39.0-57.6 SANDY SILTSTONE: cont.			RD			
46.0								
			J-9	20 39 50-4.5"	SS			1.4/1.4 100% recovery
48.0					RD			
50.0								
			C-3	30 55	DR			1.0/1.0 100% recovery
					RD			
52.0								
			J-10	17 30 50-5"	SS			1.4/1.4 100% recovery
					RD			
54.0								
56.0								3pm eyebolt on 140 lb hammer broke down 1 hr.
			J-11	17 50	SS			1.0/1.0 100% recovery
58.0		57.6-62.0 CLAYSTONE: greyish olive, (10Y 4/2); high plasticity; moist Physical Condition: little fracturing to massive; soft to friable hardness; plastic to friable strength; fresh			RD			
60.0			PB-3		PB			2.5/2.5 100% recovery 200 p.s.i.
62.0		62.0-75.0 INTERBEDDED SILTSTONE, CLAYSTONE and SANDSTONE: greyish olive, (10Y 4/2); 5" interbeds						
			J-12	24 50	SS			1.0/1.0 100% recovery
64.0		Physical condition: little fracturing to massive; soft to friable hardness; plastic to friable strength; fresh; parts easily along sand beds.						
66.0								
			J-13	2" 11/4	SS			1.3/1.3 100% recovery
68.0								

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/8-9/83 HOLE NO. 11-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		62.0-75.0 INTERBEDDED SILTSTONE, CLAYSTONE and SANDSTONE: cont.		50-4"	RO			
70.0			C-4	29 67	DR RO			1.0/1.0 100% recovery
72.0								
74.0			J-14	21 29 52	SS			
76.0		B.H. 75.0' Terminated hole; installed 75' of 2" ABS slots 35'-75' 1' bentonite seal 2½'-3½' perforations wrapped with filter fabric.						H_2O @ 3' 7am 2/9/83 Converse sampler driven with 300 lb. downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 11-1

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/8-9/83.

OVERBURDEN DEPTH (FT.) 0 TO 8.5'.

BEDROCK DEPTH (FT.) 8.5' TO 75' (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 8' DATE 2/9/83 during drilling; 7.2 DATE 4/4/83.

GAS SULFUR; DEPTH FIRST NOTICED 15', DATE 2/8/83.

E-LOG NO.

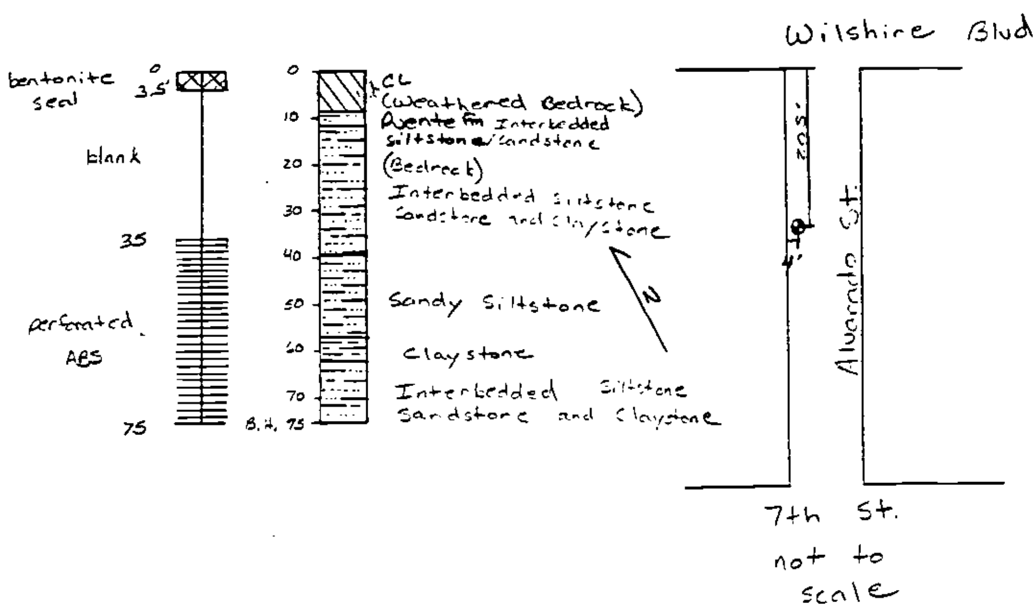
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" 0 TO 75'.

GROUND ELEVATION REF. RTO SITE PLAN AL-A-1 (1" = 40') Weese & Assoc 2/28/83

SKETCH



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Boring Log 11-2

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-21 DATE DRILLED 2/9-10/83 HOLE NO. 11-2
LOCATION Alvarado & Wilshire GROUND ELEV. 264.5'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 4⅞ HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS asphalt roadway TOTAL DEPTH 75' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC	0.0-0.3 Asphalt 0.3-0.9 Concrete OLD ALLUVIUM						10:30 am move and set up, 12 noon start drilling cased w/ 5" steel casing to 11.5'
	SC	0.9-6.0 CLAYEY SAND: mottled moderate yellowish brown (10YR 5/4), light brown (5YR 5/6); 3% coarse sand, 80% fine sand; 17% moderately plastic fines; loose; moist to wet.						
2.0								
4.0								
			J-1	3 6 3	SS			1.0/1.5 66% recovery
6.0	CL	6.0-7.5 SANDY CLAY: olive black (5Y 2/1); 15% medium to fine sand; 85% moderately plastic fines; firm to stiff; moist.			RD			drilling harder
8.0	CL	7.5-25 SILTY CLAY: olive black (5Y 2/1); 98% moderately plastic fines; 2% medium rounded gravel; stiff; moist.						
			C-1		DR			4/6 blows 1' drive 1.0/1.0 100% recovery
10.0					RD			
			J-2	3 4 5	SS			1.5/1.5 100% recovery
12.0					RD			
14.0								
16.0			J-3	6 8 6	SS			1.5/1.5 100% recovery
18.0					RD			
			C-2		DR			pocket pen 2.0 disturbed
20.0								

SHEET 1 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/9-10/83 HOLE NO. 11-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	CL	7.5 - 25.0 SILTY CLAY : cont.			RD			
			J-4	2 3 3	SS			1.5/1.5 100% recovery
22.0					RD			
24.0		24.5-25.0 gravelly lense						
	SM	25.0-28.0 SILTY SAND : greyish olive green (5GY 3/2) 30% non-plastic silt; 70% fine sand; medium dense; moist.						rig chatter
26.0			J-5	5 7 10	SS			1.5/1.5 100% recovery
28.0		PUENTE FM 28.0-75.0 SILTSTONE : dark greenish grey (5GY 4/1) interbedded w/ CLAYSTONE and fine SANDSTONE interbeds from 1/8" to 6"			RD			
30.0			C-3		DR			0.5/1.0 50% recovery pocket pen 3.5
		Physical Condition: massive to little fracturing friable hardness and strength fresh.			RD			
32.0		black sand interbeds strong sulfur odor	J-6	7 8 22	SS			1.5/1.5 100% recovery
					RD			
34.0		34.0 claystone and sandstone lenses decrease						
36.0	∠ 50°	color change to olive grey (5Y 3/2), decrease in sand lenses	J-7	7 21 33	SS			1.5/1.5 100% recovery
38.0					RD			
					PB			
40.0			PB-1					2.5/2.5 100% recovery
43.0		minor gravels in siltstone, sub rounded	J-8	13 25 50-4.5"	SS			1.4/1.4 100% recovery
44.0								

SHEET 2 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/9-10/83 HOLE NO. 11-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		28.0-75' SILTSTONE interbedded with CLAYSTONE and SANDSTONE			RO			
46.0		46.0' 3" cemented zone concretion	J-9	9 33 50-4"	SS			1.3/1.3 100% recovery
48.0					RO			
			C-4		OR			26/70 1.0/1.0 100% recovery
50.0		3" concretions			RO			
52.0			J-10	19 24 46	SS			1.5/1.5 100% recovery
					RO			
54.0								
56.0	45°	Sample J-11: thin interbedded sand lenses to 1½"	J-11	25 43 50-4"	SS			1.3/1.3 100% recovery
58.0					RO			
60.0			PB-2		FB			2.5/2.5 100% recovery
62.0			J-12	21 29 50-4.5"	SS			1.4/1.4 100% recovery
64.0								
66.0			J-13	15 29 50-4.5"	SS			1.4/1.4 100% recovery
68.0								

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/9-10/83 HOLE NO. 11-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		28.0-75.0 SILTSTONE interbedded w/ CLAYSTONE and SANDSTONE: cont.			RC			
				20	DR			1.0/1.0 100% recovery
70.0			C-5	59				2/9/83 2/10/83
					RC			H_2O @ 9' 7am
72.0		71-75 thin interbeds (1/4") of sandstone siltstone, and claystone; greenish grey to olive black.						
74.0			J-14	14 / 36 / 50-4"	SS			1.3/1.3
76.0		B.H. 75.0' Terminated hole; grouted to 6" below surface topped off with cold patch asphalt compacted in 1" lifts.						Converse sampler driven with 300 lb downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 11-2

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/9-10/83.

OVERBURDEN DEPTH (FT.) 0 TO 28'.

BEDROCK DEPTH (FT.) 28 TO 75 (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 9' during drilling DATE 2/10/83; ____ DATE ____.

GAS SULFUR; DEPTH FIRST NOTICED 31', DATE 2/9/83.

E – LOG NO.

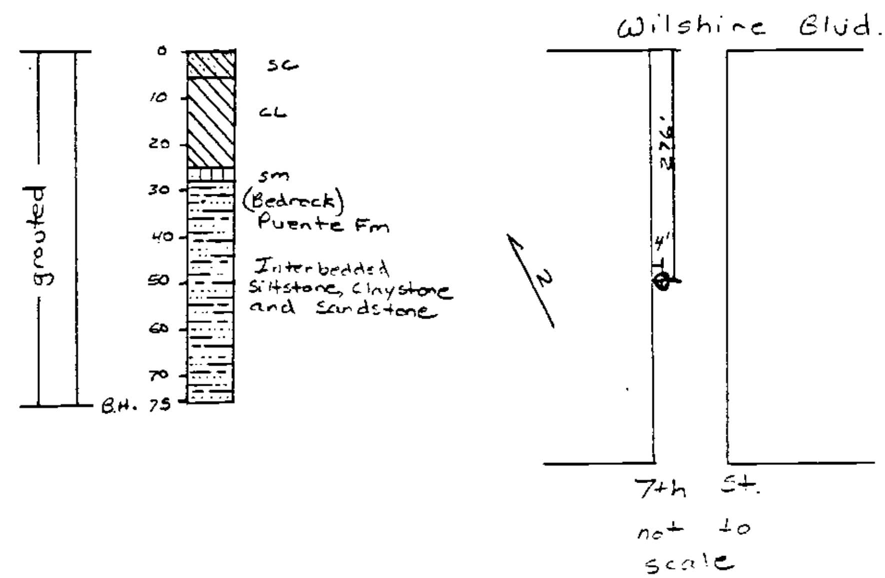
DOWN–HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTD SITE PLAN AL-A-1 (1"=40') Wesse & Assoc. 2/28/83

SKETCH



SHEET 5 OF 5

Converse Consultants

Boring Log 11-3

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/10-11/83 HOLE NO. 11-3
LOCATION Westlake and 7th GROUND ELEV. 271.5
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER ___
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8 HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS Asphalt pavement TOTAL DEPTH 75.0 NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
		OLD ALLUVIUM						
0.0	AC	0.0-0.3 Asphalt			GB			10:30 am move and set up 11:30 drilling start. 5" steel casing to 11'
	SC	0.3-2.5 CLAYEY SAND: moderate yellowish brown (10YR 5/4). 85% fine to medium sand 15% moderately plastic clay; moist.						
2.0								
	CL	2.5-5.0 SILTY CLAY: moderate yellowish brown (10 YR 5/4); 100% moderately plastic fines; firm; moist.			AO			
4.0								
	SM	5.0-7.0 SILTY SAND: moderate brown (5YR 4/4); 90% fine sand; 10% non plastic silt; loose; moist.	J-1	1 2 3	SS			1.5/1.5 100% recovery
6.0								
					AO			
	SC/CL	7.0'-9.5' CLAYEY SAND/SANDY CLAY: brownish black, (5YR 2/1). 50% fine sand, 50% moderately plastic clay; soft/loose; moist.						
8.0								
				2	DR			1.0/1.0 100% recovery
10.0	SM	9.5-10.5 SILTY SAND: greyish green (10GY 5/2); 85% fine sand; 15% low plastic fines; loose; moist.	C-1	2	RO			
	CL	10.5'-14.5 SANDY CLAY: greyish black (N2); 90% plastic fines; 10% fine sand; firm; moist; contains roots.	J-2	1 2 3	SS			1.0/1.5 66% recovery
12.0					RO			
		PUENTE FORMATION						
14.0								
		14.5-23.0 SILTY CLAYSTONE: mottled dark yellowish orange (10YR 6/6) and moderate yellowish brown (10YR 5/4); 100% highly plastic fines; stiff; moist.						
16.0			J-3	3 5 5	SS			1.0/1.5 66% recovery
		Physical Condition: moderately fractured; soft to friable hardness plastic to friable strength moderately weathered;			RO			
18.0								
		Fe staining in fractures	C-2	5 7	CR			1.0/1.0 100% recovery
20.0								

SHEET 1 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/10-11/83 HOLE NO. 11-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0		14.5-23.0 SILTY CLAYSTONE: cont.			RO			
22.0			J-4	2 5 6	SS			1.5/1.5 100% recovery
		23.0-75.0' CLAYEY SILTSTONE: greenish black (5G2/1); massive bedding; contains mica			RO			color change
24.0								
26.0		Physical Condition: little to massive fracturing; friable strength and hardness; fresh.	J-5	12 16 38	SS			1.5/1.5 100% recovery
28.0					RO			down 1:25-1:40 linkage jumped gear
			C-3	29 58	DR			1.0/1.0 100% recovery
30.0					RO			
32.0		31.0-54.0 interbedded sand lenses, very fine grained <1/8" thick	J-6	16 17 34	SS			1.5/1.5 100% recovery
34.0					RO			
36.0	55°		J-7	16 23 41	SS			1.5/1.5 100% recovery
38.0					RO			
40.0			PB-1		PB			2.5/2.5 100% recovery
42.0	50°	42 sand lenses thicken to 1/4"	J-8	16 17 37	SS			1.5/1.5 100% recovery
44.0		↓			RO			

SHEET 2 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/10-11/83 HOLE NO. 11-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		23.0-75.0 CLAYEY SILTSTONE: cont.			RO			
46.0		46.6-47' volcanic ash layer; light greenish white (5GY 9/1)	J-9	24 50-5"	SS			0.9/0.9 100% recovery
					RO			
48.0								minor rig chatter
			C-4	14 35	DR			1.0/1.0 100% recovery
50.0		49.8 2" chert lense highly fractured; little weathered			RO			
			J-10	20 33 50-5"	SS			1.5/1.5 100% recovery
52.0		52' interbeds of sand and clay increase to 1/4" to 1/2" thick			RO			
54.0								
56.0			J-11	12 24 37	SS			1.5/1.5 100% recovery
58.0					RO			
			PB-2		PB			2.5/2.5 100% recovery
60.0								
62.0			J-12	18 30 50-5.5"	SS			1.5/1.5 100% recovery
					RO			
64.0		64.5' bedding becomes massive						
66.0			J-13	16 23 34	SS			1.5/1.5 100% recovery
68.0					RO			

PROJECT 83-1101-81 DATE DRILLED 2/10-11/83 HOLE NO. 11-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		23.0-75.0 CLAYEY SILTSTONE cont. massive			RO			
			C-5	21 43	DR			1.0/1.0
70.0					RO			
72.0								
74.0			J-14	20 26 30	SS			1.5/1.5
76.0		B.H. 75' Terminated hole, grouted to surface.						H_2O @ 6' 6:45am 2/11/83; Converse sampler driven with 300 lb downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 11-3

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/10-11/83.

OVERBURDEN DEPTH (FT.) 0 TO 14.5'.

BEDROCK DEPTH (FT.) 14.5 TO 75' (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 6' during drilling DATE 2/11/83; ____ DATE ____.

GAS NO; DEPTH FIRST NOTICED ____, DATE ____.

E-LOG NO.

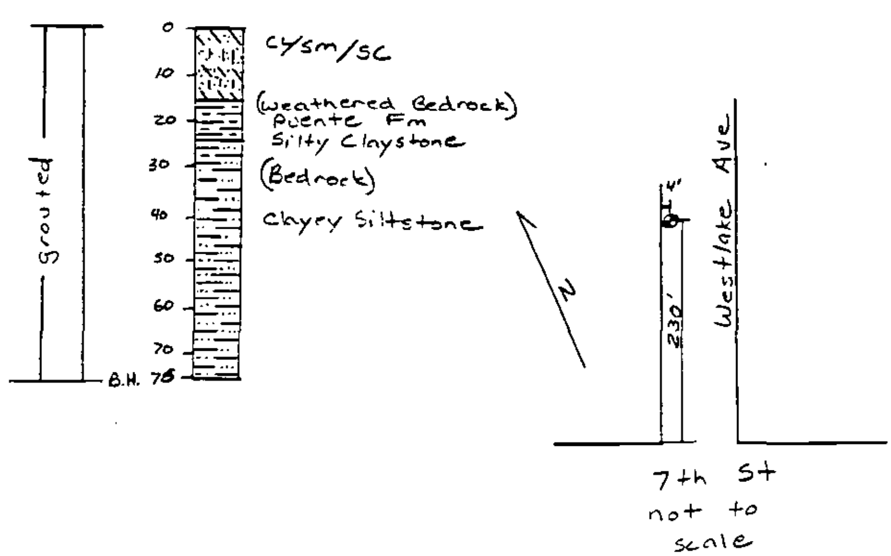
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTO SITE PLAN AL-A-1 (1"=40') Weese & Assoc. 2/28/83

SKETCH



Converse Consultants

Boring Log 11-4

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/20/83 HOLE NO. 11-4
LOCATION N-S Alley between Westlake & Alvarado GROUND ELEV. 270.5'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 4⅞" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete alleyway TOTAL DEPTH 75' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION OLD ALLUVIUM	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0		0.0-0.2 Concrete			GB			move & set up 8:00 - 9:00 set 5' of 5" steel casing
	CL	0.2-4.5 SILTY CLAY: dark yellowish brown (10YR 4/2); 98% moderately plastic fines 2% fine sand; soft to firm; moist.						
2.0								
4.0								
	CL	4.5'-22.4' CLAY: olive black (5Y2/1). 100% hi plasticity fines; firm; moist; high dry strength; no dilatency;	J-1	2 2 4	SS			1.5/1.5 100% recovery
6.0					RD			
8.0		8.0- color change to grayish brown (5YR 3/2); contains 4% fine sand, subangular	C-1	5 6	OR			1.0/1.0 100% recovery
10.0					RD			
12.0			J-2	4 7 10	SS			1.5/1.5 100% recovery
		grades to SILTY CLAY with 5% fine sand			RD			
14.0								
16.0			J-3	3 5 6	SS			1.5/1.5 100% recovery
18.0					RD			
20.0			C-2	5 7	CR			1.0/1.0 100% recovery

SHEET 1 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/20/83 HOLE NO. 11-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	CL	4.5-22.4 SILTY CLAY: cont.			RD			
			J-4	2 4 5	SS			1.5/1.5 100% recovery
22.0		PUENTE FORMATION						
		22.4-28.5 SILTY CLAYSTONE: greenish grey (5GY 6/1); mottled with moderate brown (10YR 5/4); massive bedding; Physical Condition: little fractured to massive; soft to friable hardness; plastic to friable strength; moderately weathered.			RD			
24.0								
26.0			J-5	1/2 1/2 3	SS			1.5/1.5 100% recovery
28.0					RD			
		28.5-75 SILTY CLAYSTONE: olive black (5Y 2/1), 100% moderately plastic fines. stiff to Hard; moist, massive to thinly bedded; strong sulfur odor 30.8-32.3 cemented zone $CaCO_3$ in fractures Physical Condition: 30.8-32.3 intensely to closely fractured, min. 1/8", max. 2½"; hard; strong; moderately weathered.	PB 1		PB			2.0/2.0 100% recovery refusal
30.0								
			J-6	50-3"	SS			0.2/0.2 100% recovery refusal rig chatter
					RO			
32.0								
34.0		Physical Condition: little fractured to massive; soft to friable hardness; plastic to friable strength; fresh						
36.0			J-7	9 20 26	SS			1.5/1.5 100% recovery
					RO			
38.0								
			C-3	18 48	DR			1.0/1.0 100% recovery
40.0					RO			
			J-8	9 18 34	SS			1.5/1.5 100% recovery
42.0								
44.0								

PROJECT 83-1101-81 DATE DRILLED 2/20/83 HOLE NO. 11-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		28.5 - 75' SILTY CLAYSTONE: cont.			RO			
46.0	55°	occasional very fine silty sand lenses ½" thick no sulfur odor	J-9	10 21 32	SS			1.5/1.5 100% recovery
48.0					RO			
50.0			PB-2		PB			2.5/2.5 100% recovery
52.0		increase in sand lense frequency; contains shells	J-10	11 23 34	SS			1.5/1.5 100% recovery
54.0					RO			
56.0		decrease in sand lense frequency and thickness minor ~~[illegible]~~	J-11	11 20 31	SS			1.5/1.5 100% recovery
58.0					RO			
60.0			C-4	17 39	DR			1.0/1.0 100% recovery
		occasional rounded gravels			RO			
62.0		massive bedding sand and discoloration on fracture surfaces fiborous material in fracture @ 61.2'	J-12	9 19 28	SS			1.5/1.5 100% recovery
64.0					RO			
66.0								
68.0		contains irregular sand blebs vaguely following bedding planes	J-13	6 18	SS			1.5/1.5 100% recovery

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/20/83 HOLE NO. -11-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		28.5 - 75' SILTY CLAYSTONE: cont.	J-13	31	SS			
					RD			
70.0								
					PB			2.5/2.5
72.0			PB-3					100% recovery
		increase in 1" very fine silty sand lenses						
74.0			J-14	7	SS			1.5/1.5 ?
				15				100% recovery
				33				
		B.H. 75.0' Terminated hole. Grouted to ground surface						finish 1:30 Converse sampler driven with 300 lb. downhole hammer, @ 18" drop. Pitcher samples cut with 200 p.s.i.
76.0								
78.0								

SHEET 4 OF 5

SUMMARY BORING NO. 11-4

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/20/83.

OVERBURDEN DEPTH (FT.) 0 TO 22.4.

BEDROCK DEPTH (FT.) 22.4 TO 75 (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) ____ DATE ____; ____ DATE ____.

GAS ~~SULFUR~~ GASOLINE; DEPTH FIRST NOTICED 30, DATE 2/20/83.

E-LOG NO.

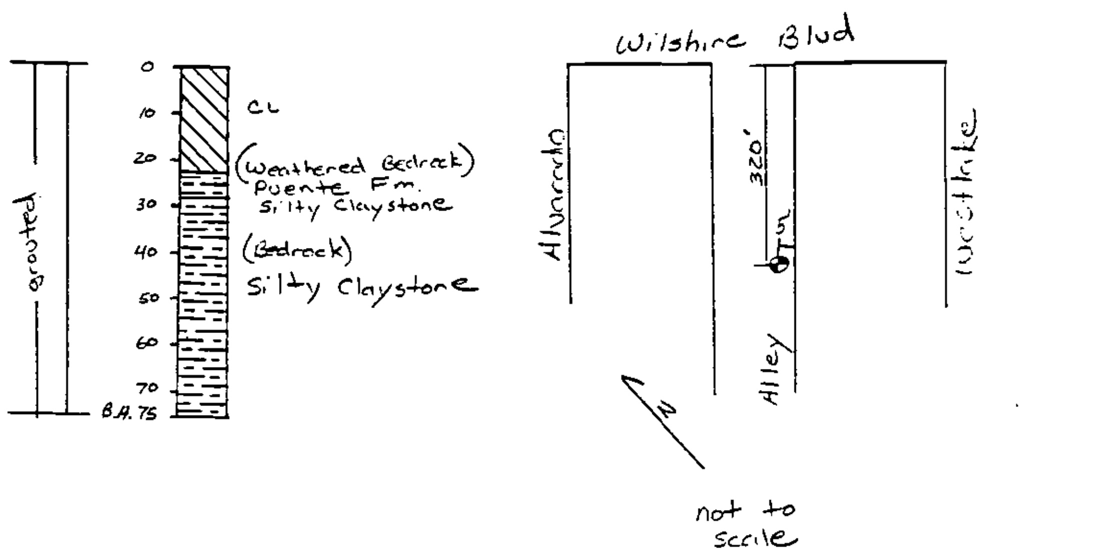
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTO SITE PLAN AL-A-1 (1"=40') Weese & Assoc 2/28/83.

SKETCH



SHEET 5 OF 5

Converse Consultants

Boring Log 11-5

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME.
CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/11/83 HOLE NO. 11-5
LOCATION Bonnie Brae N. of 7th St. GROUND ELEV. 286.2'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete gutter TOTAL DEPTH 75' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC	0.0-0.7 Concrete OLD ALLUVIUM			GE			8:30am move and set up 10:00am begin drilling
	SM	0.7-6.2 SILTY SAND: dusky yellow (5Y6/4); 90% subangular fine sand, 10% non plastic silt. dense, moist, occasional coarse sand grains; grading finer with depth						Set 6'of 5" steel casing
2.0								
4.0		color grading to yellowish grey (5Y7/2)						
			J-1	20 28 32	SS			1.5/1.5 100% recovery
6.0								
	CL	6.2'-20.5' SILTY CLAY: mottled light olive grey (5Y5/2) and moderate brown (5YR 3/4); very stiff, moist, occasional cemented zones.			RD			
8.0								
			C-1	5 10	DR			1.0/1.0 100% recovery
10.0					RD			
			J-2	5 6 5	SS			1.5/1.5 100% recovery
12.0					RD			
14.0								
16.0			J-3	5 7 9	SS			1.5/1.5 100% recovery
18.0					RD			
20.0			C-2	7 9	DR			1.0/1.0 100% recovery pocket pen. 3.25

PROJECT 83-1101-81 DATE DRILLED 2/11/83 HOLE NO. 11-5

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	CL	PUENTE FORMATION			RD			
		20.5 - 75.0' SILTY CLAYSTONE: dark greenish grey (5GY4/1) 100% highly plastic fines. contains mica; moist; massive bedding	J-4	6 10 15	SS			1.5/1.5 100% recovery
22.0					RD			
24.0		Physical Condition: little fractured to massive friable hardness and strength fresh.						
26.0			J-5	7 17 26	SS			1.5/1.5 100% recovery
28.0					RD			
30.0			PB 1		PB			1.5/2.0 pulled out of bottom 75% recovery
32.0			J-6	11 18 34	SS			1.5/1.5 100% recovery
34.0					RD			
36.0			J-7	13 16 27	SS			1.5/1.5 100% recovery
38.0					RD			
40.0			C-3	17 35	CR			[illegible] 100% recovery
					RD			
42.0			J-8	12 17 24	SS			[illegible] 100% recovery
44.0					RD			

SHEET 2 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/11/83 HOLE NO. 11-5

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		20.5'-75.0' SILTY CLAYSTONE cont. same massive bedding			RC			
46.0			J-9	11 19 31	SS			1.5/1.5 100% recovery
48.0					RO			
			PB-2		PB			2.5/2.5 100% recovery
50.0								
52.0			J-10	6 16 26	SS			1.5/1.5 100% recovery
					RO			
54.0								
56.0			J-11	12 17 30	SS			1.5/1.5 100% recovery
58.0					RO			
			C-4	20 46	DR			1.0/1.0 100% recovery
60.0					RO			
			J-12	13 16 26	SS			1.5/1.5 100% recovery
62.0					RO			
64.0								
66.0			J-13	13 16 25	SS			1.5/1.5 100% recovery
68.0								

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/11/83 HOLE NO. 11-5

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		20.5'-75.0' SILTY CLAYSTONE: cont.						
		same massive bedding						
70.0								
72.0			PB 3		PB			2.5/2.5 100% recovery
74.0			J-14	9 17 24	SS			1.5/1.5 100% recovery
76.0		B.H. 75.0' Terminated hole; installed 75' of 2" ABS; 35-75' slotted 3½'-2½' bentonite seal. surface cap placed						finished drilling @ 5:00 pm; Converse sampler driven with 300 lb. downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 11-5

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/11/83.

OVERBURDEN DEPTH (FT.) 0 TO 20.5.

BEDROCK DEPTH (FT.) 20.5 TO 75 (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 8.8' DATE 4/4/83; ____ DATE ____.

GAS NO; DEPTH FIRST NOTICED ____, DATE ____.

E-LOG NO.

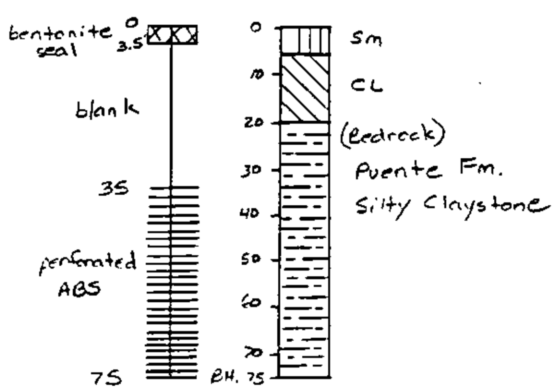
DOWN-HOLE SURVEY NO.

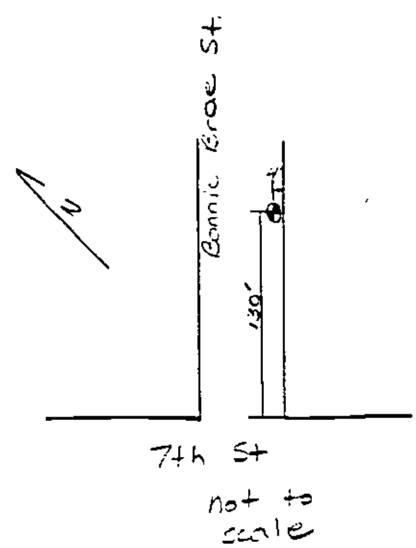
CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" 0 TO 75.

GROUND ELEVATION REF. RTD SITE PLAN AL-A-1 (1"=40') Weese & Assoc. 2/28/83

SKETCH





SHEET 5 OF 5

Converse Consultants

Boring Log 11-6

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME.
CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/12/83 HOLE NO. 11-6
LOCATION N-S Alley between Bonnie Brae & Westlake GROUND ELEV. 277.5'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete alleyway TOTAL DEPTH 75' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC	0.0-0.5 Concrete			AO			move and set up 6:30-7:45 Set 5" steel casing to 11'
	CL	OLD ALLUVIUM 0.5'-11.0' SILTY CLAY: mottled moderate brown (5YR 4/4) and light olive grey (5Y 6/2); 100% moderately plastic fines; stiff; moist.						
2.0								
4.0								
			J-1	5 7 10	SS			1.5/1.5 100% recovery
6.0					RO			
8.0								
			C-1	5 8	DR			1.0/1.0 100% recovery
10.0					RO			
		11.0'-75.0' CLAYEY SILTSTONE: olive black (5GY 2/1); massive bedding;	J-2	12 19 28	SS			1.5/1.5 100% recovery
12.0					RO			
		Physical Condition: little fractured to massive; soft to friable hardness; plastic to friable strength; fresh						
14.0								
16.0								
		strong sulfur odor	J-3	15 35 50	SS			1.5/1.5 100% recovery
18.0					RO			
			C-2	19 41	CR			1.0/1.0 100% recovery
20.0								

PROJECT 83-1101-81 DATE DRILLED 2/12/83 HOLE NO. 11-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0		11.0-75.0' CLAYEY SILTSTONE cont.			RD			
		19.5-20.5 interbedded with medium greenish grey (5GY5/1) very fine sand.	J-4	13	SS			1.5/1.5 100% recovery
22.0				27				
				48				
		20.5 return to massive bedding			RD			
24.0								
26.0	45°		J-5	13	SS			1.5/1.5 100% recovery
				31				
				42				
28.0					RD			
			PB-1		PB			2.5/2.5 100% recovery
30.0								
		sample J-6 : irregular very fine sand blebs	J-6	17	SS			1.5/1.5 100% recovery
32.0				29				
				40				
					RD			
34.0								
36.0			J-7	11	SS			1.5/1.5 100% recovery
				25				
				40				
38.0		38-38.5 gravelly lense			RD			rig chatter
			C-3	23	DR			1.0/1.0 100% recovery
40.0				50				
					RD			
			J-8	15	SS			1.5/1.5 100% recovery
42.0				23				
				34				
					RD			
44.0								

SHEET 2 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/12/83 HOLE NO. -11-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		11.0 - 75.0 CLAYEY SILTSTONE: cont.			RO			
46.0			J-9	15 / 20 / 31	SS			1.5/1.5 100% recovery
48.0					RO			
50.0			PB-2		PB			2.5/2.5 100% recovery
52.0			J-10	10 / 20 / 30	SS			1.5/1.5 100% recovery
54.0					RO			
56.0			J-11	17 / 21 / 33	SS			1.5/1.5 100% recovery
58.0					RO			
60.0			C-4	26 / 47	DR			1.0/1.0 100% recovery
					RO			
62.0		sample J-12: contains irregular very fine sand blebs	J-12	10 / 21 / 34	SS			1.5/1.5 100% recovery
64.0					RO			
66.0			J-13	13 / 25 / 37	SS			1.5/1.5 100% recovery
68.0					RO			

PROJECT 83-1101-81 DATE DRILLED 2/12/83 HOLE NO. 11-6

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		11.0-75.0' CLAYEY SILTSTONE: cont.			RD			
70.0								
72.0		continued massive	PB-3		PB			2.5/2.5
74.0			J-14	13 24 46	SS			1.5/1.5
76.0		B.H. 75.0' Terminated hole backfilled with grout tremied in to surface						finish drilling noon; Converse sampler driven with 300 lb. downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.
78.0								

SHEET 4 OF 5

SUMMARY BORING NO. 11-6

PROJECT 83-1101-81 STATION HOLE yes DATE DRILLED 2/12/83.

OVERBURDEN DEPTH (FT.) 0 TO 11.

BEDROCK DEPTH (FT.) 11' TO 75' (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) ____ DATE ____; ____ DATE ____.

GAS ~~SULFUR~~ PETROLEUM; DEPTH FIRST NOTICED 11', DATE 2/12/83.

E-LOG NO.

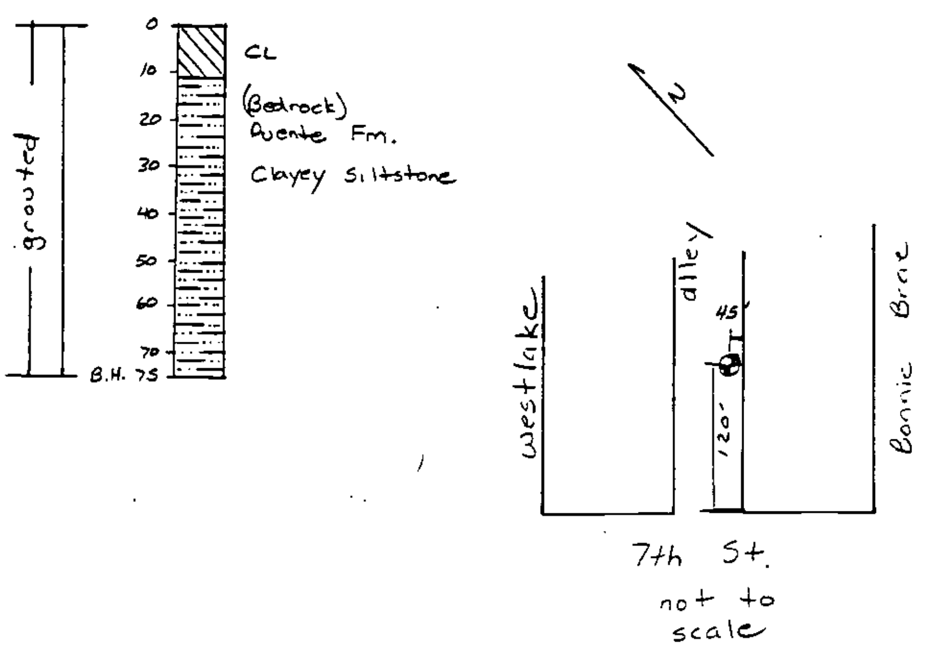
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTO SITE PLAN AL-A-1 (1"=40') Weese & Assoc. 2/28/83

SKETCH



SHEET 5 OF 5

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

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

BORING LOG 11-7

Proj: DESIGN UNIT A 170 **Date Drilled** 1/13-14/84 **Ground Elev.** 252.3

Drill Rig FAILING 250 **Logged By** D. Gillette **Total Depth** 62.5'

Hole Diameter 4" **Hammer Weight & Fall** 300 lb. @ 15"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AF	0.0-5.0 ARTIFICIAL FILL			A	
4.5		4.5-5.0 asphalt				
5	AC	5.0-10.0 SILTY CLAY: dusky brown; plastic; trace fine sand; firm to stiff; very moist; organic odor	PB1		PB	4.5 AC possible old lake liner
8			C-1	8	DR	
					RD	
10	SC	10.0-15.0 SANDY CLAY: greyish orange; slightly plastic, very moist	PB2		PB	
13			C-2	15	DR	
					RD	
15	TP	15.0-62.5 BEDROCK-PUENTE FORMATION CLAYEY SILTSTONE interbedded with SILTY SANDSTONE: light brown and medium grey; moist moderately plastic bedding planes 1/8 to 1/2" dips about 50°	PB3		PB	
18.5			C-3	18	DR	
20					RD	

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

Sheet 1 of 3

Project DESIGN UNIT A170 Date Drilled 1/13-14/84 Hole No. 11-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	TP	15.0-62.5 <u>BEDROCK-PUENTE FORMATION</u>: (cont.)			RD	
		21.0 olive grey and dusky yellowish brown				
22			PB 4		PB	1.9/2.5 recovery
		Physical Condition: little fractured to massive plastic to friable strength; fresh to little weathered				
24			C-4	78	DR	
					PB	
26						
		poorly consolidated silty claystone with interbeds of laminated silty sandstone and siltstone - laminated layers appear "varved" with slight organic odor	PB5		PB	2.0/2.5 recovery
28						
30			C-5	103/ 11"	DR	
					RD	
32						
			PB6		PB	2.0/2.5 recovery
34						
			C-6	78	DR	
36					RD	
38			PB7		PB	2.3/2.5 recovery
40						
		41.0 laminated sandstone & silty claystone	C-7	87	DR	
42					RD	
44			PB		PB	

Project DESIGN UNIT A170 Date Drilled 1/13-14/84 Hole No. 11-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	TP	15.0-62.5 BEDROCK-PUENTE FORMATION: (cont.)	PB8		PB	2.3/2.5 recovery
46			C-8	83	DR	1-13-84
					RD	1-14-84
48						
50	50°		PB9		PB	2.1/2.5 recovery
52		51.0 silty sandstone and clay-stone with well defined bedding planes	C-9	100K / 11½"	DR	
					RD	
54						
56			PB10		PB	2.1/2.5 recovery
58			C-10	100 / 10½"	DR	
					RD	
60			PB11		PB	2.0/2.5 recovery
62						
	B.H.	62.5' Terminated Hole				
64						
66						
68						

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

BORING LOG 11-8

Proj: DESIGN UNIT A 170 **Date Drilled** 1-14-83 **Ground Elev.** 252.5'

Drill Rig FAILING 250 **Logged By** D. Gillette **Total Depth** 55.5'

Hole Diameter 4" **Hammer Weight & Fall** 140 lbs @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AF	0.0-0.4 ARTIFICIAL FILL SILT AND SILTY CLAY: dark yellowish orange			A	
2			C-1	4	DR	
					RD	
4	CL	4.0-10.0 CLAY: greyish brown; contains trace of sand				
			PB1		PB	2.5/2.5 recovery
6						
8			C-7	8	DR	
					RD	
10	SC	1.0-18.5 SANDY CLAY: greyish black				
			PB2		PB	2.5/2.5 recovery
12						
		greyish orange; cemented sand nodules	C-3	15	DR	
14					RD	
16			PB3		PB	2.5/2.5 recovery
18						
	SP	18.5-21.5 SAND AND GRAVEL: dark greenish grey	No Rec.	100	DR	
20					RD	

Sheet 1 of 3

Project DESIGN UNIT A170 Date Drilled 1-14-84 Hole No. 11-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SP	18.5-21.5 SAND AND GRAVEL: (cont.)				
		21.5-55.5 BEDROCK-PUENTE FORMATION CLAYEY SILTSTONE INTERBEDDED WITH SILTY SANDSTONE: olive grey and dusky yellowish brown				
22			PB4		PB	1.5/2.5 recovery
24			C-4	100/9"	DR	
					RD	
26						
28			PB5		PB	2.0/2.5 recovery
30	50°		C-5	100K/11"	DR	
					RD	
32						
			PB6		PB	1.5/2.0 recovery
34						
					RD	
36						
			PB7		PB	2.3/2.5 recovery
38						
			C-6	100/9"	DR	
40					RD	
42			PB8		PB	2.3/2.5 recovery
44						

Project DESIGN UNIT A 170 **Date Drilled** 1-14-84 **Hole No.** 11-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	TP	21.5-55.0 PUENTE FORMATION INTERBEDDED CLAYEY SILTSTONE AND SILTY SANDSTONE	PB8		PB	
			C-7	100K 9"	DR	
46					RD	
48			PB9		PB	2.5/2.5 recovery
50			C-8	100k 10"	DR	
					RD	
52	50°					
			PB10		PB	2.5/2.5 recovery
54						
	B.H	55.5' Terminated Hole				
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

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

BORING LOG 11-9

Proj: DESIGN UNIT A170 **Date Drilled** 1-16-84 **Ground Elev.** 246.2'

Drill Rig FAILING 250 **Logged By** Dan Gillette **Total Depth** 54.0'

Hole Diameter 4" **Hammer Weight & Fall** 300 lbs @ 15-20"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CH/CH	0.0-11.0 CLAY: brownish black; wet; slightly organic				
2			C-1	8		
					RD	
4	CH	4.0 dusky brown; very moist; firm	PB1		PB	2.5/2.5 recovery
6			C-2	12	DR	
8					RD	
10		11.0-54.0 BEDROCK-PUENTE FORMATION CLAYEY SANDSTONE INTERBEDDED WITH CLAYEY SILTSTONE: dark yellowish brown; moist to very moist	PB2		PB	2.5/2.5 recovery
12	50°		C-3	22	DR	
14					RD	
16		Physical Condition: little fracturing to massive; soft to friable hardness; plastic to friable strength; fresh to little weathered	PB3		PB	2.5/2.5 recovery
18			C-4	89	DR	
20		29.0 greyish olive; moist			RD	

Sheet 1 of 3

Project DESIGN UNIT A170 Date Drilled 1-16-84 Hole No. 11-9

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	TP	11.0-54.0 PUENTE FORMATION (cont.) CLAYEY SANDSTONE INTERBEDDED WITH CLAYEY SILTSTONE			RD	
			PB4		PB	1.2/2.5 recovery
22						
			C-5	78	DR	
24	50°	interbedded clayey sandstone and clayey siltstone; beds ¼ to ½" well defined; moist			RD	
26			PB5		PB	2.5/2.5 recovery
28			C-6	100k / 9"	DR	
30					RD	
32			PB6		PB	2.5/2.5 recovery
34			C-7	100 / 9"	RD	
36					RD	
38		38.0-46.0 silty sandstone; moderately hard - moist	PB 7		PB	2.5/2.5 recovery
	50°					
40			C-8	89	DR	
					RD	
42			PB8		PB	2.5/2.5 recovery
44						

Project DESIGN UNIT A170 Date Drilled 1-16-84 Hole No. 11-9

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	TP	11.0-54.0 PUENTE FORMATION SILTY SANDSTONE INTERBEDDED WITH CLAYEY SILTSTONE	PB8		PB	
	50°		C-9	100/9"	DR	
46						
48			PB 9		PB	2.5/2.5 recovery
50			PB10		PB	2.5/2.5 recovery
52						
			C-10	100/11"		
54	B.H.	54.0' Terminated Hole				
56						
58						
60						
62						
64						
66						
68						

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 11-10

Proj: DESIGN UNIT A170 **Date Drilled** 1-17-84 **Ground Elev.** 248.1

Drill Rig FAILING 250 **Logged By** D. Gillette **Total Depth** 60.0'

Hole Diameter 4" **Hammer Weight & Fall** 300 lbs @ 15-20"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	CH/OH	0.0-10.0 CLAY: Grayish brown; wet soft			A	
2			C-1	2	DR	
					RD	
4			PB1		PB	2.5/2.5 recovery
6			C-2	5	DR	
	CH	7.0 light brown; firm; very moist				
8					RD	
10	TP	10.0-60.0 BEDROCK-PUENTE FORMATION CLAYEY SILTSTONE interbedded with SILTY SANDSTONE: mottled greenish gray and moderate brown; moist; moderately hard: slightly weathered	PB 2		PB	2.5/2.5 recovery
	50°					
12			C-3	18	DR	
					RD	
14						
16			PB3		PB	2.5/2.5 recovery
		Physical Condition: little fractured to massive soft to friable hardness; plastic to friable strength				
18			C-4	30	DR	
					RD	
20						

Project DESIGN UNIT A170 Date Drilled 1-17-84 Hole No. 11-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	TP	10.0-60.0 PUENTE FORMATION CLAYEY SILTSTONE interbedded with SILTY SANDSTONE: (cont.)			RD	
			PB4		PB	2.5/2.5 recovery
22						
	50°		C-5	95	DR	
24					RD	
		laminated beds of SANDSTONE SILTY SANDSTONE and CLAYEY SILTSTONE: beds ¼ to½", siltstone beds; moderately brown; slightly organic				
26			PB5		PB	2.5/2.5 recovery
28						
			C-6	108 10"	DR	
30					RD	
32			PB6		PB	2.5/2.5 recovery
34			C-7	100 9"	DR	
					RD	
36						
			PB7		PB	2.5/2.5 recovery
38						
40			C-8	100 8"	DR	
					RD	
42						
			PB8		PB	2.5/2.5 recovery
44						

Project DESIGN UNIT A170 Date Drilled 1-17-84 Hole No. 11-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	TP	10.0-60.0 PUENTE FORMATION CLAYEY SILTSTONE interbedded with SANDSTONE and CLAYEY SILTSTONE	PB8		PB	
			C-9	100k 10"	DR	
46	50°				RD	
48			PB9		PB	2.5/2.5 recovery
50					RD	
52						
54						
56						
58			PB10		PB	2.5/2.5 recovery
60	B.H.	60.0' Terminated Hole				
62						
64						
66						
68						

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

BORING LOG 11-11

Proj: DESIGN UNIT A170 **Date Drilled** 1-15-84 **Ground Elev.** 251.6'

Drill Rig FAILING 250 **Logged By** D. Gillette **Total Depth** 78.0'

Hole Diameter 4" **Hammer Weight & Fall** 300 lb @ 15"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AF	0.0-4.0 ARTIFICIAL FILL: sand; gravel; wood; glass; boulders; wet; medium dense			A	
2			C-1	12k 12"	DR	
					RD	
4	ML	4.0-9.0 CLAYEY SILT: pale yellowish brown mottled; very moist	PB 1		PB	2.5/2.5 recovery
6			C-2	12	DR	
8					RD	
9	TP	9.0-78.0 BEDROCK-PUENTE FORMATION SILTY CLAYSTONE interbedded with SILTY SANDSTONE: moderate brown; moist	PB 2		PB	2.5/2.5 recovery
12		Physical Condition: little fracturing to massive soft to friable hardness; plastic to friable strength; fresh to little weathered	C-3	53	DR	
14					RD	
16	50°	16.0 olive gray and dusky yellowish brown	PB 3		PB	2.5/2.5 recovery
18			C-4	68	DR	
20					RD	

Project DESIGN UNIT A170 Date Drilled 1-15-84 Hole No. 11-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	TP	9.0-78.0 PUENTE FORMATION SILTY CLAYSTONE interbedded with SILTY SANDSTONE:moist			RD	
			PB4		PB	2.5/2.5 recovery
22						
			C-5	100k 9"	DR	
24		thinly bedded (¼-½") laminated silty claystone beds and silty sandstone (fine sand) beds; well defined bedding			RD	
26			PB5		PB	2.5/2.5 recovery
28						
			C-6	100 11"	DR	
30	50°				RD	
32			PB6		PB	2.5/2.5 recovery
34			C-7	100 11"	DR	
36					RD	
38			PB7		PB	2.5/2.5 recovery
			C-8	100 9"	DR	
40					RD	
42			PB8		PB	2.5/2.5 recovery
44						

Sheet 2 of 4

Project DESIGN UNIT A170 Date Drilled 1-15-84 Hole No. 11-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	TP	9.0-78.0 PUENTE FORMATION SILTY CLAYSTONE interbedded with SILTY SANOSTONE: (cont.) moist	PB		PB	2.5/2.5 recovery
			C-9	100 10"	DR	
46					RD	
48			PB-9		PB	2.5/2.5 recovery
50			C-10	100 9"	DR	
					RD	
52						
54	50°		PB10		PB	2.5/2.5 recovery
56			C-11	100 9"	DR	
					RD	
58						
60		60-78.0 bedding becomes much sandier (20% silty clay beds; 80% silty sand beds)	PB11		PB	2.5/2.5 recovery
62			C-12	100 10"	DR	
					RD	
64			PB12		PB	2.5/2.5 recovery
66						
68			C-13	100 11"	DR	

Project DESIGN UNIT A170 Date Drilled 1-15-84 Hole No. 11-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	TP	910-78.0 PUENTE FORMATION SILTY CLAYSTONE interbedded with SILTY SANDSTONE: (cont.) moist			RD	
70			PB13		PB	2.5/2.5 recovery
72	50°		C-14	100 10"	DR	
74					RD	
76			PB 14		PB	2.5/2.5 recovery
78		B.H. 78.0' Terminated Hole				
80						
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

A170.

Converse Consultants

MEMORANDUM

Date November 29, 1983

To File

From H.A.Spellman

Project No. 83-1140-26/71

Subject Record of MRTC Boring 12A, Virgil/Wilshire Blvd.

Man-sized auger Boring 12A, drilled today, 11/29/83, is the 1st of 5 borings drilled under the MRTC Ammendment No. 1, to MRTC Contract No. 503. These are the notes from John Stellar's log and his downhole observations. The boring is located in Design Unit A220 on the west side of Virgil, 212 feet south of Wilshire Boulevard. The purpose of the boring was to evaluate Puente bedrock and groundwater conditions at tunnel grade. The boring is at approximately tunnel line Station 303, collar Elevation 263 feet and tunnel invert grade at this location is about Elevation 190 feet; total depth of the hole is 70 feet.

Type Material

0'	to 2'	- fill, includes 1 foot of AC concrete, sandy clay, medium brown, moist, medium dense to soft
2'	to 6'	- silty sand, medium brown, moist, medium dense
6'	to 11'	- sand, light yellow, slightly moist, medium dense, clean fine sand; slight belling of hole 8 feet to 10 feet; bag sample at 10 feet
11'	to 22'	- gravelly sand, light brown to yellow, medium dense, 10% to 15% gravel to 1/2-inch diameter; less than 1 gpm seep at 21 feet
22'	to 70'	- Bedrock - Puente Formation, primarily siltstone with thin interbeds of claystone and sandstone, light brown (weathered) 22 to 34 feet, blue gray (fresh) 34 to 70 feet; slightly moist, low hardness to friable, weak, 22 to 50 feet; moderately hard, 50 to 70 feet, bedding plane attitude at 40 feet strike N20°E, dip 18° westerly; at 50 feet N24°E, dip 25° westerly; bag sample at 42 feet; no hard cemented beds here.

Bucket auger drilled the "soft" bedrock easily, which should translate into good excavation for tunneling equipment.

Caving: The hole stood very well from 0 to 70 feet; there was slight belling of 6 inches to 8 inches in the hole from 8 to 10 feet. Bedrock at tunnel grade is considered excellent tunneling material and should stand well.

Groundwater: No groundwater was encountered in the bedrock; there was a seep of less than 1 gallon per minute perched on top of the bedrock at 22 feet. Two feet of water accumulated at bottom of hole in 2 hours.

Gas and Oil: No gas was detected by the meter and no petroleum products were encountered in the boring.

Downhole Observers: John Stellar (CCI) observed the downhole conditions. Interested observers were Bud Maduke, Frank McClean (MRTC), and Buzz Spellman (CCI).

HAS:dmG

cc: B.I. Maduke & R.J. Proctor

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 12-1

Proj: DESIGN UNIT A170 Date Drilled 1-20-85 Ground Elev. ______

Drill Rig Failing 1500 Logged By M. Schluter Total Depth 121.0'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.8 A.C. PAVEMENT			C	1-20-85
	CL	0.8-1.2 SILTY CLAY, FILL				
2	SP	ALLUVIUM 1.2-5.5 SAND: moderate brown; fine to medium; trace fines, moist to very moist, medium dense,	C-1	6 11	DR 325	
4					A	
6	SM	5.5-9.0 SILTY SAND: grayish black, olive black, fine to coarse sand, with gravel to 1" and petroleum, mottled, petroleum odor, slighty sticky, includes siltstone fragments, dense				Perched groundwater at 5.5'
8			J-1	19 51	SS	SPT Refusal @ 12" 0.5/1.0 Recovery
					RD	
10		BEDROCK 9.0-121' PUENTE FORMATION INTERBEDDED SILTSTONE/CLAYSTONE: olive gray, grayish black, petroleum along fractures and bedding, petroleum odor, Physical Condition: little weathered, low hardness, weak, fractured	C-2	34 54	DR 325	Slight Oil film on drilling fluid tub
12					RD	
			PB 1		PB	
14						2.2/2.5 Recovery
					RD	
16						
		17.5-18.0' Calcareous cemented siltstone layer	J-2	39 50	SS	SPT Refusal @ 7" moderate rig chatter 0.5/0.6 Recovery
18					RD	
20						

Project DESIGN UNIT A170 Date Drilled 1-20-85 Hole No. 12-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20		9.0-121.0 INTERBEDDED SILTSTONE/ CLAYSTONE: (Cont.)				
		olive gray, dark greenish gray	C-3	45 69	DR 325	Refusal at 11"
22	60-70°	intensely to very intensely laminated bedding, color banding				
24		Physical Condition: low hardness, weak, closely fractured; trace to little petroleum, slight petroleum odor				
26						
			J-3	31 50	SS	SPT refusal @ 11" 0.7/0.9 recovery
28						
30			C-4	61 60	DR 325	Refusal at 10"
		Intensely laminated to very thinly bedded			RD	
32	65°		PB 2		PB	
34						2.3/2.5 recovery
					RD	
36						
			J-4	38 50	SS	SPT refusal @ 9" 0.6/0.8 recovery
38					RD	
40		No observable petroleum, slight petroleum odor	C-5	67 60	DR 325	refusal at 10"
42					RD	
44						

Sheet 2 of 6

Project DESIGN UNIT A170 Date Drilled 1-20/21-85 Hole No. 12-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44		9.0-121.0 INTERBEDDED SILTSTONE AND CLAYSTONE: (Cont.)			RD	
46		Slight petroleum odor				
			J-5	38 / 50	SS	SPT refusal @ 10"
48					RD	
50	50-60°	Brownish black claystone with slickened surfaces	C-6	69 / 100	DR 325	Refusal at 9" 1-20-85
					RD	
52			PB 3		PB	
54	55°					2.2/2.5 recovery
56					RD	
			J-6	50 / 50	SS	SPT refusal @ 7.5"
58					RD	
60						
		Light olive gray, olive gray and greenish gray	C-7	85 / 65	DR 325	Refusal at 8"
62		Physical Condition : intensely laminated to thinly bedded, low to moderately hard, weak to moderately strong			RD	
64		moist				
66						SPT refusal @ 9"
68			J-7	48 / 50	SS	

Project DESIGN UNIT A170 Date Drilled 1-21-85 Hole No. 12-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68		9.0-121.0 INTERBEDDED SILTSTONE/CLAYSTONE: (continued)			RD	
70			C-8	55	DR	refusal @ 9"
				50	325	
					RD	
72	50-55°		PB 4		PB	2.2/2.5 recovery
74						
76					RD	
			J-8	63	SS	SPT refusal @ 6" 0.5/0.5 recovery
78					RD	
80						
			C-9	70	DR	refusal @ 8"
				60	325	
82	55-60°	Physical Condition: intensely bedded to thinly laminated, low to moderately hard, little weathered to fresh, moderately strong			RD	
84		moist				
86						
			J-9	72	SS	SPT refusal @ 6" 0.3/0.5 recovery
88					RD	
90			C-10	85	DR	refusal @ 7"
				50	325	
92					RD	

Project DESIGN UNIT A170 Date Drilled 1-21-85 Hole No. 12-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92		9.0-121.0 INTERBEDDED SILTSTONE/CLAYSTONE: (continued)	PB 5		PB	2.4/2.5 recovery
94						
					RD	
96						
			J-10	76	SS	SPT refusal @ 6"
98					RD	
100						
			C-10	125	DR 325	refusal @ 6"
102					RD	
104						
106		olive gray, light olive gray, moist				
		Physical Condition: moderate hardness, moderate strength, fresh, little fractured to massive	J-11	50 / 50	SS	SPT refusal @ 9"
108					RD	
110			C-11	103	DR 325	refusal @ 6"
					RD	
112			PB 6		PB	2.3/2.5 recovery
114						
					RD	
116						

Project DESIGN UNIT A170 Date Drilled 1-21-85 Hole No. 12-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116		9.0-121.0 <u>INTERBEDDED SILTSTONE/CLAYSTONE</u>: (continued)			RD	
		as previously described	J-12	62	SS	
118					RD	
120			C-11	86	DR	refusal @ 8"
				50	325	
122	B.H.	121.0' Terminated hole Flushed Boring and Installed Piezometer <u>Perforated</u> 20-40', 60-80', 100-120' <u>Non Perforated</u> 0-20', 40-60', 80-100' Backfilled with pea gravel. Installed casing and cover.				1-21-85 perched groundwater at 5.5'
124						
126						
128						
130						
132						
134						
136						
138						
140						

SUMMARY BORING NO. 13-7

PROJECT 83-1140-26 STATION HOLE ________ DATE DRILLED 3-16-84.

OVERBURDEN DEPTH (FT.) 0.0 TO 15.5.

BEDROCK DEPTH (FT.) 15.5 TO 70.0 (T.D.).

WATER PRESS. TEST ____; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) ____ DATE ____; ____ DATE ____.

GAS ____; DEPTH FIRST NOTICED ____, DATE ____.

E-LOG ____.

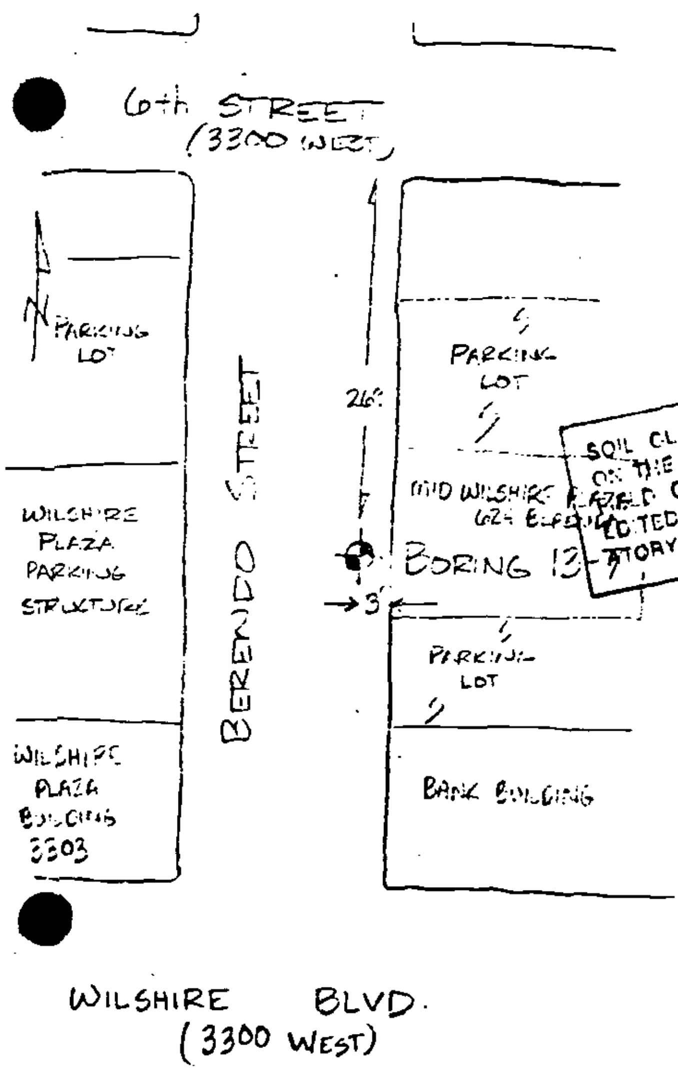
DOWN-HOLE SURVEY ________.

CROSS-HOLE SURVEY ________.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. ________________.

SKETCH



SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET ____ OF ____

Converse Consultants

Boring Log 13-7

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT SCRTD 83-1140-26 DATE DRILLED 3-16-84 HOLE NO. 13-7

LOCATION BERENDO STREET, 269' SOUTH OF 6th STREET ON BERENDO STREET GROUND ELEV. ____

DRILLING CONTRACTOR PITCHER DRILLING Co LOGGED BY MARK SCHLUTER DEPTH TO GROUND WATER ____

TYPE OF RIG FAILING 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 325#@18"/140#@30"

SURFACE CONDITIONS A.C. STREET SURFACE TOTAL DEPTH 70.0' NO. CORE BOXES ____

DEPTH 0	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0-0.8 A.C. PAVEMENT			C			STARTED DRILLING @ 0830
	ML	0.8-2.5 CLAYEY SILT OLIVE BLACK 5Y2/1 85-95% FINES 15-5% SAND F/M LOW PLASTICITY; MOIST; VERY SOFT TO SOFT			A			
2								
			C-1	6 17	DR 325			
4	CL	2.5-8.0 SANDY SILTY CLAY DARK YELLOWISH BROWN 10YR 2/2 80-90% FINES 20-10% SAND F/M LOW-MEDIUM PLASTICITY, MOIST, FIRM, SLIGHTLY POROUS, TRACE GRAVEL [illegible], TRACE LIGHT BROWN OXIDATION STREAKS [illegible], CLAY FILLED DESICCATION FRACTURES, 2-3 mm, TRACE BLACK SILTY INCLUSIONS INCREASING SAND CONTENT F/M DARK YELLOWISH BROWN 10YR 3/2.			RD			ROTARY WASH DRAG BIT
6			C-1	6 14 18	[illegible]		1.5/1.5	
8	SC/CL	8.0-12.5 CLAYEY SAND/SANDY CLAY MODERATE YELLOWISH BROWN 10YR 5/4, LIGHT OLIVE GRAY 5Y 5/2 65-75% SAND F/M [illegible] 35-25% FINES MOIST, LOOSE, MED[illegible] MOTTLED, [illegible] TRACE [illegible] DESICCATION [illegible] FRACTURES [illegible]			RD			
10								
	SC				[illegible]			
12								
	SM	12.5-15.5 SAND/GRAVELLY SAND DARK YELLOWISH BROWN 10YR 4/2 85-95% SAND F/C SUBANGULAR 15-5% GRAVEL MOIST, MEDIUM DENSE, MODERATELY WELL GRADED, GRAVEL LENSES, GRAVEL 5-35 mm, ANGULAR, [illegible], INCLUDES SILTSTONE FRAGMENTS	C-3	[illegible] 32	DR 325			TOP RING DISTURBED
14					RD			
16	Ms/Cs	15.5-70.0 PUENTE/FERNANDO FORMATIONS INTERBEDDED SILTSTONE/CLAYSTONE LIGHT OLIVE GRAY 5Y5/2 LIGHT BROWN 5YR 5/6 (WEATHERED/[illegible]) OLIVE GRAY 5Y4/1 95-100% FINES 5-0% SAND (FINE) LOW-MEDIUM SILTSTONE PLASTICITY, MEDIUM-HIGH CLAYSTONE PLASTICITY, MOIST, DAMP, THINLY TO VERY THINLY BEDDED, OCCASIONAL CEMENTED LAYERS, MICACEOUS	C-2	[illegible] 10 18	CC 140		1.2/1.5	
18					RD			
20								

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 1 OF ____

PROJECT SCRTD 83-1140-26 DATE DRILLED 3-16-84 HOLE NO. 13-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	36	PHYSICAL CONDITIONS: LITTLE FRACTURED → MASSIVE, FRACTURES WEATHERED TO A MODERATE BROWN 5YR 4/4 (OXIDIZED), VERY SOFT → SOFT, PLASTIC → FRIABLE, MODERATE WEATHERING, FIRM → VERY STIFF	C-4	18 27	DP 325			
22					RD			
24		OXIDIZED, MOTTLED LIGHT OLIVE GRAY 5Y 6/1, TO MODERATE BROWN 5YR 4/4 (RUSTIC BROWN), VERY STIFF → HARD, SOFT ROCK, OCCASIONAL GRAYISH BROWN 5YR 3/2	C-5	41 59	DP 325			
					RD			
26		THINLY → MEDIUM BEDDED, OCCASIONAL VERY THINLY BEDDED TO LAMINATED INTERBEDS	J-3	25 26 50/5	SS 140		1.5 1.5	SPT REFUSAL @ 17"
28					RD			
30	29	VERY THINLY BEDDED, OCCASIONALLY LAMINATED	C-6	38 60	DP 325			
32					RD			
34	30 24	LIGHT OLIVE GRAY 5Y 6/1, OLIVE GRAY 5Y 4/1, DUSKY YELLOWISH BROWN 10YR 2/2, LITTLE WEATHERING → FRESH, SOFT → MODERATELY HARD, MASSIVE, MODERATELY STRONG	C-7	36 50/4	DP 325			CCI REFUSAL @ 10"
36		DARK GREENISH GRAY 5GY 4/1, OLIVE BLACK 5Y 2/1, THINLY → MEDIUM BEDDED, FRESH, MODERATELY HARD, MASSIVE, MODERATELY STRONG, BREAKS ALONG BEDDING, SLIGHTLY MOIST / DAMP	J-4	12 23 50/3	SS 140		1.2 1.3	SPT REFUSAL @ 15"
38					RD			
40			C-8	61 50/2	DP 325			CCI REFUSAL @ 8"
42		— CONTINUED —			RD			
44								SHEET 2 OF ___

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

PROJECT SCRTD 53-1140-26 DATE DRILLED 3-16-84 HOLE NO. 13-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (FT)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44		PUENTE/FERNANDO FORMATIONS INTERBEDDED SILTSTONE & CLAYSTONE - CONTINUED -	C-9	53 50/3	DR 325			CCI REFUSAL @ 9"
46					RD			
48		PREDOMINANTLY DARK GREENISH GRAY 5GY 4/1 MEDIUM - THINLY BEDDED	C-10	80 50/2	DR 325			CCI REFUSAL @ 8"
50								
52					RD			
54			C-11	77 50/2	DR 325			CCI REFUSAL @ 8"
56					RD			
58	/35 /33	THINLY - VERY THINLY BEDDED PREDOMINANTLY SILTSTONE WITH OCCASIONAL CLAYSTONE, SILTSTONE GRADES TO A FINE SANDSTONE MICRO FOSSIL SHELLS → DIATOMS, BROKEN AND FRAGMENTED TO INTACT ≤1mm, WHITE	C-12	41 50/2	DR 325			CCI REFUSAL @ 9"
60								
62					RD			
64	/20		C-13	80 50/1	DR 325			CCI REFUSAL @ 7"
66								
68								

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT THE [illegible] EDITED FIELD CLASSIFICATION. THESE [illegible] FIELD CLASSIFICATIONS [illegible] EDITED OR MODIFIED TO INCLUDE [illegible] LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET 3 OF ___

PROJECT SCRTD 83-1140-26 DATE DRILLED 3-16-84 HOLE NO. 13-7

DEPTH 68	CLASS.		FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
			PUENTE/FERNANDO FORMATIONS			RC			
			SILTSTONE WITH INTERBEDDED CLAYSTONE						
			— CONTINUED —		64	DC 325			CCI REFUSAL @ 9"
70		1/32		C-14	50/3				
			END OF BORING 70.0'						FINISHED BORING @ 1455
			TREMIED A 3 SAC/65 GALLON CEMENT SLURRY INTO HOLE						
			CLEANED SITE						

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT CURRENT, UNEDITED FIELD CLASSIFICATION. THESE LOGS [illegible] EDITED OR MODIFIED TO INCLUDE THE [illegible] ATORY TESTING AND VISUAL CLASS[illegible]

SHEET 4 OF ____

SUMMARY BORING NO. 13-8

PROJECT SCRTD 83-INC-26 STATION HOLE ______ DATE DRILLED 3-17-84.

OVERBURDEN DEPTH (FT.) 0.0 TO 13.0.

BEDROCK DEPTH (FT.) 13.0 TO 80.0 (T.D.).

WATER PRESS. TEST ______; INTERVAL(S) ______ TO ______, ______ TO ______.

GROUND WATER DEPTH (FT.) ______ DATE ______; ______ DATE ______.

GAS ______; DEPTH FIRST NOTICED ______, DATE ______.

E - LOG ______.

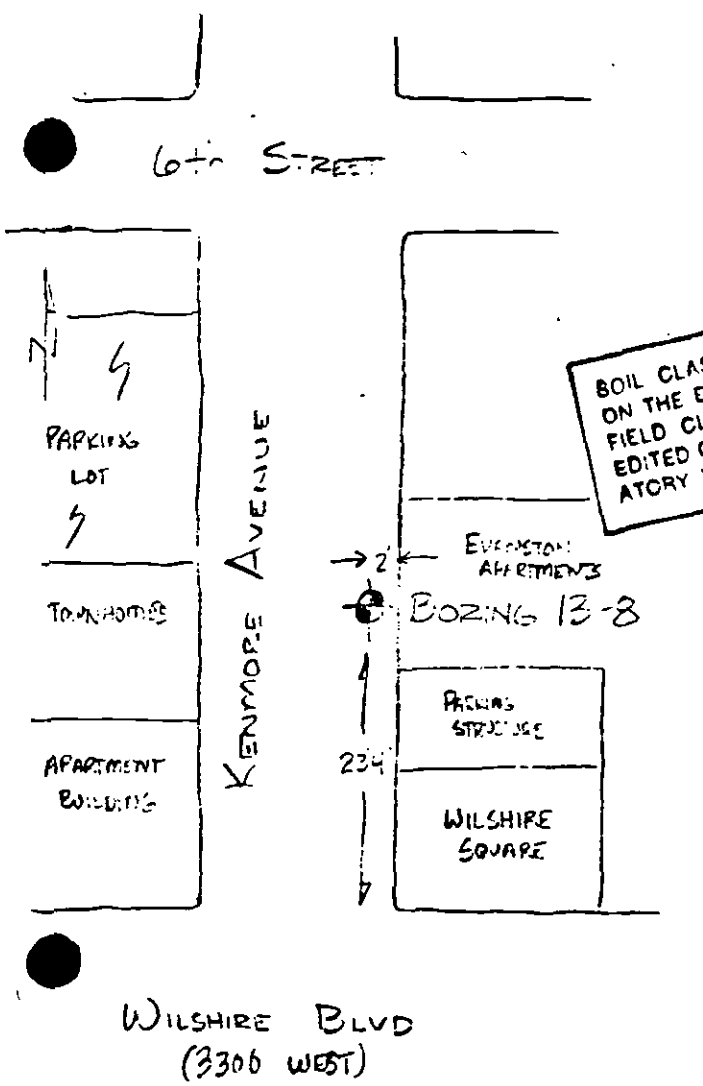
DOWN-HOLE SURVEY ______.

CROSS-HOLE SURVEY ______.

PVC CASING (I.D.): 4" ______ TO ______; 3" ______ TO ______; 2" ______ TO ______.

GROUND ELEVATION REF. ______.

SKETCH



SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT OUR INITIAL UNEDITED FIELD CLASSIFICATION. THESE LOGS HAVE NOT BEEN EDITED OR MODIFIED TO INCLUDE THE RESULTS OF LABORATORY TESTING AND VISUAL CLASSIFICATION.

SHEET ____ OF ____

Converse Consultants

Boring Log 13-8

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT [illegible] 83-1140-2[illegible] DATE DRILLED 3-17-84 HOLE NO. 13-8
LOCATION Kenmore Avenue 23[illegible] North of Wilshire Blvd. [illegible] Kenmore Ave GROUND ELEV. ____
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Mark Schluter DEPTH TO GROUND WATER ____
TYPE OF RIG Failing 1500 HOLE DIAMETER 4⅞" HAMMER WEIGHT AND FALL 325# @ 18"/140# @ 30"
SURFACE CONDITIONS A.C. Street Surface TOTAL DEPTH 80.0' NO. CORE BOXES ____

DEPTH ft	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0-0.4 A.C. Pavement			C			Started drilling @ 0800
	SM	0.4-3.0 SILTY SAND; Yellowish gray 5Y7/2, light olive gray 5Y5/2, 80-90% sand F/C, 20-10% fines, moist, loose, [illegible] well graded, trace [illegible], oxidized streaks (light [illegible] 5YR5/4)			A			
4		3.0-12.0 SILTY SAND/SAND; [illegible] 5Y7/2, [illegible] 5Y6/4, 60-80% sand (fine), 40-20% fines (silt), predominantly fine sand, [illegible], trace [illegible], oxidized streaks, light [illegible], cohesionless 5Y5/6 (San Pedro Formation?)	C-1	20	RC			Rotary wash drag bit
6			C-1	12 / 22 / 40	SS 140		1.0 / 1.5	
8					RC			
10		Trace [illegible], [illegible] oxidized streaks [illegible]	C-2	16 / 17				
12								
14	/21	13.0- PUENTE/FERNANDO FORMATION; interbedded siltstone & claystone; light olive gray 5Y6/1, 5Y4/1, 85-100% fines, 15-0% sand (fine), siltstone - low to medium plasticity, claystone - medium-high plasticity, moist/damp, thinly-[illegible] bedded, [illegible], [illegible] oxidized streaks, [illegible], very stiff to hard	C-3	16 / 21	CF 140			
16			C-3					
18		PHYSICAL CONDITION: Little fractured to massive, soft rock, friable to low hardness, weak to moderately strong, moderate weathering, breaks along bedding			RC			
20	/24		C-4	11 / 70	CF 325			

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS ON THE BORING LOGS REPRESENT OUR FIELD CLASSIFICATION. THESE LOGS [illegible] EDITED OR MODIFIED TO INCLUDE THE [illegible] ATORY TESTING AND VISUAL CLASS[illegible]

SHEET 1 OF ____

PROJECT SCRTD 83-1140-26 DATE DRILLED 3-17-84 HOLE NO. 13-8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT	DRILL MODE	[illegible]	CORE REC. %	REMARKS
20		PUENTE/FERNANDO FORMATION DARK GREENISH GRAY 5GY 4/1 OLIVE GRAY 5Y 4/1 VERY THINLY → THINLY BEDDED			RD			
22								
		DUSKY YELLOWISH BROWN 10YR 2/2 OLIVE GRAY 5Y 4/1 MODERATE → LITTLE WEATHERING MODERATELY HARD, MASSIVE, MOD STRONG	C-5	70 50/5	DK 325			
24					RD			
26			J-3	15 51	SS 140		1.0 1.0	SPT REFUSAL @ 12"
28					RD			
30	/35	OLIVE GRAY 5Y 3/2 LIGHT OLIVE GRAY 5Y 5/2 THINLY - [illegible] BEDDING	C-6	35 50/5	[illegible]			CCI REFUSAL @ 11"
32					RD			
34	/32	DARK GREENISH GRAY 5GY 4/1 FRESH, MASSIVE, MODERATELY HARD, MODERATELY STRONG	C-7	52 50/3	DK 325			CCI REFUSAL @ 9" CASING SINKING, FLUID EROSION REDUCED SKIN FRICTION, INSTALLED ADDITIONAL CASING TO 7'
36					RD			
38								
40	/32	— CONTINUED —	C-8	59 50/4	DK 325			CCI REFUSAL @ 10"
42					RD			
44								SHEET 2 OF ___

SOIL CLASSIFICATION AND FIELD DESCRIPTIONS SHOWN ON THE BORING LOGS REPRESENT [illegible] FIELD CLASSIFICATION. THESE [illegible] EDITED OR MODIFIED TO INCLUDE [illegible] ATORY TESTING AND VISUAL CLASS[illegible]

PROJECT SCRTD 83-1100-26 DATE DRILLED 2-17-84 HOLE NO. 13-8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44	/31	PUENTE/FERNANDO FORMATION/ INTERBEDDED SILTSTONE & CLAYSTONE. — CONTINUED —	C-9	56 50/4	DR 325			CCI REFUSAL @ 10" ADDED ADDITIONAL 8' OF CASING - 15' TOTAL
46		DARK GREENISH GRAY 5GY4/1 WITH: MEDIUM DARK GRAY N4 MODERATE BROWN 5YR 3/4 (OXIDIZED) THINLY - MEDIUM BEDDED, FRESH, MASSIVE, MODERATELY HARD, MODERATELY STRONG.			RD			
48		SILTSTONE WITH INTERBEDDED SANDSTONE (FINE)	C-10	45 50/4	DR 325			CCI REFUSAL @ 10"
50								
52					RD			
54		— CONTINUED — PREDOMINANTLY SILTSTONE THINLY - MEDIUM BEDDING	C-11	67 50/2	DR 325			CC REFUSAL @ 8"
56					RD			
58	/33	VERY THINLY → THINLY BEDDED	C-12	48 50/3	DR 325			CCI REFUSAL @ 9"
60		60.5 CEMENTED LAYER 4" THICK			RD			SLIGHT DRILL RIG CHATTER
62								
64		- CONTINUED -	C-13	57 70/2	DR 325			CCI REFUSAL @ 8"
66					RD			
68								SHEET 3 OF ___

SOIL CLASSIFICATION AND FIELD DE... ON THE BORING LOGS REPRESENT... FIELD CLASSIFICATIO... THESE LO... EDITED OR MODIFIED TO INCLUDE THE ... ATORY TESTING AND VISUAL CLASS... ...SHOWN ...SED

PROJECT SCRTD 83-1140-26 DATE DRILLED 3-17-84 HOLE NO. 13-8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (FT)	DRILL MODE		CORE REC. %	REMARKS
		PUENTE / FERNANDO FORMATION INTERBEDDED SILTSTONE & CLAYSTONE			RD			
		— CONTINUED —	C-14	69 50/3	DR 325			CCI REFUSAL @ 9"
70								
72					RD			
74		DARK GREENISH GRAY 5GY4/1 FRESH, MASSIVE, THINLY-MEDIUM BEDDING, MODERATELY HARD, MODERATELY STRONG	C-15	61 75/3	DR 325			CCI REFUSAL @ 9"
76					RD			
78								
	31		C-16	97 50/1.5	DR 325			CCI REFUSAL @ 7.5"
80		END OF BORING 80.0' TREMIED A 3 SAC/70 GALLON CEMENT SLURRY INTO HOLE CLEANED SITE						FINISHED DRILLING @ 1415
82								

SOIL CLASSIFICATION AND ... SHOWN ON THE BORING LOGS ... FIELD CLASSIFICATION ... UNEDITED ... EDITED OR MODIFIED ... HAVE NOT BEEN ... RESULTS OF LABORATORY TESTING AND ... CLASSIFICATION

SHEET 4 OF

1981 BORING LOGS

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Boring Log 10

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 80-1280-22 DATE DRILLED 1/31/81 - 2/3/81 HOLE NO. 10
LOCATION Wilshire Blvd. at Valencia St. GROUND ELEV. 320'
DRILLING CONTRACTOR Pitcher LOGGED BY Gallinatti DEPTH TO GROUND WATER ~20'
TYPE OF RIG Failing HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lbs; 30 inches
SURFACE CONDITIONS Sidewalk, business area TOTAL DEPTH 202.0 NO. CORE BOXES 8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0 - 0.5 CEMENT			AD			Begin drilling 9:00 1/31/81. Auger to 2.5', set mud tub & begin rotary drilling.
0.5	CL	0.5 - 12.0 SANDY CLAY: moderate brown (5YR 4/4); ~85% moderate plasticity fines; ~15% fine to medium grained sand; soft, damp.						
2.5					RD			
4								set 4' of 5" casing
6		GRADING SANDIER						
8		(~70% fines, ~30% sand)						
10			I-1	9	SS			1.5/1.5 recovery
				20				
				22				
11.5					RD			
12	SP	12.0 - 18.0 SAND: moderate brown (5YR 4/4); 100% fine to medium grained sand; loose.						
16								material at 20' is saturated.
18								~20' - water table
18	SP	18.0 - 22.5 COBBLED SAND: moderate brown (5YR 4/4); 80-85% fine to medium sand; 15-20% gravel and cobbles; up to 3"; loose, water saturated.						(18'-22.5' perched water table - "underground stream")
20								

SHEET 1 OF 10

PROJECT 80-1380-22 DATE DRILLED 1/31/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	SP	18.0 - 22.5 GRAVELLY SAND: (cont)			DR			DR => 53 blows No RECOVERY - just too. cobbles / gravel - fell out on sample
			S-2	29 30	SS			0.3/0.75 recovery
22		WEATHERED FERNANDO FORMATION 22.5 - 38.0 CLAYSTONE: pale yellowish brown (10 YR 6/2); 100% moderate to high plasticity fines; soft; damp;			RD			DR sample taken w/ 325 lbs. down-hole jars, 5" tube + 1" rings.
24								(GRAVEL probably causing high blow counts material is not dense.
26								
28								30'-31' - Two DR runs just filled w/ cobble sluff
30					DR			DR => 45 Blows NO RECOVERY => gravel sl DR => 53 Blows NO RECOVERY => gravel sl
		Some oxide staining	S-1		PB			
32								2.0/2.5 recovery - some sluff on top.
34			S-3	7 12 25	SS			1.5/1.5 recovery
					RD			
36								
38		FERNANDO FORMATION 38.0 - 202.0 CLAYSTONE: Greyish brown (5YR 3/2); 100% moderate to high plasticity fines; stiff to very stiff; damp; No bedding						11:30 1/31/81 40' - Begin continuous sampling w/ Pitcher Barrel
40	70°	PHYSICAL CONDITION: MASSIVE, low hardness, friable, Fresh.	BOX 1		PB	1		2.7/2.8 recovery
42		40.7 Steep (70°) fracture filled with light colored material - thin.						
						2		
44								

SHEET 2 OF 10

PROJECT 80-1280-22 DATE DRILLED 1/31/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44		380-2020 CLAYSTONE: (cont.)	BOX 2			2		2.8/2.8 recovery
46		PHYSICAL CONDITIONS: massive, low hardness, friable, fresh.				3		2.8/2.8 recovery
48						4		2.8/2.8 recovery
50		no bedding	S-2					
52			BOX 1 (cont.)			5		2.7/2.9 recovery
			BOX 2					
54						6		2.7/2.5 recovery
56						7		2.5/2.5 recovery material is expanding out of tube.
58								
60		distinct smell of methane when clay is cut or broken: "swamp gas" sulphur?	S-3			8		2.3/2.5 recovery
62			BOX 2 (cont.)			9		1.7/2.5 recovery
64						10		2.4/2.5 recovery
66			BOX 3			11		1.7/2.5 recovery
68								

SHEET 3 OF 10

PROJECT 20-1280-22 DATE DRILLED 1/31/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68		38.0 -202.0 CLAYSTONE: (cont)	BOX 3		PB	11		
			S-4			12		2.0/2.5 recovery
70		PHYSICAL CONDITION: massive, low hardness, friable, fresh.						Tube is badly bent, couldn't be extruded. Not an undisturbed sample
72		No Bedding	S-5			13		2.5/2.5 recovery tube bent - can't be extruded
74		Clay still has pungent odor (sulphur/methane ?)	S-6			14		2.5/2.5 recovery * undisturbed sample
76			BOX 3 (cont)			15		1.7/2.5 recovery
78								
80						16		2.5/2.8 recovery
82						17		1.6/2.8 recovery
84						18		2.6/2.8 recovery
86			BOX 4					
88			S-7			19		2.8/2.8 recovery tube is bent, can't be extruded.
90			BOX 4 (cont)			20		1.4/2.8 recovery
92								

PROJECT 80 - 1280 - 22 DATE DRILLED 1/31/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92		38.0 - 202.0 CLAYSTONE: (cont.)			PB	20		
		PHYSICAL CONDITION: massive, low hardness, friable, fresh.	S-8			21		2.8/2.8 recovery * undisturbed sample
94		NO BEDDING						
96		ODOR: (sulphur/methane)				22		0.0/2.8 recovery NO RECOVERY * poor recovery due mainly to bad hole conditions - crooked hole causing tubes to get caught in bit & chewed up.
98						23		0.0/2.8 recovery NO RECOVERY all samples are com... bein... up w/ poor recov... try... no... down w... bit ... exc... but ... help
100								
102						24		0.0/2.8 recovery NO RECOVERY
104		104.2' GAS TEST - No Combustibles 20% O_2						104.2' - stop drilling 1/31/81 - start drilling 2/1/81.
						25		0.0/2.8 recovery bent tube NO RECOVERY
106								still ... problem... w/ ... in ... bottom of ... hole
108						26		0.0/2.8 recovery NO RECOVERY bent tube
110						27		0.0/2.0 recovery bent tube NO RECOVERY
112						28		rig moved; delay while drillers recenter rig. NO RECOVERY - CLAY in the tube, but tube is totally mangled; broken in 3 pieces / unrecoverable
114			BOX 4 (cont.)			29		1.2/2.5 recovery
116								

SHEET 5 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/1/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
116		38.0 - 202.0 CLAYSTONE: (cont.)			PB	29		Bit on Pitcher Barrel cracked; replace bit, clean out hole, continue sampling w/ new bit
		PHYSICAL CONDITION: massive, low hardness, friable, fresh			RD			
118			Box 4 (cont)		PB	30		2.5/2.5 recovery
		No Bedding						
120		ODOR: (sulphur/methane?)				31		1.9/2.5 recovery
122								11:30 - smooth drilling, good recovery.
						32		
124			S-9					2.5/2.5 recovery
			Box 4			33		2.3/2.5 recovery
126			Box 5					
128		~127-130 CLAY MATERIAL IS EXPANSIVE				34		2.6/2.5 recovery (expanded out of tube)
130		130-134 scattered small shell fragments.				35		2.2/2.5 recovery
132						36		2.0/2.5 recovery
134								
136						37		0.0/2.5 recovery. No RECOVERY due to collapsed tube (to much pump pressure)
138			S-10			38		2.5/2.5 recovery
140								

SHEET 6 OF 10

PROJECT 90-1280-22 DATE DRILLED 2/1/91 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
140		38.0 – 202.0 CLAYSTONE: (cont.)	Box 5 (cont.)		PB	39		2.4/2.5
		PHYSICAL CONDITION: massive, low hardness, friable, fresh.	Box 6					
142		No Bedding				40		.5/2.5 recovery collapsed tube
144		ODOR: (Sulphur/Methane?)						
						41		1.9/2.5 recovery
146								
148						42		2.0/2.5 recovery
150						43		2.2/2.5 recovery
152								
			S-11			44		2.5/2.5 recovery * undisturbed sample
154								
			S-12			45		2.5/2.5 recovery bent tube, could not be extruded. – not undisturbed
156								
158						46		0.0/2.5 recovery NO RECOVERY – tube collapsed
160						47		160' – stop drilling for 2/1/91 – Begin drilling 2/2/91 – go down w/ bit and clean out hole; continue sampling
162								0.0/2.8 recovery NO RECOVERY
			S-13			48		S-13: Bent tube, couldn't be extruded; not an undisturbed sample
164								

SHEET 7 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/2/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
164		38.0 - 202.0 CLAYSTONE: (cont.)	S-13		PB	48		2.5/2.8 recovery (NOT an undisturbed sample)
		PHYSICAL CONDITION: massive, low hardness, friable, fresh.						
166		166-168 FAT CLAY	BOX 6 (cont.)			49		2.4/2.8 recovery
		No more sulphur/methane(?) odor.	BOX 7					
168						50		2.5/2.8 recovery
170			S-14					# undisturbed sample
172	(SM) 55	171.9 - 172.4 SILTY SAND layer; ~20% low plasticity fines; ~80% very fine sand; small shell fragments	BOX 7 (cont.)			51		2.1/2.8 recovery
174						52		1.8/2.8 recovery
176		No Odor						
178						53		2.2/2.8 recovery
180		scattered small shell fragments	S-15			54		2.8/2.8 recovery Bent tube - can't be extruded; Not an undisturbed sample
182			BOX 7 (cont.)			55		2.7/2.8 recovery
184			BOX 8					
186			S-16			56		2.8/2.8 recovery # undisturbed sample
188								

SHEET 8 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/2/81 HOLE NO. 10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
188		38.0 – 202.0 CLAYSTONE: (cont.)	BOX 8 (cont.)		PB	57		2.6/2.8 recovery
190								
			S-17			58		2.8/2.8 recovery Battered tube - can't be extruded - NOT an undisturbed sample
192								
194			BOX 8 (cont.)			59		2.7/2.8 recovery
196								
			S-18			60		2.8/2.8 recovery
198								
200						61		0.0/2.8 recovery NO RECOVERY - Mangled tube
202	B.H.	202.0 Bottom of Hole water sampled 1" 2/23/81						202.0' - Terminate Hole; stop circulation at 12:45; last sample out of hole at 1:00; ESA runs e-logs; flush hole, install two piezometers; 2" PVC from 0' to 200' w/ cloth covered perforations from 45' to 60' and 160' to 195'. 1" PVC from 0' to 42' w/ perforations from 22' to 42', Bentonite Plug from 42' to 47'. Clean-up site. Move off site 9:00 2/3/81

SHEET 9 OF 10

SUMMARY BORING NO. 10

PROJECT 80-1230-10 STATION HOLE no DATE DRILLED 1/31/81 – 2/3/81

OVERBURDEN DEPTH (FT.) 0' TO 22.5'.

BEDROCK DEPTH (FT.) 22.5' TO 202' (T.D.).

WATER PRESS. TEST no; INTERVAL(S) ___ TO ___, ___ TO ___.

GROUND WATER DEPTH (FT.) ~20' DATE 1/31/81; ___ DATE ___.

GAS TRACE; DEPTH FIRST NOTICED ___, DATE ___.

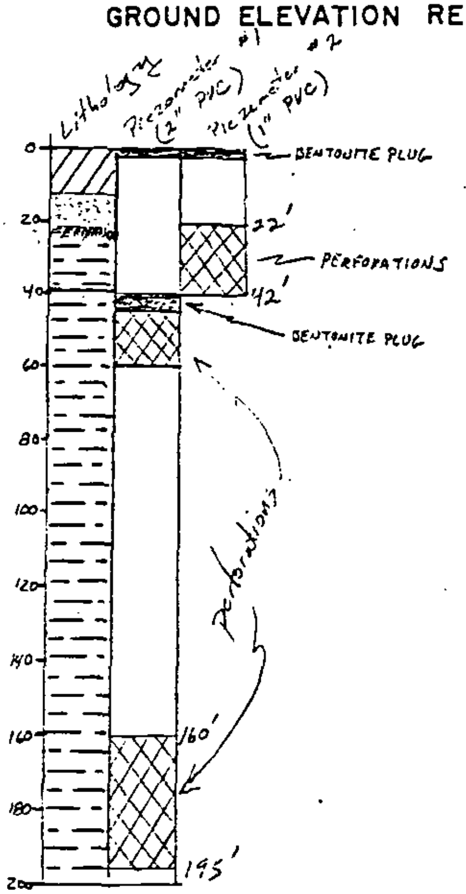
E – LOG yes.

DOWN-HOLE SURVEY no.

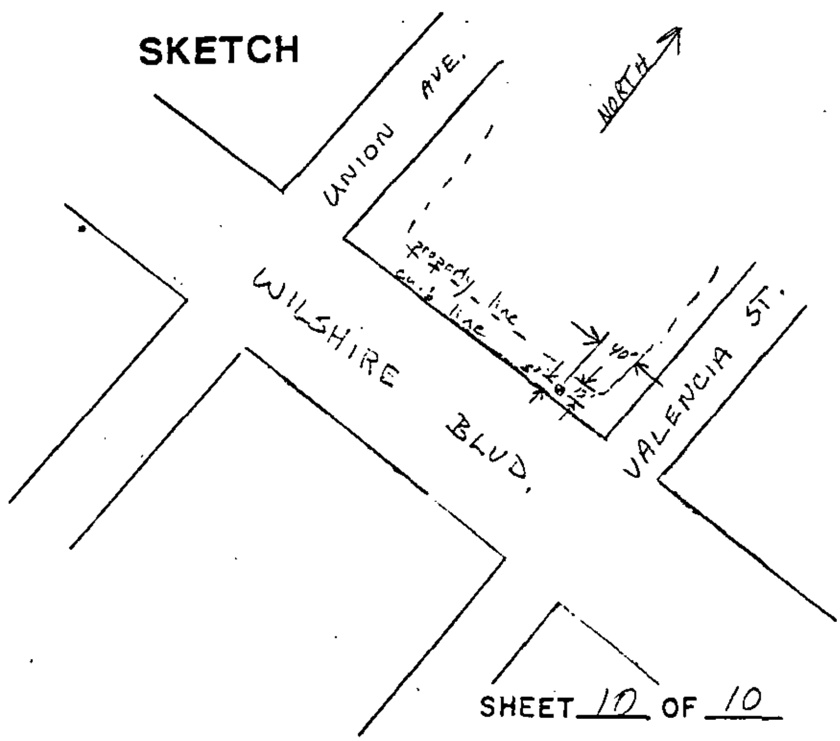
CROSS-HOLE SURVEY no.

PVC CASING (I.D.): 4" ___ TO ___; 3" ___ TO ___; 2" 0' TO 200'. 1" 0' to 42'

GROUND ELEVATION REF. 320'.



SKETCH



SHEET 10 OF 10

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Earth Sciences Associates
Geo/Resource Consultants

Boring Log 11

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 80-1280-22 DATE DRILLED 29-31 Jan '81 HOLE NO. 11
LOCATION south side Wilshire Bl. just east of Alvarado St. GROUND ELEV. 271'
DRILLING CONTRACTOR J.N. Pitcher Co. LOGGED BY S. Slaff DEPTH TO GROUND WATER 29'
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 5/8" HAMMER WEIGHT AND FALL 140 lbs 30 inches
SURFACE CONDITIONS asphalt over concrete TOTAL DEPTH 201.1' NO. CORE BOXES

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0		0.0'-0.1' ASPHALT 0.1'-0.9' CONCRETE			AD			Began drilling 10:00 29 Jan. 1981. Drilled 0.0'-0.9' with "garbage barrel" (cylindrical bit).
	ML	0.9'-20.9' CLAYEY SILT: Mottled: light olive brown (5Y 5/6) and light olive (10Y 5/4); 98% moderate plasticity fines; 2% fine sand; medium stiff; dry; sand is subangular.						Drilled 0.9'-3.0' with 6" flight auger. Set 6" steel surface casing to 3.0' with 2' stick-up. Drilled 3.0'-5.0' with 4 5/8" drag bit.
2.0					RD			
4.0								
		As above, mottled with dusky yellow (5Y 6/4) and moderate brown (5YR 4/4); 98% low plasticity fines; 2% fine sand; micaceous; spongy.	J-1	6 9 12	SS			1.5/1.5 recovery
6.0					RD			
8.0								
10.0			J-2	7 12 32	SS			1.5/1.5 recovery
12.0					RD			
14.0								
	CL 45	14.5'-16.1' SILTY CLAY: grayish olive green (5GY 3/2); 98% low plasticity fines; 2% fine sand; stiff; dry to moist.	J-3	10 14 21	SS			1.5/1.5 recovery
16.0	ML	As above, becoming banded; stiff, moist, weakly foliated.			RD			
18.0			C-1	14 34	DR			Drive sample using Modified California sampler and 325 lb. down-hole jars. 1.0/1.0 recovery
20.0								

SHEET 1 OF 10

PROJECT 80-1280-22 DATE DRILLED 29 January 1981 HOLE NO. 11

DEPTH	CLASS.		FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0		ML	0.9'-20.9' CLAYEY SILT (continues)	J-4	14 26 38	SS			1.5/1.5 recovery 13:30
			20.9'-201.1' PUENTE FORMATION: SILTSTONE: grayish olive green (5GY 3/2); 95% moderate plasticity fines; 5% very fine sand; no dilatancy; medium plasticity; hard; dry; micaceous; glauconitic(?). PHYSICAL CONDITION: massive; soft to friable (hardness); soft to friable (strength); fresh.			RD			
22.0									drill rate = .8min/ft.
24.0									
			As above; small amount of clay cement; sulfurous-organic odor.	S-1		PB	1		2.6/2.8 recovery
26.0									
28.0		51 Y		box #1			2		1.8/2.8 recovery drilling with 230 p.s.i. hydraulic draw-down.
30.0									standard penetration >4.5 tons/ft.²
			Grading from thickly to thinly bedded.				3		2.8/2.8 recovery
32.0		55 Y							
34.0							4		2.7/2.8 recovery drill rate = .8min/ft.
			Organic-sulfurous odor decreasing.						14:30
36.0							5		2.8/2.8 recovery
38.0			Grading from dry to moist.	box #2					
40.0		52 Y					6		2.4/2.8 recovery
42.0				S-2			7		2.0/2.8 recovery
44.0									

SHEET 2 OF 10

PROJECT 80-1280-22 DATE DRILLED 29-30 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		20.9'-201.1' SILTSTONE: (continues)	S-2		PB	7		2.0/2.8 recovery
		PHYSICAL CONDITION: (continues as previously described).	box #2			8		2.8/2.8 recovery
46.0								
48.0	45	SILTSTONE contains interbedded uncemented sandstone. Sandstone beds are greenish gray (5GY 6/1), .25-1.5" thick, comprise 5-20% of siltstone unit.				9		2.1/2.8 recovery
50.0			box #3			10		2.8/2.8 recovery
52.0								
54.0	58	Some siltstone beds are dusky yellow (5Y 6/4), comprise 1-5% of siltstone unit.				11		2.8/2.8 recovery
56.0		Samples break easily along bedding planes; most easily along sandstone bedding planes.				12		2.8/2.8 recovery
58.0		No sulfurous-organic odor.	S-3			13		2.6/2.8 recovery
60.0								
62.0	50		box #4			14		2.2/2.8 recovery Did not add any chemicals to drilling fluid all day.
64.0								64.2' end of 29 Jan '81
						15		groundwater at 6.1' at 07:00 30 Jan. '81. 2.8/2.8 recovery drilling with 200 p.s.i. hydraulic draw-down.
66.0								
68.0						16		2.8/2.8 recovery

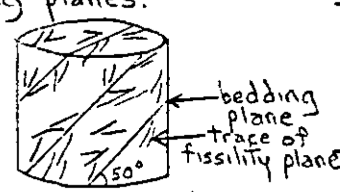
SHEET 3 OF 10

PROJECT 80-1280-22 DATE DRILLED 30 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		20.9'-201.1' SILTSTONE: (continues) PHYSICAL CONDITION: (continues as previously described).	box #4		PB	16		2.8/2.8 recovery
70.0	58 Y		box #5			17		2.7/2.8 recovery
		Scattered coarse sand-sized grains of organic matter, apparently wood.						
72.0						18		2.7/2.8 recovery
74.0								
76.0			S-4			19		2.8/2.8 recovery
78.0			box #5			20		2.8/2.8 recovery
60.0	47 Y							
82.0			box #6			21		2.8/2.8 recovery
84.0						22		2.8/2.8 recovery
86.0			S-5			23		2.3/2.8 recovery Sample stuck in Pitcher tube.
88.0								
90.0	53 Y		S-6			24		2.7/2.8 recovery
92.0								

PROJECT 80-1280-22 DATE DRILLED 30 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92.0		20.9'-201.1' SILTSTONE: (continues). PHYSICAL CONDITION: (continues as previously described).	S-6 box #6		PB	24 25		2.7/2.8 recovery 2.3/2.8 recovery
	51 Y							
94.0								
		Grading from thinly bedded to massive; decreasing proportion of sandstone beds.	box #7			26		2.8/2.8 recovery
96.0								
98.0		Sandstone beds <5% of siltstone unit.				27		2.7/2.8 recovery drill rate = .6'/minute
100.0	49 Y							
						28		2.6/2.8 recovery
102.0								
			box #8			29		1.8/2.8 recovery easy drilling
104.0								
	49 Y							
106.0								
		Siltstone has very slight fissility (probably due to sub-parallel orientation of mica grains). Bedding is indistinct at this depth, but fissility seems to be along planes 10-30° to bedding planes.	S-7			30		2.8/2.8 recovery
108.0								
			box #8			31		2.0/2.8 recovery 11:00 drill rate = .9ft/minute
110.0								
	50 Y							
112.0						32		2.8/2.8 recovery
114.0								
						33		2.8/2.8 recovery
116.0								



Cross Section View

SHEET 5 OF 10

PROJECT 80-1280-22 DATE DRILLED 30 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
116.0		20.9'-201.1' SILTSTONE: (continues). PHYSICAL CONDITION: (continues as previously described).	box #8		PB	33		2.8/2.8 recovery
118.0	48/		box #9			34		2.4/2.8 recovery
120.0						35		2.3/2.8 recovery
122.0		Bedding more distinct.	S-8			36		2.7/2.8 recovery
124.0								
126.0	51/		box #9			37		1.9/2.8 recovery
128.0		Sandstone beds are thicker than above - up to 2.5" thick.				38		2.6/2.8 recovery
130.0								
132.0	53/	Siltstone is quartz-rich and contains mafic minerals. Probably a near-shore deposit derived from metamorphic and igneous rocks.	box #10			39		2.7/2.8 recovery
134.0						40		2.1/2.8 recovery
136.0						41		2.4/2.8 recovery
138.0	49/							2.8/2.8 recovery
140.0								

SHEET 6 OF 10

PROJECT 80-1280-22 DATE DRILLED 30 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
140.0	49	20.9'-201.1' SILTSTONE: (continues). PHYSICAL CONDITION: (continues as previously described).	S-9		PB	42		2.8/2.8 recovery
142.0			box #10			43		2.7/2.8 recovery
144.0			box #11					
146.0						44		1.9/2.8 recovery
	51	147.1-147.6 SANDSTONE: dark greenish gray (5G 4/1); 90% fine sand; 10% medium plasticity fines; no dilatancy; medium plasticity; very dense; moist; micaceous (sericite or muscovite); sand is rounded; quartz-rich; uncemented; friable.						
148.0						45		1.9/2.8 recovery
150.0						46		2.5/2.8 recovery
152.0								14:20 2% combustible gas
154.0		Siltstone is massive, unfractured.	box #12			47		2.7/2.8 recovery
156.0			S-10			48		2.7/2.8 recovery
158.0								
160.0			box #12			49		2.8/2.8 recovery
162.0	47					50		2.8/2.8 recovery
164.0								

PROJECT 80-1280-22 DATE DRILLED 30-31 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
164.0		20.9'-201.1' SILTSTONE: (continues). PHYSICAL CONDITION: (continues as previously described).	box #13		PB	50		2.8/2.8 recovery
						51		2.8/2.8 recovery
166.0	51 Y							
168.0						52		2.8/2.8 recovery
170.0						53		2.1/2.8 recovery
172.0	52 Y							
174.0						54		2.7/2.8 recovery
176.0			S-11			55		2.6/2.8 recovery
178.0								end of day 30/1/81
			box #14			56		2.6/2.8 recovery 31/1/81 07:00 Artesian water flow from hole at .25 gal/min with 3' hydrolic head. Temperature = 81° F. Small gas bubbles (.2" diameter) rising constantly. Large gas bubbles (2" diameter) rise 6 times/minute. 3-4% combustible gas background with kicks up to 12%.
180.0	52 Y							
182.0						57		2.7/2.8 recovery
		sandstone beds interbedded in siltstone are greenish gray (5GY 6/1); 75% fine sand; 25% moderate plasticity fines; very dense; moist; micaceous; uncemented; beds are .25"-.75" thick; comprise ~5% of sample.						drilling with 200 p.s.i. hydraulic draw-down.
184.0						58		2.8/2.8 recovery
186.0	52 Y		box #15					2.8/2.8 recovery
188.0			S-12			59		

SHEET 8 OF 10

PROJECT 80-1280-22 DATE DRILLED 31 Jan. 1981 HOLE NO. 11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
188.0		20.9'- 201.1' SILTSTONE: (continues) PHYSICAL CONDITION: (continues as previously described)	S-12		PB	59		2.8/2.8 recovery
	48/							
190.0								08:55
			box #15			60		2.6/2.8 recovery gas pocket: large bubbles and tar in drilling fluid.
192.0	50/							
						61		2.8/2.8 recovery
194.0								standard penetration >4.5 tons/ft.²
196.0						62		2.8/2.8 recovery
	50/							
198.0			box 16					10:15 15% combustible gas.
			S-13			63		2.5/2.5 recovery
200.0								
		B.H. 201.1' Terminated hole.						10:30 31 January 1981 Reamed hole, ran electric logs. 13:15-artesian water flow rate = 1gal/6minutes with 3' hydrolic head. 13:30-artesian flow rate = 1gal/4.5min. with 3' head. Conducted down-hole seismic testing. 16:30-artesian flow rate = 1gal/2min. with 3' head. 1gal/1.3min. with 2' head. 17:00 - artesian flow rate = 1gal/min with .5' head.
202.0								07:00 1 February 1981 artesian flow rate = 1gal/1.3min with .5' head. Collected water sample for analysis.
								06:45 2 February 1981 artesian flow rate = 1gal./1.3min. Gas bubbles still rising through water. Conducted gas analysis with chromatograph. 09:15 After bailing hole down to 50.0' determined artesian aquifer is at 29.0' water sampled 2/7/81

SHEET 9 OF 10

SUMMARY BORING NO. 11

PROJECT 80-1280-22 STATION HOLE YES DATE DRILLED 29-31 Jan. 1981.

OVERBURDEN DEPTH (FT.) 0.9' TO 20.9'.

BEDROCK DEPTH (FT.) 20.9' TO 201.1' (T.D.).

WATER PRESS. TEST YES; INTERVAL(S) 63' TO 83', 35' TO 55', 55'-75',

GROUND WATER DEPTH (FT.) 29' DATE 2 Feb '81; artesian DATE 31/1/81.

GAS YES; DEPTH FIRST NOTICED 153.0', DATE 30 Jan '81

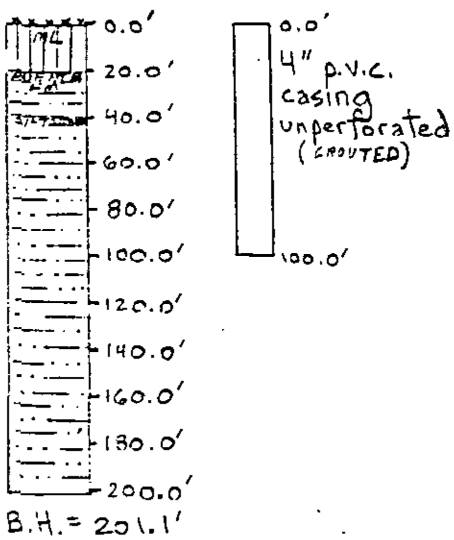
E-LOG YES.

DOWN-HOLE SURVEY YES.

CROSS-HOLE SURVEY YES.

PVC CASING (I.D.): 4" 0.0' TO 100'; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. 271'.



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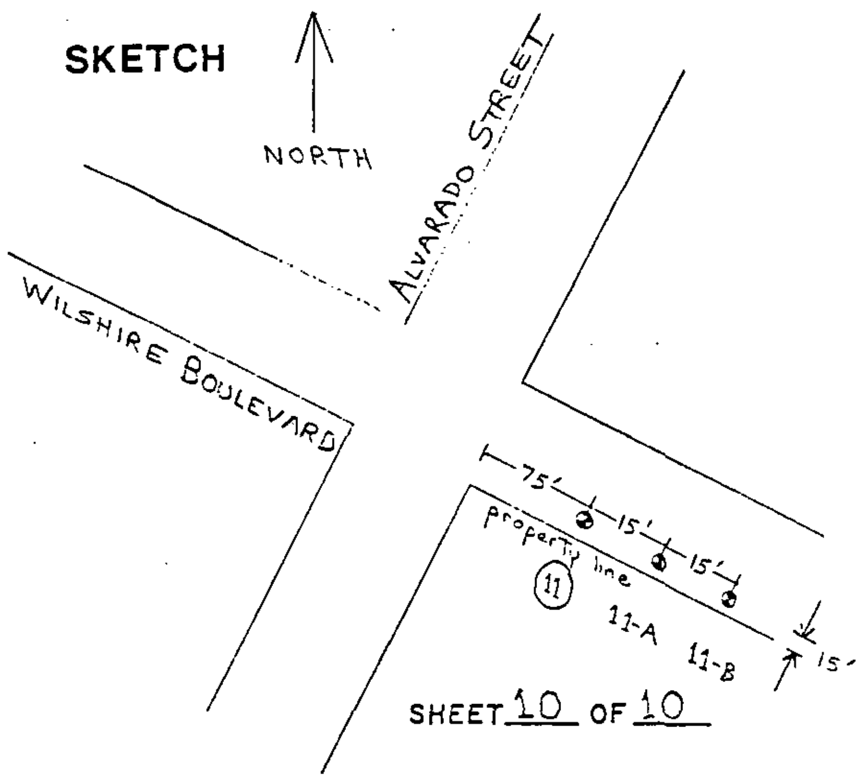

SHEET 10 OF 10

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Boring Log 12

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 80-1280-22 DATE DRILLED 2/11/81 - 2/13/81 HOLE NO. 12
LOCATION Wilshire Blvd. west of Hoover St. GROUND ELEV. 245'
DRILLING CONTRACTOR Pitcher LOGGED BY Gallinatti DEPTH TO GROUND WATER none
TYPE OF RIG Failing HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lbs; 30 in.
SURFACE CONDITIONS Sidewalk next to park TOTAL DEPTH 200.1 NO. CORE BOXES 14

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0		0.0-1.0 CEMENT			AD			Begin drilling 9:30 2/11/81. Auger to 6'; set 7' of 5" surface casing and begin rotary drilling.
1	CL	1.0-8.0 GRAVELLY CLAY: light brown (5YR 5/6); ~80% moderate plasticity fines; ~20% fine sand to 1/4" gravel; medium stiff; damp.						
4								[P.P.] stands for pocket penetrometer: data obtained from field-extruded PB samples.
6					RD			
8	CL	8.0-10.0 CLAY: medium dark gray (N4); ~95% moderate plasticity fines; ~5% fine grained sand; firm; damp.						
10	CL	10.0-32.0 SANDY CLAY: Light brown (5YR 5/6); ~80% fines moderate plasticity; ~20% fine grained sand; soft; damp.	S-1	3 / 5 / 8	SS			1.3/1.5 recovery
12					RD			
14		increase in SAND content						
16								
18		~30-40% fine to coarse sand; material is soft & mottled, could be fill						
20								SHEET 1 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/11/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20	CL	10.0-32.0 SANDY CLAY: (cont)	C-1		DR			DR ⇒ 12 blows 0.8/1.0 recovery
				2	SS			0.0/1.5 recovery
22				3				
				3				
					RD			DR sample taken w/ modified California sampler, using 32.5 lbs down-hole jars; C-1 taken w/ 5" tube plus 1" ring.
24		Decrease in SAND content						
26								
28								
30			J-2	6	SS			1.5/1.5 recovery
				10				
		PUENTE FORMATION:		12				
32		32.0-77.8 SILT and CLAYSTONE: thinly bedded (0.05' to 0.2' thick); ~50% SILT layers and 50% laminated CLAYSTONE layers: SILT is brownish gray (5YR 4/1); 100% low plasticity fines; stiff; damp; easily carved with knife (P.P. = 3.5); CLAYSTONE dk. yellowish brown (10YR 4/4); 100% moderate plasticity fines; hard; carved with difficulty with knife (P.P. >5); damp to dry. Occasional thin SAND laminations.			RD			34' - Begin continuous Pitcher Barrel sampling 11:30 2/11/81
34			BOX 1		PB	1		2.2/2.8 recovery
36	50							
						2		2.7/2.8 recovery
38								
40	50	35' - 90° turn in strike of bedding; no apparent fracture, & dip remains constant				3		2.8/2.8 recovery
	40							
42		32' - 50' Sulphur smell when material is cut w/ knife or broken				4		2.7/2.8 recovery
44								

SHEET 2 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/11/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44		32.0-77.8 SILT and CLAYSTONE: (cont.)	BOX 1 / BOX 2		PZ	4		
46		44' - disrupted bedding 0.4' 	S-1			5		2.8/2.8 recovery
48	40	[P.P. > 5]	BOX 2 (cont.)			6		2.4/2.8 recovery
50		50' - disrupted bedding 0.4'  no fractures				7		2.8/2.8 recovery
52	40	53' - disrupted bedding						
54	40	0.4'  no fractures				8		2.9/2.8 recovery
56			BOX 3			9		2.7/2.8 recovery
58								
60	40	61.6' - Disrupted bedding				10		2.7/2.8 recovery
62		0.4'  crushed cemented material	S-2			11		1.6/2.4 recovery
64			BOX 3 (cont.)			12		2.3/2.8 recovery
66	40							
68						13		

SHEET 3 OF 10

PROJECT 80-1090-22 DATE DRILLED 2/11/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68		32.0 - 77.8 SILT and CLAYSTONE: (cont.)	BOX 3		PB	13		2.8/2.8 recovery
70	40	[P.P. > 5]	BOX 4					
						14		2.6/2.8 recovery
72								
74						15		2.8/2.8 recovery
76						16		2.0/2.0 recovery
			S-3					
78		77.8 - 81.5 SILTSTONE: very hard, cemented siltstone; barely scratches w/ knife.			RD			cemented zone - rotary drill thru it
80								
		81.5 - 200.1 SILTSTONE w/some thin (0.05') CLAYSTONE interbeds:			PB	17		2.8/2.8 recovery
82		Same as 32.0-77.8 except claystone makes up only ~20% of the rock; thin SAND laminations make up ~5% of the rock.	S-4					
84			BOX 4 (cont.)			18		2.7/2.8 recovery
	40							
86		[P.P. > 5]						
			BOX 5			19		2.8/2.8 recovery
88		80'-90' Sulphur smell when material is cut or broken						
90	40					20		2.7/2.8 recovery
92								

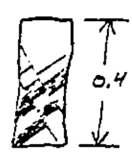
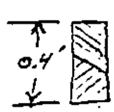
SHEET 4 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/12/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92		81.5 - 200.1 SILT w/ CLAYSTONE: (cont.)	BOX 5		PB	21		2.8/2.8 recovery
94		PHYSICAL CONDITIONS: no fractures; can be gouged w/ a knife; friable; [P.P. >5].						
	40					22		2.8/2.8 recovery
96			BOX 6					
98			S-5			23		2.8/2.8 recovery
100	40							
			BOX 6 (cont.)			24		2.8/2.4 recovery
102								
						25		2.8/2.8 recovery
104								
	40							
106						26		2.6/2.8 recovery
108			BOX 7					
						27		2.8/2.8 recovery
110								
	40							
112						28		2.8/2.8 recovery
114								
			S-6			29		
116								

SHEET 5 OF 10

PROJECT: 80-1280-22 DATE DRILLED 2/12/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
116		81.5 - 200.1 SILT w/ CLAYSTONE: PHYSICAL CONDITION: no fractures; can be gouged w/ a knife; friable to moderately strong; [p.p. >5]	S-6		PB	29		2.8/2.8 recovery ★ undisturbed sample
118	40		S-7			30		2.8/2.8 recovery Bent tube - can't be extruded
120			Box 7 (cont.)			31		120.2 - stop 2/11/81 Begin 2/12/81 1.5/2.8 recovery
122								
124	40		Box 8			32		2.7/2.8 recovery
126						33		2.7/2.8 recovery
128		 0.4'				34		2.8/2.8 recovery
130	40	130.2' - 131.0' - some "micro-faults" with small but distinct offset of 2-3 mm; "faults" dipping ~60°				35		2.8/2.8 recovery
132		132.1' - disrupted bedding						
134		 0.4'	S-8			36		2.8/2.8 recovery ★ undisturbed sample
136	40		Box 9			37		2.8/2.8 recovery
132								
140								

SHEET 6 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/12/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
140		81.5 – 200.1 SILT w/ CLAYSTONE: (contd)	BOX 9		PB	38		2.8/2.8 recovery
142		PHYSICAL CONDITION: no fractures; friable to moderately strong; can be gouged w/ a knife with some difficulty; [pp > 5]						
	40					39		2.7/2.6 recovery
144								
146	50	145.5 – small "microfault" with 5 mm displacement dipping ~60°	BOX 10			40		2.8/2.8 recovery
148			S-9			41		2.8/2.8 recovery Bent tube – can't be extruded
150								
152		152.2 – disrupted bedding 	BOX 10 (cont.)			42		2.2/2.2 recovery
154	45					43		2.8/2.8 recovery
156								
158			S-10			44		1.9/2.2 recovery & undisturbed sample
			BOX 10 (cont)			45		2.3/2.5 recovery
160	45	160'–161' burrowing / slumping ?? caused many irregularities in the bedding.						
162			BOX 11			46		2.6/2.7 recovery
164								

SHEET 7 OF 10

PROJECT 80-1280-22 DATE DRILLED 2/12/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
164		81.5 – 200.1 SILT w/ CLAYSTONE: (cont.)	S-11		PB	47		0.8/1.0 recovery
			BOX 11 (cont.)			48		WON'T EXTRUDE - too tight.
166								2.8/2.8 recovery
								POSSIBLY BAD TUBE OR EXPANSIVE MATERIAL
	50	167.2' Disrupted bedding  0.4'						
168						49		2.4/2.8 recovery
170								
			S-12			50		2.5/2.5 recovery
172								
			BOX 11 (cont.)			51		2.0/2.0 recovery
174	40							
			BOX 12			52		2.2/2.7 recovery
176								
178						53		2.8/2.8 recovery
	40							
180						54		2.8/2.8 recovery
182								
184						55		2.8/2.8 recovery
			BOX 13					
186	50							
			S-13			56		2.7/2.7 recovery ★ undisturbed sample
18[illegible]								SHEET 8 OF 10

PROJECT 90-1280-22 DATE DRILLED 2/12/81 HOLE NO. 12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
188		81.5 - 200.1 SILT w/ CLAYSTONE:	S-13		PB	56		
		physical condition: no fractures, gouged w/ difficulty with a knife; [p.p. >5]	BOX 13 (cont.)			57		2.4/2.8 recovery
190	50							
192						58		2.8/2.8 recovery
194						59		2.0/2.8 recovery
196		196' - zone of bioturbated material (~0.4' thick)						
198	50		BOX 14			60		2.7/2.8 recovery
200	B.H.	200.1 Bottom of Hole						2/12/81 Terminate Hole at 200.1'; stop circulation at 4:30, last sample out of hole at 4:45. 2/13/81 - ESA e-logs hole. Instal piezometer (2" PVC) with cloth-covered perforations from 60' to 80' and from 160' to 195'. gravel packed to surface. Clean-up and move off. Off site 3:00 2/13/81, water sampled 2/16/81

SHEET 9 OF 10

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Summary Data
Boring No. 12

SUMMARY BORING NO. 2

PROJECT 80-1280-22 STATION HOLE no DATE DRILLED 2/11/81-2/13/81

OVERBURDEN DEPTH (FT.) 0' TO 32'.

BEDROCK DEPTH (FT.) 32' TO 200.1' (T.D.).

WATER PRESS. TEST no; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) ____ DATE ____; ____ DATE ____.

GAS TRACE (H_2S) DEPTH FIRST NOTICED ____, DATE ____.

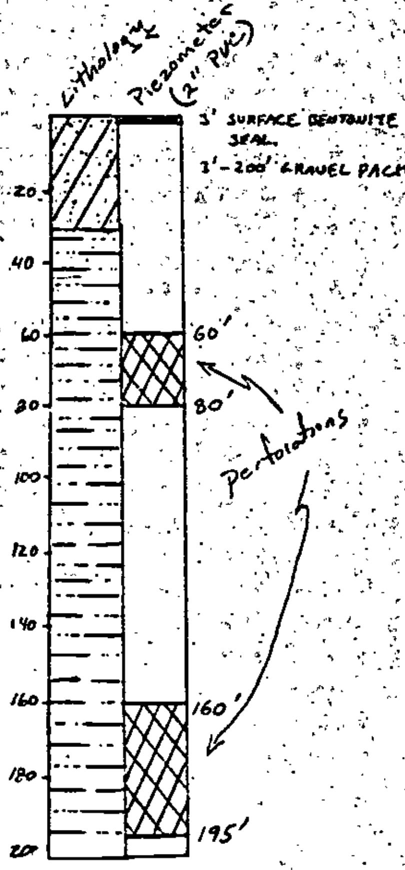
E-LOG yes.

DOWN-HOLE SURVEY no.

CROSS-HOLE SURVEY no.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" 0' TO 200'.

GROUND ELEVATION REF. 245'



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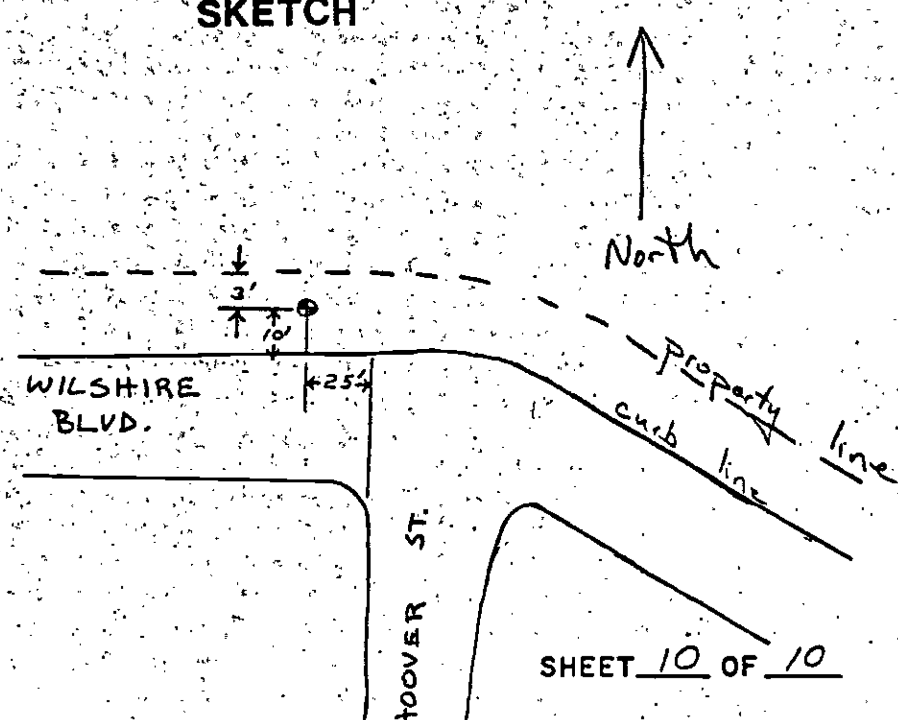


SHEET 10 OF 10

APPENDIX B - GEOPHYSICAL EXPLORATIONS

B.1 DOWNHOLE SURVEY

B.1.1 Summary

A downhole shear wave velocity survey was performed in Boring CEG 11 in Design Unit A 170. This boring is located near the Wilshire/Alvarado Station. Measurements were made at 5 foot-intervals from the ground surface to a depth of 200 feet. A description of the technique and a summary of the results are presented in this appendix.

B.1.2 Field Procedure

Shearing energy was generated by using a sledge hammer source on the ends of a 4- by 6- inch timber positioned under the tires of a station wagon, tangential to each borehole. A 12-channel signal enhancement seismograph (Geometrics Model ES 1210) allowed the summing of several blows in one direction when necessary to increase the signal-to-noise ratio. Shear waves were identified by recording wave arrivals with opposite first motions on adjacent channels of the seismograph.

B.1.3. Data Analysis

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

B.1.4 Discussions of Results

Estimated velocities are summarized in Table B-1. Velocity estimates are based on selections of linear portions of the downhole arrival time curves.

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

In general, near-surface (upper 65 feet) shear wave velocities of Borehole CEG 11 were found to be on the order of 1,180 ±110 feet per second. From depths of 65 to 200 feet, shear velocity estimates increase to 1,360 ±110 feet per second.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed in Boring CEG 11. The crosshole technique for determining shear wave velocities in-situ was utilized in a three-borehole array at this location. Compressional and shear velocity estimates were obtained (see Figure B-2, B-3). Crosshole seismic wave measurements were performed in an array of three boreholes spaced approximately 15 feet apart. 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 to the walls by means of expanding heavy rubber balloons which protrude from one side of the geophone housings. The hammer is then used to create vertically polarized shear waves with either an up or down first motion. A 12-channel signal enhancement seismograph with oscilloscope and electrostatic paper camera is used as a signal storage device.

B.2.3. Data Analysis

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

B.2.4. Discussion of Results

Seismic wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet. The wave velocity is equal to the difference in travel path distance from the generating source to each geophone divided by the difference in shear wave arrival times. The results of the compressional and shear wave velocity analyses are shown in Figures B-2 and B-3 and are summarized in Table B-2.

TABLE B-1
DOWNHOLE VELOCITIES

Boring No.	Depth (ft)	COMPRESSIONAL WAVE $\bar{V}p$	σp	Ep	Np	Vp*	SHEAR WAVE $\bar{V}s$	σs	Es	Ns	Vs*
11	10–65	5656	315	283	12	5660±600	1177	3	59	12	1180±60
	65–200	4909	260	245	28	4910±710	1356	41	68	28	1360±110

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

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

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

Es = estimated accuracy of shear survey

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

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

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

TABLE B-2
CROSSHOLE VELOCITIES

Boring No.	Depth (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\overline{V}p$	σp	Ep	Np	Vp*	$\overline{V}s$	σs	Es	Ns	Vs*
11	10	4649	197	232	4	4650±430	943	0	47	4	940±50
	15	5287	654	264	4	5290±920	921	3	46	6	920±50
	20	5284	25	264	4	5280±290	1157	8	58	8	1160±70
	25	5433	545	272	4	5430±810	1318	28	66	9	1320±90
	30	5355		536	1	5360±540	1101	42	55	6	1100±100
	35	5353	157	268	7	5350±420	1182	2	59	8	1180±60
	40	5121		512	1	5120±510	1277	23	64	5	1280±90
	45	4567	428	228	4	4570±660	1088	11	54	5	1090±70
	50	4660	26	233	4	4660±260	1237	56	62	8	1240±120
	55	5220		522	1	5220±520	1356	11	68	9	1360±80
	60	4637		464	2	4640±460	1238	38	62	9	1240±100
	65	4351	191	218	7	4350±410	1411	15	71	10	1410±90
	70	4701		470	2	4700±470	1530	40	77	10	1530±120
	75	4610		461	1	4610±460	1387	56	69	12	1390±130
	80	4573	55	230	4	4570±280	1532	66	77	8	1530±140
	85	4495		450	2	4500±450	1488	0	74	8	1490±70
	90	4220		422	3	4220±420	1102	9	55	11	1100±60
	95	4014	326	201	9	4010±530	1360	40	68	12	1360±110
	97	4400		440	1	4400±440	1524	23	76	13	1520±100

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

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

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

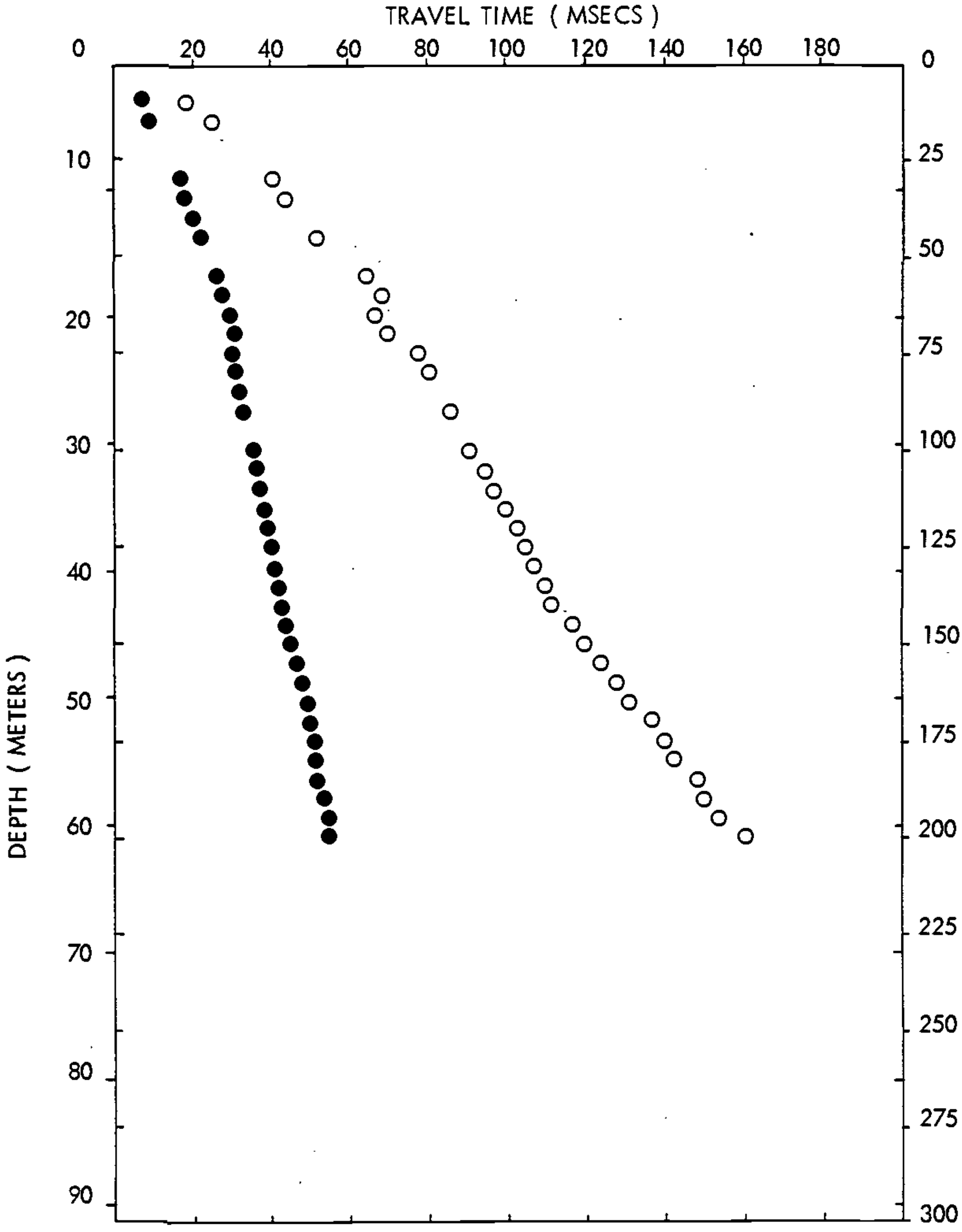
Es = estimated accuracy of shear survey

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

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

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



COMPRESSIONAL WAVE (Solid Circles)
SHEAR WAVE (Open Circles)

DOWNHOLE TRAVEL TIME PROFILE - BORING 11

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METRO RAIL PROJECT

Project No. 83-1101

Figure No. B - 1

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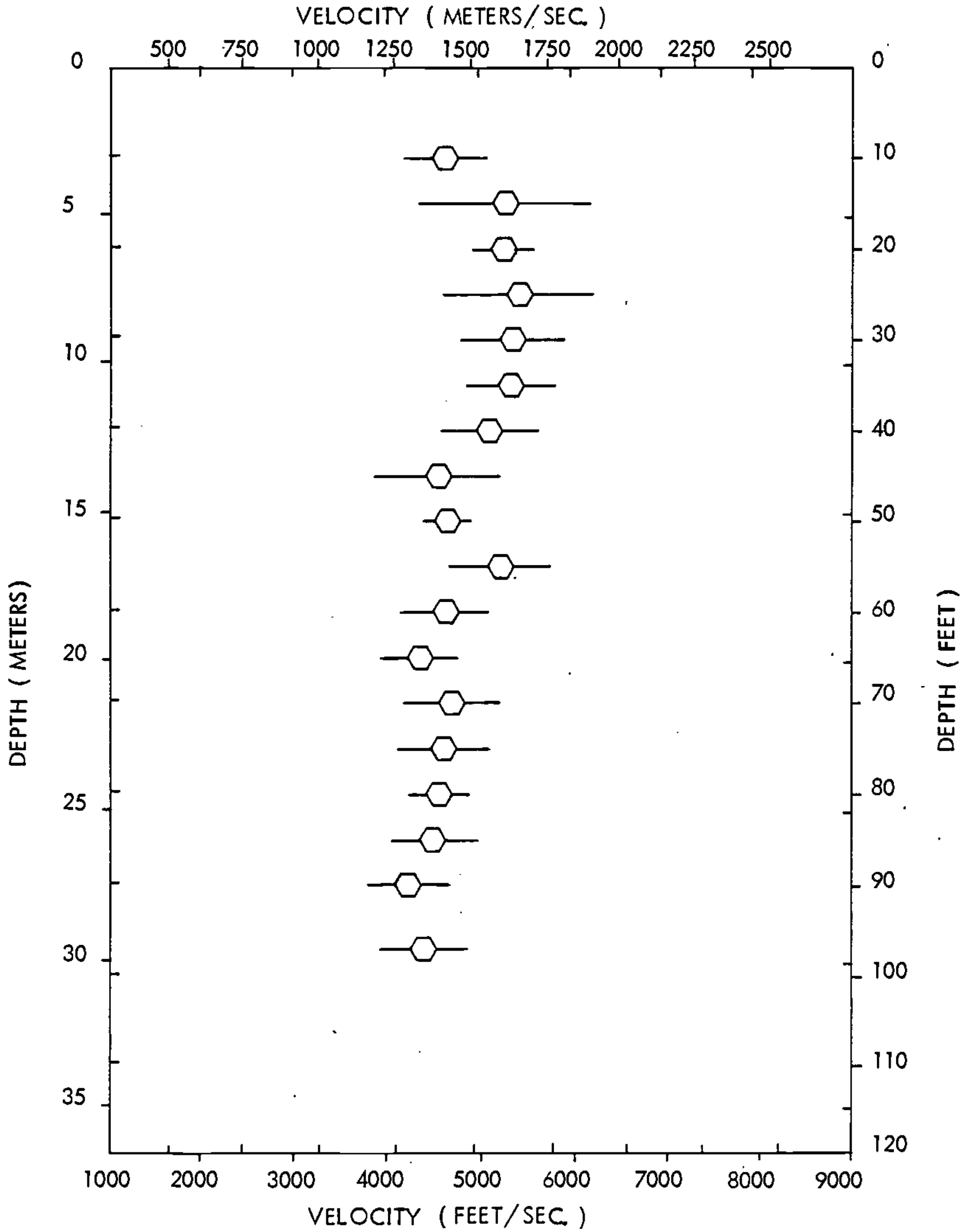

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 11

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Project No. 83-1101

Figure No. B - 2

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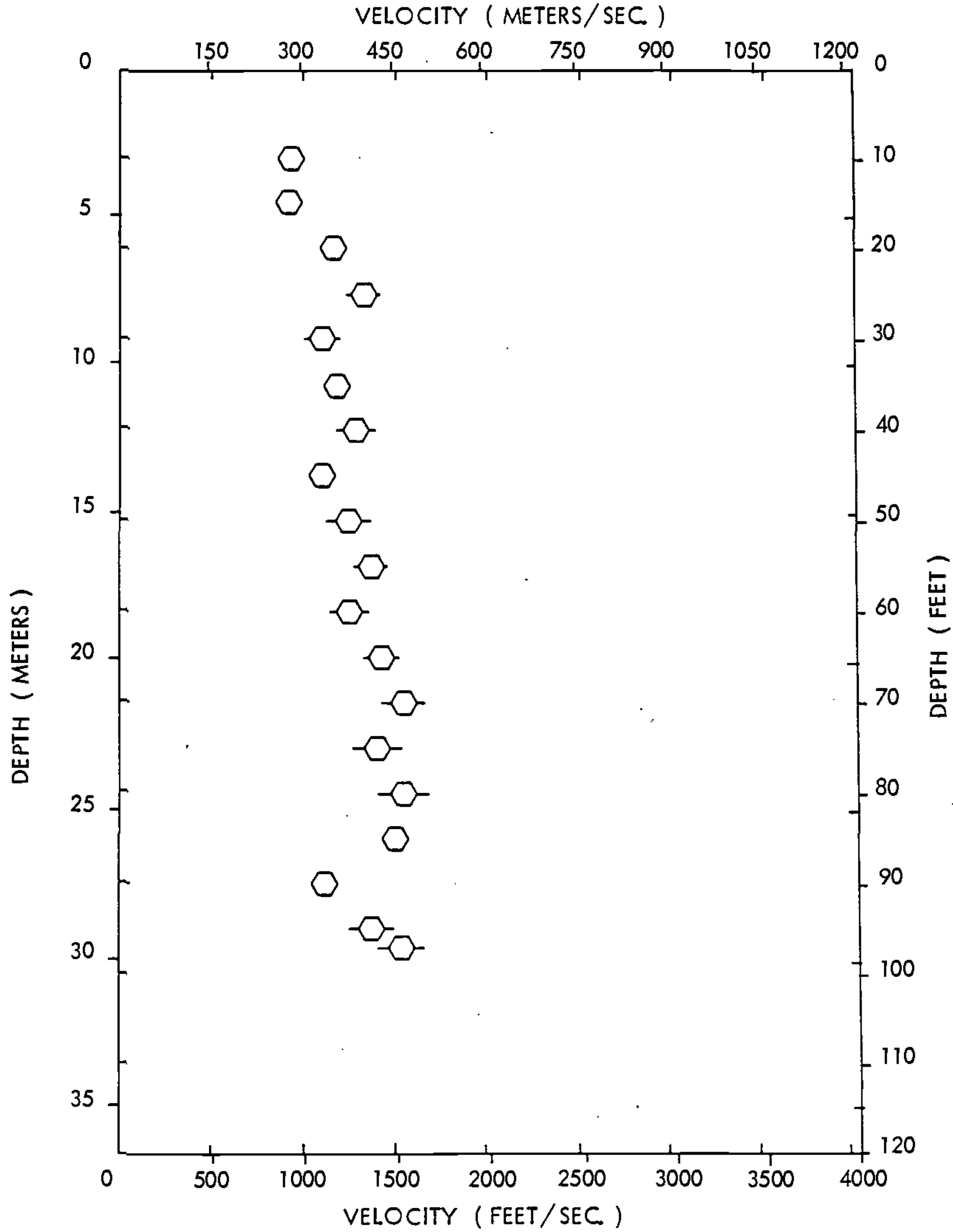

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 11

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83-1101

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Figure No.
B - 3

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Appendix C

Oil and Gas Analyses

APPENDIX C-GAS CHROMATOGRAPHIC ANALYSIS

C.1 INTRODUCTION

Concentrations of certain gases are known to result in fires and explosions in tunnels; methane is the gas most commonly associated with such hazards. Methane and other natural hydrocarbon gases are expected to occur along the proposed Metro Rail tunnel alignment, especially where the alignment crosses oil fields. Certain non-hydrocarbon gases can be corrosive or result in health hazards to the miners, and these gases are also expected. These gases include hydrogen sulfide, carbon monoxide, and carbon dioxide. To provide a measure of the distribution and extent of of the hazardous hydrocarbon and non-hydrocarbon gases, a program of in-situ quantitative analyses was conducted by Converse's special consultant, RYLAND-CUMMINGS, INC.

The hydrocarbon gases tested were: methane; ethane; propane; n-butane; isobutane; n-pentane; isopentane; and C_6+, undifferentiated. The non-hydrocarbon gases tested were: nitrogen; oxygen; carbon monoxide; carbon dioxide; and hydrogen sulfide.

C.2 FIELD PROGRAM

Specific hydrocargon and non-hydrocarbon gases were collected at shallow depths at Boring CEG-11, located at the corner of Wilshire and Alvarado. Samples of air were analyzed to provide an ambient base. Approximately 10 ml of gas were analyzed for each sample. All samples were analyzed in the field using an analytical gas chromatograph.

Gas Collection - Air Samples

Samples of air were collected, using a syringe specifically designed for gas chromatographic analysis. The air sample was injected into the gas chromatograph and analyzed in the field.

Gas Collection - Borehole Samples

Most of the natural hydrocarbon gases are heavier than air and must be pumped to the surface to be sampled. One gas, methane, is lighter than air; and another gas, ethane, has approximately the same density as air.

The gas in the borehole was collected through a perforated tube that was inserted into the borehole, and the gas was pumped to the surface by a vacuum pump. The vacuum pump was operated by a portable 120-volt, 1500-watt generator; the generator also supplied power to the gas chromatograph and strip chart recorder. The borehole was temporarily sealed above the level of sampling using an inflated bicycle inner tube. The seal prevented contamination of air or gases from the surface.

The hole was pumped for several minutes; the air and gases wasted before a representative sample was collected for analysis. The purpose for wasting these gases was to purge the borehole of any anomalous accumulations of gas or air due to the drilling operation. After this purge, a sample of gas was collected using the special syringe, and the gas was inserted into the gas chromatograph for analysis in the field.

C.3 DESCRIPTION OF ANALYTICAL GAS CHROMATOGRAPH

The instrument used for quantitative analysis was a Carle thermal conductivity analytical gas chromatograph, Series-S, with a minimum detectability limit of 5×10^{-10} g/ml of propane at 150^{o}C. The unit uses a built-in valve programmer that automatically actuates the correct sequence of internal switching events that are required to perform the complete analysis. Because the instrument is fully automated, errors that might be introduced during the analysis by the operator are eliminated. The gases that were detected were recorded on a strip chart; the written record is called a chromatogram. Chromatograms of the samples and a legend are attached to Appendix C.

Chromatographic System and Operation

A sample of gas is injected into the chromatograph. The injected sample is carried through the instrument by an inert gas (helium) at a constant temperature (70^{o}C), at a constant pressure (60 psi), and at a constant flow rate (30 ml/min). The gas flows through a series of columns, or tubes, that are packed with materials that have specific adsorptive properties; these properties help to separate individual gases from the sample as it flows through the instrument. Each column is designed to separate and identify specific gases. A pressure regulator is used to assure uniform pressure to the column inlet, thereby resulting in a constant rate of flow throughout the analysis.

Depending on the complexity of the gas to be detected, the gas stream may be shunted through a series of valves that direct the gas sample into different columns containing the appropriate adsorptive materials for proper separation.

The column selectively retards the gas components according to their molecular weight and polar characteristics until the components form separate concentrations, or bands, in the carrier (helium) gas. These bands are recorded on a strip chart as a function of time.

The Chromatograph; Methods of interpretation

The record of the gases is printed on a strip chart; the abscissa is time, and the ordinate is millivolts. The chromatogram can be used immediately to qualitatively identify the gases in the sample. Quantitative analyses require additional steps and auxiliary operations. Several different methods can be used to quantify the data; each method has advantages and disadvantages, and not every method is applicable to a particular problem.

A series of gas standards that have different, known percents of the components are allowed to flow through the instrument; the components are recorded on a strip chart. The areas and heights of the peaks are calculated for each different component and for each percent; these data are used to draw a set of graphs of percent of gas vs. peak area of peak height. These graphs provide a basis for comparison to the unknown volumes of gas sampled in the field. The procedure would be as follows: the area corresponding to a gas depicted on the field chromatogram is measured (using, for example, a compensating polar planimeter); that area can be compared to the standard to determine the volume percent of gas in the unknown sample.

To determine weight percent, the data on the field chromatogram must be normalized with respect to the total area of all components. To convert the field data to weight percent, a correction factor corresponding to the gas must be used. The correction factor is necessary because the areas on the graph corresponding to each component are not directly proportional to the percent composition. This is so because different compounds have different responses to the detector depending on the molecular weight of the gas. To determine the correction factor, the relative thermal response per mole of the gas is divided into the molecular weight.

C.4 RESULTS

The chromatogram for Boring CEG-11 is attached. The results of the analyses, reported as parts per million, are given in Table C-1. The reason for selecting "parts per million" to report the results is because this measure provides the most direct conversion to percent by volume; percent by volume is the basis for classifying tunnels in terms of safety (California Administrative Code, Title 8, Article 8, Section 8422). Table C-1 also identified (1) the lower limit of flammability, (2) tunnel classification at the 5 percent and 20 percent lower explosive limit (LEL), and (3) the threshold limit values of selected non-hydrocarbon gases. These columns, abstracted from the more complete Tables C-2 and C-3, are included in Table C-1 for convenience. Table C-2 indicates the limits of flammability for the gases. Table C-3 indicates the threshold limit value (TLV) of selected non-hydrocarbon gases.

Samples Collected in Air

None of the gases detected reached a value that would be considered hazardous (Table C-1).

Hydrocarbon gases in air are not necessarily from natural sources, such as emanations from oil fields. Automobile exhaust is a major source. Exhaust from automobiles includes ethane, propane, isobutane, n-butane, isopentane, n-pentane, C_6+ (California Air Resources Board, Nov. 1980, Hydrocarbon profile of motor vehicle exhaust, 1980, Project HS-11-SHC, 4p). Hydrogen sulfide can come from either natural or industrial sources. There is no need for differentiating the sources for this project. However, they can be differentiated by studying the isotopic composition of the gases.

Methane is likely to have a natural source. Because the gas is lighter than air, it can work its way up through the rocks and soils, eventually reaching the surface. Some of the hydrogen sulfide undoubtedly has a natural source. The gas could be smelled near some of the open boreholes and from the water pumped from the subsurface; the gas is highly soluble in water (Table C-4). During our testing, we noticed that the gas did not flow continuously out of the boreholes; rather, it came out in pulses. Detection of hydrogen sulfide by smell does not necessarily indicate a hazardous condition; the lower limit of detection can be less than 10 ppm (Table C-3), depending on the sensitivity of the individual.

Samples Collected in Boreholes

Gas samples were collected in the boreholes from levels above the uppermost perched water table or within the saturated zone of the uppermost perched water table. A sample from Boring CEG-11 was collected in a cased piezometer; perforations in the casing were within the saturated zone and the gas sampling point was above the line of the water in the cased piezometer. Field conditions did not allow for sampling of gas below the perched water table or at tunnel level or at the point of origin of the gas. Details of the sampling depth and the depth of the water at the time of sampling are given in Table C-1.

Sources of Gas

Geologic exploration for natural gas fields clearly indicates that perched ground water acts to seal the gases below the water (Masters, 1979). The water inhibits the upward migration of the gases. In some field examples discussed in Masters (1979), the gases and water are in the same permeable sanstone, and no impermeable barrier or lithology exists between the water and the gases. Although small amounts of hydrocarbon gases can be adsorbed in the water, the limit of saturation for these gases is extremely low, not exceeding 65 ppm (Table C-4). Among the non-hydrocarbon gases, only carbon dioxide and hydrogen sulfide are significantly soluble (1449 ppm and 3375 ppm, respectively; Table C-4). Because these gases have difficulty entering the water, the gases tend to accumulate at and below the lower level of the perched water table. And, because small amounts of gas are present in the water, not much gas is available to leak out of the water. Thus, only a very small amount of hydrocarbon gases detected in the boreholes came from within the water. The gases can enter the water and bubble up through it if the gases are subjected to a high differential pressure. Gases can also enter the water-saturated zone and bubble up through it if the source of the gases is within the saturated zone.

A review of the lithologic logs of the boreholes along the proposed alignment indicates geologic conditions analogous to those described in Masters (1979). Direct evidence of such conditions along the alignment comes from reports of the drilling operations. The gas "sniffers" detected gas concentrations during the drilling and after the holes had been capped temporarily. The lower level of detection of the "sniffers" was above the lowest limit of sensitivity of the gas chromatograph; the chromatograph recorded levels of gas concentrations lower than that which would trigger the "sniffers." Apparently, the "sniffers" detected the pulse of the gas that was trapped below the water table when the water table was pierced by the drilling. These geologic conditions have significance along the proposed alignment because the natural gases that formed at depth and related to the oil fields are likely to be trapped below the perched water tables. The gases that accumulate along the base of the perched water would likely migrate laterally. Because the gases can migrate laterally below the perched water table, the gases may be present outside the immediate vicinity of known oil fields. The concentrations of gas would depend on the permeability of the rock and soils as well as the concentration and production of gases at the source. Consequently, gases may also be present along the alignment in areas away from the known oil fields. The gases can accumulate in pockets of zones in the soils or bedrock, against faults, or against other impermeable barriers such as igneous dikes. These accumulations can be miles away from known or suspected sources.

The lateral migration of gases from their source in one oil field can cause them to mix with other gases from another oil field. A gas sample from a borehole may not provide a characteristic signature of the gases produced by the nearby oil field due to contamination related to the lateral migration of these gases.

C.5 CONCLUSIONS

In areas between the oil fields, such as Wilshire/Alvarado Station, we may expect to find gas in the subsurface. These areas may be classified as gassy (5% lower explosive limit) and/or potentially gassy.

Because of the lateral migration of gases below the zones of perched water, it is likely that gases have accumulated under pressure in the stratigraphic and structural traps (e.g., faults of igneous dikes along the southern part of the Santa Monica Mountains) at distances away from the immediate areas of known oil fields. Such areas should be approached cautiously with appropriate testing of gases during the driving of the tunnel. In addition, extreme caution should be exercised whenever the driving of the tunnel approaches the area below a perched water zone, and appropriate gas testing should be done.

REFERENCE:

Masters, J.A., 1979, Deep basin gas trap, western Canada: Bull. AAPG, v. 63, no. 2, p. 152-181.

TABLE C - 1
SUMMARY OF DATA FROM GAS CHROMATOGRAMS

BORING NUMBER: 1, 2, 10

GAS	Lower Limit of Flammability* (ppm)	TUNNEL CLASSIFICATION** GASSY 5% LEL (ppm)	TUNNEL CLASSIFICATION** EXTRA HAZARDOUS 20% LEL (ppm)	1 Air (ppm)	1 Cased Piezometer Sampled @ 10' Water Level @ 10' (ppm)	2 Air (ppm)	2 Cased Piezometer Sampled @ 10' Water Level @ 20' (ppm)	10 Air (ppm)	10 Cased Piezometer Sampled @ 15' Water Level @ 20' (ppm)
Methane	50,000	2,500	10,000	-	100	-	200	-	-
Ethane	30,000	1,500	6,000	trace	300	-	300	-	-
Propane	21,200	1,060	4,240	-	trace	-	trace	-	-
n-Butane	18,600	930	3,720	-	trace	-	trace	-	-
Isobutane	18,000	900	3,600	-	trace	-	trace	-	-
n-Pentane	14,000	700	2,800	-	trace	-	trace	-	-
Isopentane	13,200	660	2,640	-	trace	-	trace	-	-
C_6+	‡	-	-	800	3,000	1,600	1,600	400	1,200
Nitrogen	-	-	-	772,000	771,000	770,000	770,000	770,000	770,000
Oxygen	-	-	-	200,000	200,000	200,000	200,000	201,000	200,000
Carbon monoxide	125,000	6,250	25,000	-	trace	-	trace	-	-
Carbon dioxide	-	-	-	27,000	28,000	28,000	28,000	28,000	28,000
Hydrogen sulfide	43,000	2,175	8,700	trace	trace	-	trace	-	trace

BORING NUMBER: 11, 19, 21, 22, 23, 23A

GAS	11 Air (ppm)	11 Cased Piezometer Sampled @ 60' Water Level @ 45' (ppm)	19 Air (ppm)	19 Cased Piezometer Sampled @ 9' Water Level @ 10' (ppm)	21 Air (ppm)	21 Refer to Notes a, b, c (ppm)	22 Air (ppm)	22 Alluvium Open Hole Sampled @ 15' Water Level @ 20' (ppm)	23 Air (ppm)	23 Cased Piezometer Sampled @ 12' Water Level @ 15' (ppm)	23A Air (ppm)	23A Cased Piezometer Sampled @ 13' Water Level @ 15' (ppm)
Methane			-	trace	-	a trace b - c -	trace	trace	-	-	-	trace
Ethane			-	2,000	103	a trace b trace c trace	103	1,500	-	-	153	350
Propane			-	-	-	a trace b - c -	-	-	-	-	trace	-
n-Butane			-	trace	-	a trace b - c -	-	trace	-	trace	trace	-
Isobutane			-	150	-	a trace b - c -	-	trace	-	trace	trace	-
n-Pentane			-	trace	-	a trace b - c -	-	-	-	trace	trace	-
Isopentane			-	trace	-	a trace b - c -	-	-	-	trace	trace	-
C_6+			2,000	4,500	300	a 600 b 2,000 c trace	100	1,000	3,500	3,500	2,500	2,500
Nitrogen			770,000	766,000	770,000	a 770,000 b 770,000 c 770,000	770,000	770,000	769,000	769,000	768,000	768,000
Oxygen			200,000	199,000	201,000	a 201,000 b 200,000 c 201,000	201,000	200,000	200,000	200,000	200,000	200,000
Carbon monoxide			trace	trace	trace	a trace b - c -	-	trace	-	trace	trace	trace
Carbon dioxide			28,000	28,000	28,000	a 28,000 b 27,000 c 28,000	28,000	27,000	27,000	27,000	29,000	27,000
Hydrogen sulfide			trace	trace	trace	a trace b trace c 100	trace	trace	trace	trace	trace	trace

BORING NUMBER: 1, 2, 10

GAS	THRESHOLD LIMIT VALUES OF SELECTED NON-HYDROCARBON GASES ppm §	THRESHOLD LIMIT VALUES OF SELECTED NON-HYDROCARBON GASES ppm §§	1 Air (ppm)	1 Cased Piezometer @ 10' (ppm)	2 Air (ppm)	2 Cased Piezometer @ 16' (ppm)	10 Air (ppm)	10 Cased Piezometer @ 15' Water Level @ 20' (ppm)
Carbon monoxide	50	100, 200; 400; 1,200; 2,000	-	trace	-	trace	-	-
Carbon dioxide	5,000	5,000; 50,000; 90,000	27,000	28,000	28,000	28,000	28,000	28,000
Hydrogen sulfide	10	10; 100; 200	trace	trace	-	trace	-	trace
Oxygen	#	-	200,000	200,000	200,000	200,000	201,000	200,000

BORING NUMBER: 11, 19, 21, 22, 23, 23A

GAS	11 Air (ppm)	11 Hole (ppm)	19 Air (ppm)	19 Hole (ppm)	21 Air (ppm)	21 Hole (ppm)	22 Air (ppm)	22 Hole (ppm)	23 Air (ppm)	23 Hole (ppm)	23A Air (ppm)	23A Hole (ppm)
Carbon monoxide			trace	trace	trace	a trace b - c -	-	trace	-	trace	trace	trace
Carbon dioxide			28,000	28,000	28,000	a 28,000 b 27,000 c 28,000	28,000	27,000	27,000	27,000	29,000	29,000
Hydrogen sulfide			trace	trace	trace	a trace b trace c 100	trace	trace	trace	trace	trace	trace
Oxygen			200,000	199,000	201,000	a 201,000 b 201,000 c 201,000	201,000	200,000	200,000	200,000	200,000	200,000

* See Table F1-2 for levels of selected gases.

** Based on information in Table F1-2; see California Administrative Code, Title 8, Article 8, Appendix 8, Part 9, section 7955a, 55b, 7955c, 7955d. NOTE: samples normalized to indicate ppm. Small errors result from rounding of values.

† Less than 100 ppm.

‡ Not differentiated.

§ Title 8, California Administrative Code, General Industry Safety Order. NOTE: Nitrogen dioxide not tested. TLV requirements; not more than 5 ppm.

§§ See Table F1-3 for details of different levels.

\# Not less than 180,000 ppm.

Notes for Boring 21 Description
a = Alluvium Open Hole Sampled @ 15'
b = Cased 3/4" Piezometer Sampled @ 15'; water @ 16'
c = Cased 2" Piezometer Sampled @ 50'; water @ 55'

TABLE C - 2
LIMITS OF FLAMMABILITY

Gas	Formula	Limits of Flammability in Air			
		Percent by Volume*		Parts per Million	
		Lower	Upper	Lower	Upper
Methane	CH_1	5.00	15.00	50,000	150,000
Ethane	C_2H_6	3.00	12.50	30,000	125,000
Propane	C_3H_8	2.12	9.35	21,200	93,500
n-Butane	C_4H_{10}	1.86	8.41	18,600	84,100
Isobutane	C_4H_{10}	1.80	8.44	18,000	84,400
n-Pentane	C_5H_{12}	1.40	7.80	14,000	78,000
Isopentane	C_5H_{12}	1.32	-	13,200	-
Hexane**	C_6H_{14}	1.18	7.40	11,800	74,000
Heptane (C_7)	-	1.10	6.70	11,000	67,000
Octane (C_8)	-	0.95	-	9,500	-
Nonane (C_9)	-	0.83	-	8,300	-
Decane (C_{10})	-	0.77	5.35	7,700	53,000
Carbon monoxide	CO	12.50	74.20	125,000	742,000
Hydrogen sulfide	H_2S	4.30	28.50	43,000	285,000

*Handbook of Chemistry and Physics, 41st ed., p. 1927-1929.

**Instrument used in analyses combined all hydrocarbon gases, C_6 and greater, including those greater than C_{10}.

TABLE C - 3
THRESHOLD LIMIT VALUE OF SELECTED NON-HYDROCARBON GASES

Gas	Concentration by Volume in Air* Parts per Million	Comments*
Carbon monoxide	100	Threshold limit value (TLV); no adverse effects.
	200	Headache after about 7 hours if resting; about 2 hours of work.
	400	Headache and discomfort, possibility of collapse after 2 hours at rest or 45 minutes of exertion.
	1,200	Palpitation after 30 minutes rest or 10 minutes of exertion.
	2,000	Unconsciousness after 30 minutes rest or 10 minutes of exertion.
Carbon dioxide	5,000	TLV; lung ventilation slightly increased.
	50,000	Breathing is labored.
	90,000	Depression of breathing begins.
Hydrogen sulfide	10	TLV.
	100	Irritation to eyes and throat; headache.
	200	Maximum concentration tolerable for one hour.
	1,000	Immediate unconsciousness.
Sulfur dioxide (not tested)	1 to 5	Can be detected by taste at lower level, by smell at upper level.
	5	TLV; onset or irritation to nose and throat.
	20	Irritation to eyes.
	400	Immediately dangerous to life.

*National Coal Board, 1978, Spoil Heaps and Lagoons, Technical Handbook, N.C.B., London.

TABLE C-4
SOLUBILITY OF GASES IN WATER

Gas	Solubility in Water Parts per Million
Hydrocarbon*	
Methane	24.4 ± 1.0
Ethane	60.4 ± 1.3
Propane	6.24 ± 2.1
n-Butane	61.4 ± 2.6
Isobutane	48.9 ± 2.1
n-Pentane	38.5 ± 2.0
Isopentane	48.9 ± 1.6
(C_6)	9.5 ± 1.3
(C_7)	2.93 ± 0.20
(C_8)	0.66 ± 0.06
Non-Hydrocarbon**	
Nitrogen	17.5
Oxygen	39.3
Carbon monoxide	26.0
Carbon dioxide	1,449
Hydrogen sulfide	3,375

*McAuliffe, C., 1963, Solubility in Water of C_1 - C_9 hydrocarbons: Nature, v. 200, no. 4911, p. 1092-1093.

**Handbook of Chemistry and Physics, 41st ed., p. 1706-1707.

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Gas Chromatography
Boring No. ____

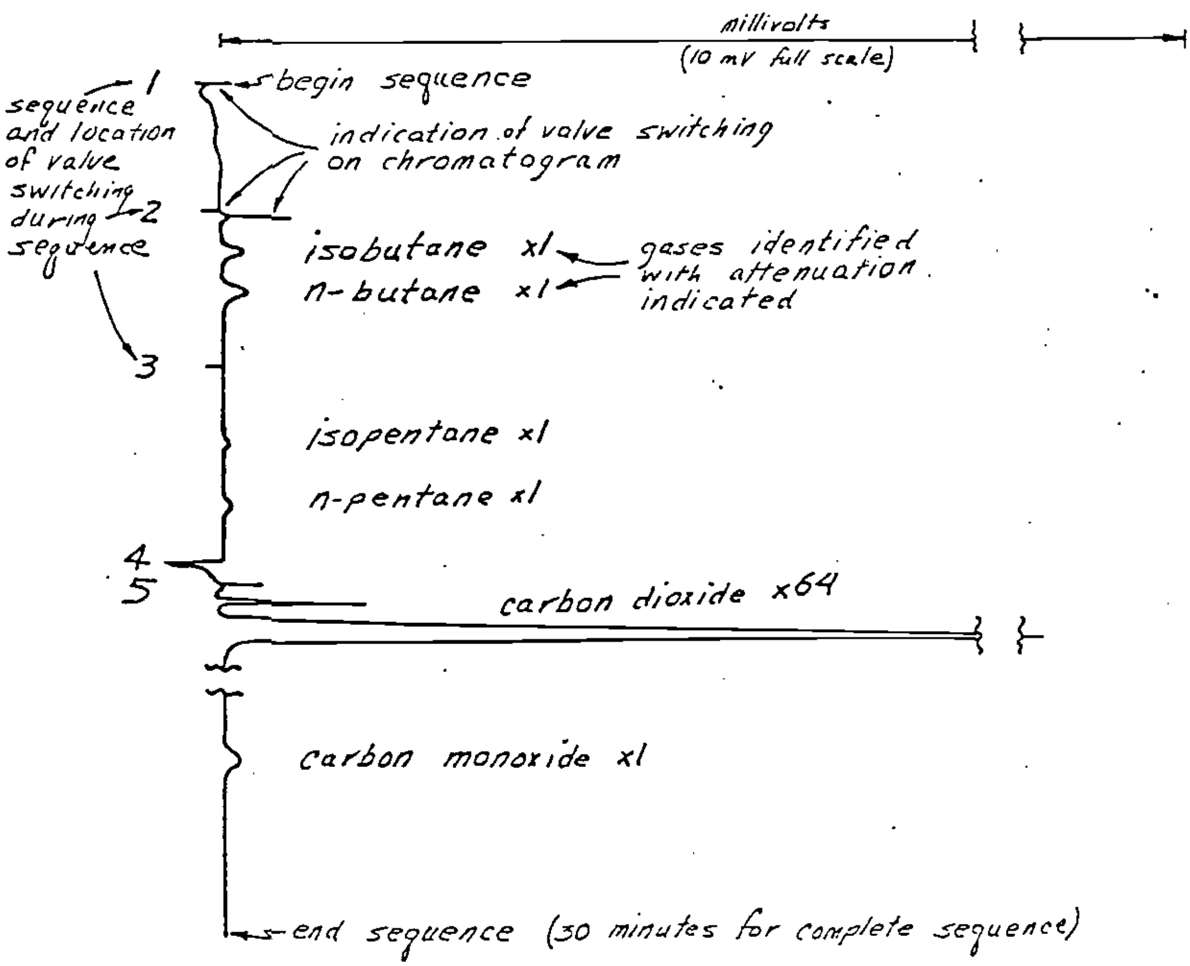
Date Sampled ________ Tested by ________

Depth of Sample ________ Column Temp. ________ Chart Speed ________

Formation ________ Helium ________

Sample Size ________ Attenuation Range ________

LEGEND



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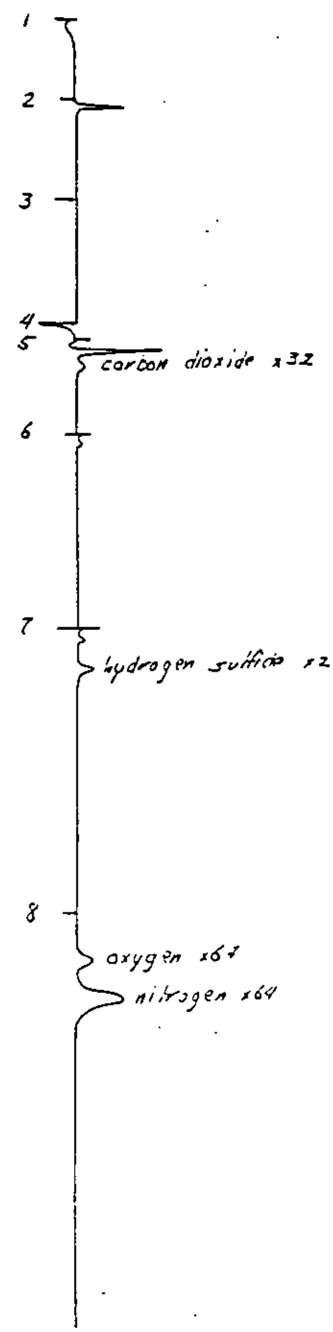
Ryland-Cummings, Inc.

Gas Chromatography
Boring No. 11

Date Sampled 2/21/81 Tested by SLR
Depth of Sample 40 ft water @ 40 ft Column Temp. 70°C Chart Speed 0.5 in/min
Formation Cased piezometer Helium 30 ml/min flow rate @ 60 psig
Sample Size 10 ml Attenuation Range as shown



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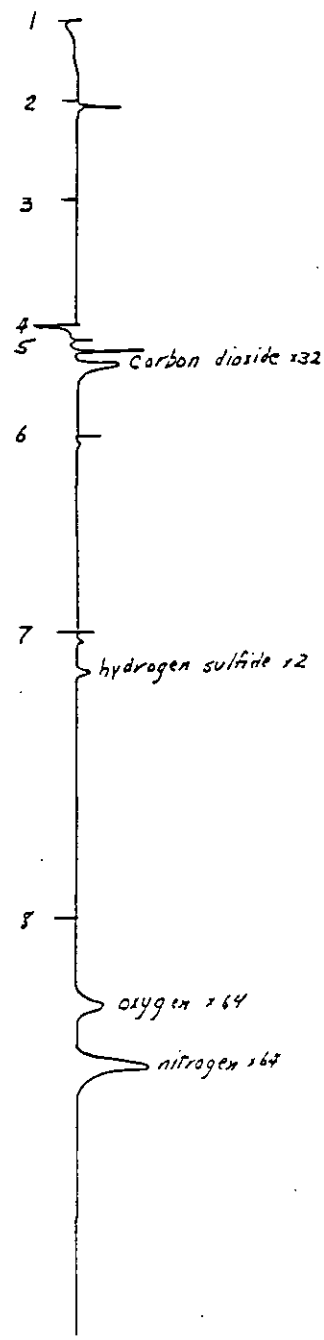
Ryland-Cummings, Inc.

Gas Chromatography
Boring No. 11

Date Sampled 2/2/81 Tested by SLR
Depth of Sample Air Column Temp. 70°C Chart Speed 0.5 in/m
Formation Helium 30 ml/min flow rate @ 60 psig
Sample Size 10 ml Attenuation Range as shown



Append

Water Quality Analy

APPENDIX D - WATER QUALITY ANALYSIS

D.1 INTRODUCTION

Water samples from Borings CEG 9, 10 and 11 were subjected to chemical analysis by Jacobs Laboratories (formerly PJB Laboratories) in Pasadena, California. The sample from Boring CEG 11 was collected at the ground surface on February 1, 1983, the day after drilling was completed. At the time the sample was taken, water was flowing out of the hole at a rate of about 0.75 gpm with 0.5 feet of head. The primary purposes of obtaining and testing the water samples were as follows:

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

Chemical constituents tested by PJB Laboratories were discussed with representatives of the BSGC and included:

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

D.2 ANALYSIS AND RESULTS

In our opinion neither a complicated chemical analysis of interpretation were required for the pupose of the 1981 geotechnical study. Therefore, the normal or standard water chemical analysis tests were performed by PBJ Laboratories, the resuls of which are presented herein. The results of the water quality tests are summarized in the following data summary sheets.

D.3 CONCLUSIONS

Specific water quality parameters were selected from all the chemical tests which are judged to have the most bearing on design or future construction of the project. These parameters and the results of the test are reported in Table D-1, and include interpretations of possible water type and origin of oil field brine sources.

TABLE D-1
SELECTED WATER QUALITY PARAMETERS

Boring No.	PVC Diam. (in.)	Depth Water Sampled (ft)	Date Sampled	pH @ 25° C	Total Dissolved Solids (ppm)	Sulfate SO_4 (ppm)	Boron, B (ppm)	Possible Water Type & Comments
1	2	25.5	02-19-81	7.9	1,258	475	0.98	Na/HCO_3
2	2	11.0	02-19-81	7.7	412	57	0.90	Na/HCO_3
3	2	33.0	02-19-81	7.0	3,722	152	5.0	Na/Cl
4	2	30.0	02-19-81	7.6	5,085	79	7.0	Na/Cl
6	2	19.0	02-20-81	7.5	20,230	27	38.0	Na/Cl - oil field brine?
9	2	105.5	02-23-81	7.7	485	82	0.74	Na/HCO_3
10	1	23.0	02-23-81	7.4	4,461	2,200	2.44	Ca/SO_4
11	2	-*	02-02-81	7.2	19,670	5	37.5	Na/Cl - artesian oil field brine?
12	2	20.0	02-18-81	7.5	6,038	40	14.0	Na/Cl
14	2	24.0	02-18-81	7.9	677	67	0.22	Na/HCO_3
16	1	35.0	02-18-81	7.4	1,139	231	0.14	Na/HCO_3
16	2	40.0	02-18-81	7.5	6,926	25	10.0	Na/Cl
17	2	25.5	02-18-81	7.6	795	87	0.12	Na/HCO_3
19	2	32.0	02-20-81	7.0	15,425	240	10.5	Na/Cl - oil field brine?
21	3/4	19.0	01-07-81	7.6	867	263	0.58	Na/HCO_3
21	2	19.0	01-07-81	7.4	1,448	67	1.74	Na/Cl
22	3/4	16.2	02-16-81	8.0	718	149	0.24	Na/HCO_3
22	2	18.3	02-16-81	7.7	779	124	0.42	Na/HCO_3
23	2	7.5	02-13-81	7.5	589	6	0.22	Na/HCO_3
23A	2	20.0	02-20-81	7.7	863	154	0.38	Na/HCO_3
25	2	109.0	02-13-81	7.6	494	65	0.12	Na/HCO_3
26	1	31.0	02-12-81	7.4	660	161	0.20	Na/HCO_3
27	2	27.5	02-13-81	7.8	725	245	0.32	Na/HCO_3
28A	2	30.0	03-19-81	7.8	805	272	1.16	Na/HCO_3
29	2	84.5	02-25-81	8.0	5,996	2,600	2.6	Na/SO_4
30	2	21.1	03-19-81	7.9	620	202	1.14	Na/HCO_3
31	2	28.7	03-02-81	8.6	511	161	0.58	Na/HCO_3 - Topanga Sandstone & Basalt

* Flowing at rate of 0.75 gpm at time of sampling.

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Water Quality

Jacobs Laboratories

April 6, 1981

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

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

Attention: Buzz Spellman

Report of Chemical Analysis

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

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

Respectfully submitted,

William, R. Ray
Manager, Water Laboratory

asl

Converse Ward Davis Dixon

Lab No. P81-02-123-2

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

Sample labeled: Geology Hole #11 Sample #1 Flow Rate 0.75 gal/min.

Conductivity: 29,070 μ mhos/cm

Turbidity: NTU

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

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	845	42.25
Magnesium, Mg	210	17.27
Sodium, Na	6,500	282.75
Potassium, K	49	1.25
	Total	343.52
Anions determined:		
Bicarbonate, as HCO_3	362	5.93
Chloride, Cl	11,785	332.44
Sulfate, SO_4	5	0.10
Fluoride, F	0.4	0.02
Nitrate, as N	0.3	0.02
	Total	338.51
Carbon dioxide, CO_2, Calc.	33	
Hardness, as $CaCO_3$	2,970	
Silica, SiO_2	58	
Iron, Fe	< 0.01	
Manganese, Mn	0.09	
Boron, B	37.5	
Total Dissolved Minerals, (by addition: $HCO_3 \rightarrow CO_3$)	19,670	

Converse Ward Davis Dixon

Lab No. P81-03-017-4

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

Sample labeled: HOLE 9-2"

Conductivity: 853 μ mhos/cm

Turbidity: - NTU

pH 7.7 @ 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	32	1.60
Magnesium, Mg	7.5	0.62
Sodium, Na	127	5.52
Potassium, K	12	0.31
	Total	8.05
Anions determined:		
Bicarbonate, as HCO_3	202	3.31
Chloride, Cl	101	2.84
Sulfate, SO_4	82	1.71
Fluoride, F	0.7	0.04
Nitrate, as N	0.4	0.02
	Total	7.95
Carbon dioxide, CO_2, Calc.	6	
Hardness, as $CaCO_3$	111	
Silica, SiO_2	20	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.74	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	485	

Converse Ward Davis Dixon

Lab No. P81-03-017-8

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

Sample labeled: HOLE 10-1"

Conductivity: 5,620 μ mhos/cm

Turbidity: NTU

pH 7.4 @ 25°
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	411	20.51
Magnesium, Mg	230	18.92
Sodium, Na	670	29.15
Potassium, K	25	0.64
	Total	69.22
Anions determined:		
Bicarbonate, as HCO_3	303	4.97
Chloride, Cl	731	20.60
Sulfate, SO_4	2,200	45.83
Fluoride, F	0.6	0.03
Nitrate, as N	1.2	0.09
	Total	71.52
Carbon dioxide, CO_2, Calc.	17	
Hardness, as $CaCO_3$	1,970	
Silica, SiO_2	34	
Iron, Fe	< 0.01	
Manganese, Mn	0.02	
Boron, B	2.44	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	4,461	

Appendix E

Geotechnical Laboratory Testing

APPENDIX E: GEOTECHNICAL LABORATORY TESTING

E.1 INTRODUCTION

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

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

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

The laboratory test data from the present investigation are presented in Table E-2, while data from the 1981 geotechnical investigation are presented in Table E-3. The geologic units listed in these tables are described in Section 5.0 of the report.

E.1.1 Data Analysis

The summary of laboratory test results is presented in Table E-1. Figures E-1 through E-5 summarize strength and modulus data for fine-grained alluvium and bedrock at this site and other nearby station sites.

Data from the various tests were organized by test type and geologic unit. Where the number of tests was sufficient to warrant, a statistical evaluation including averaging and computation of standard deviation was performed. The arithmetic average, or mean, was computed for each test type except for the permeability tests. The geometric mean was used for the permeability tests. The geometric mean, m_s, of a population of n samples is defined as:

$$m_s = (a_1 x a_2 \ x \ldots \ x a_n)^{1n}$$

Data obtained for each geological unit were summarized, averaged and evaluated for use in developing recommendations for the design unit. Test results which were considered non-representative due to sample disturbance or other factors were not reported or summarized.

E.2 INDEX AND IDENTIFICATION

E.2.1 Visual Classification

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

E.2.2 Grain Size Distribution

Grain-size distribution tests were performed on representative samples of the geologic units to assist in the soils classification and to correlated test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D-422-63 test method. Combined sieve and hydrometer analyses were performed on selected samples which had a significant percentage of soil particles passing the No. 200 sieve. Results of these analyses are presented in the form of grain-size distribution or gradation curves on Figures E-6 through E-9.

It should be noted that the grain-size distribution tests were performed on samples secured with 2.42- and 2.87- inch ID samplers. Thus, material larger than those dimensions may be present in the natural deposits although not indicated on the gradation curves.

E.2.3 Atterberg Limits

Atterberg Limit Tests were performed on selected soil 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 E-10 and Table E-2.

E.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 Table E-2.

E.2.5 Unit Weight

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

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

E.3 ENGINEERING PROPERTIES: STATIC

E.3.1 Unconfined Compression

Unconfined compression tests were performed on selected samples of cohesive soils and bedrock from the test borings for the purpose of evaluating the undrained, unconfined shear strength of the various fine-grained geologic units. The tests were performed in accordance with the ASTM D-2166 test method. Results of the unconfined compression tests are presented on Tables E-2 and E-3.

E.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:

E.3.2.1 Consolidated Undrained (CU) Tests

- The undisturbed test specimen was trimmed to a length to diameter ratio of approximately 2.0.
- The specimen was then covered with a rubber membrane and placed in the triaxial cell.
- The triaxial cell was filled with water and pressurized, and the specimen was saturated using back-pressure.
- When saturation was complete, the specimen was consolidated at the desired effective confining pressure.
- After consolidation, an axial load was applied at a controlled rate of strain. In the case of the undrained test, flow of water from the specimen was not permitted, and the resulting pore water pressure change was measured.
- The specimen was then sheared to failure or until a 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 in Figures E-11, E-12 and E-13.

E.3.3 Direct Shear

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

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

Progressive direct shear tests were performed on selected undisturbed samples. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. A new normal load was then applied, and the specimen was sheared a second time. This progress was repeated for several different normal loads. Results of the progressive direct shear tests are summarized on Table E-2 and E-3.

E.3.4 Swell

E.3.4.1 Free Swell

Free swell tests were performed on selected undisturbed samples of cohesive, potentially expansive soils. The test procedure entailed placing the undisturbed soil sample in a consolidometer, applying a vertical confining load, and inundating the sample with tap water. The resulting one-dimensional swell of the sample was measured and recorded. Results of these tests are presented on Table E-2.

E.3.4.2 Pressure Swell

Swell tests were performed on selected undisturbed samples of cohesive, potentially expansive soils to evaluate the pressures exerted against non-yielding surfaces. The test procedure entailed placing the undisturbed soil sample in a consolidometer, applying a small vertical seating load, and inundating with tap water. The sample is then monitored for swell. If the sample has a tendency to swell, the vertical load is increased to prevent swell. This procedure is repeated until there is no swell in a 24 hour period. The final load is then recorded as the load to prevent swell. The results of these tests are presented on Table E-2.

E.3.5 Consolidation

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

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

Results of consolidation tests on the undisturbed samples are presented on Figures E-14 through E-18.

E.3.6 Permeability

Permeability tests were performed on undisturbed specimens selected for testing, or in conjunction with the static and cyclic triaxial tests, using the same selected undisturbed samples of soil. Permeability was measured during back-pressure saturation by applying a differential pressure to the ends of the sample and measuring the resulting flow. Results of the tests are tabulated on Tables E-2 and E-3.

E.3.7 Porosity

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

$$e = \frac{1-Vs}{Vs} \text{ where } Vs = \frac{d}{G \times w} \text{ and } n = \frac{e}{1+e}$$

w = unit weight of water
d = unit dry weight of water
G = specific gravity of soil solids.

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

E.4 ENGINEERING PROPERTIES: DYNAMIC

E.4.1 Resonant Column

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

E.4.1.1 <u>Sample Preparation and Handling</u>

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

E.4.1.2 <u>Test Conditions and Parameters</u>

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test pogram, several of the specimens were tested at confining pressures, (σ_{3c}), varying from 15 to 50 psi. Alhough the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressure, shear moulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures E-19 and E-20.

E.4.1.3 <u>Data Reduction</u>

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

E.4.2 <u>Dynamic Triaxial Compression</u>

This test is designed to evaluate the stress-strain properties of the soils under dynamic loading conditions. This test is designed to obtain dynamic stress-strain data at various strain levels. Shear strain data is obtained generally in the range of 10^{-4} to 10^{-2} inch/inch.

E.4.2.1 <u>Sample Preparation and Handling</u>

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.

*ASTM Special Technical Publication 479.

E.4.2.2 Test Conditions and Parameters

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

- 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 0.5 Hz was used for this test program.

E.4.2.3 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 cross sectional 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 unsaturated specimens tested at their in-situ moisture contents. Shear strain values are the strains on a plane located at 45^{o} to the principal stress plane, which has been shown to be the plane of maximum shear strain during triaxial loading.

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

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

Results of the dynamic triaxial tests are presented on Figures E-21 through E-24. 21.

TABLE E-1
SUMMARY OF LABORATORY TEST RESULTS
WILSHIRE / ALVARADO STATION - DESIGN UNIT A170

Unit		Total Density (pcf)	Dry Density (pcf)	Moisture Content (%)	Unconfined Compressive Strength (ksf)	Direct Shear Test Peak Shear Strength ϕ (deg)	c (ksf)	Peak Undrained Strength ϕ (deg)	c (ksf)	Peak Effective Strength ϕ (deg)	c (ksf)	Youngs Modulus E/σ3c (ksf/ksf)	Permeability (cm/sec)
Fine Grained Alluvium	Number of Tests	7	7	7	2	6 D.S.		3 Tx		3 Tx		3 Tx	1
	High	129	105	46	3.67	26°	.9	29°	.7	28°	.23	310	-
	Low	108	74	22	2.09	26°	.6	30.5°	.3	30°	.16	58	-
	Mean	121.7	95.4	29.1	-	26°	.8	30°	.4	29°	.20	115	1.5 E-8
	Std. Deviation	±7.5	±12.1	±9.6	-	-	-	-	-	-	-	-	-
Claystone/ Siltstone	Number of Tests	62	62	62	45	-0-		8 Tx		6 Tx		3 C 6 Tx	2
	High	126	103	46	23.0	-	-	29°	4.2	51°	0	301	4.8 E-7
	Low	105	70	22	.7	-	-	19.5°	0	46.5°	0	83	4.6 E-7
	Mean	116.3	87.7	33.2	9.2	-	-	29°	1.6	50°	0	175	-
	Std. Deviation	±5.1	±8.4	±10.2	±6.6	-	-	-	-	-	-	-	-

TABLE E-2
LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
11-1	C-1	13	Sandy Clayey Siltstone	C	74	50					25	1.35						
	PB-1	21	Clayey Siltstone	C	86	34				1.61								
	C-2	30	Clayey Siltstone	C	80	39					43	0.						
	PB-2	42	Clayey Siltstone	C	74	53				2.33								
	C-3	51	Clayey Siltstone	C	99	27				2.74								
11-2	C-1	9	Silty Clay	A_2	102	24				3.67								
	C-2	20	Silty Clay	A_2	94	27	39	20			20	.63			X	X		
	J-5	26	Silty Clay												X			
	PB-1	41	Clayey Siltstone	C	91	31				7.88								
	C-4	49	Clayey Siltstone	C	96	28				7.68								
11-3	C-2	19	Clayey Siltstone	C	74	46				2.09								
	C-3	29	Silty Claystone	C	85	35					41	1.85						
	PB-1	41	Clayey Siltstone	C	63	55									X	X		X(3)
	C-4	50	Clayey Siltstone	C	77	39				8.41								
	C-5	70	Clayey Siltstone	C	84	36	60	35					.9 (.244)	1.10			X	

TABLE E-2
LABORATORY TEST DATA

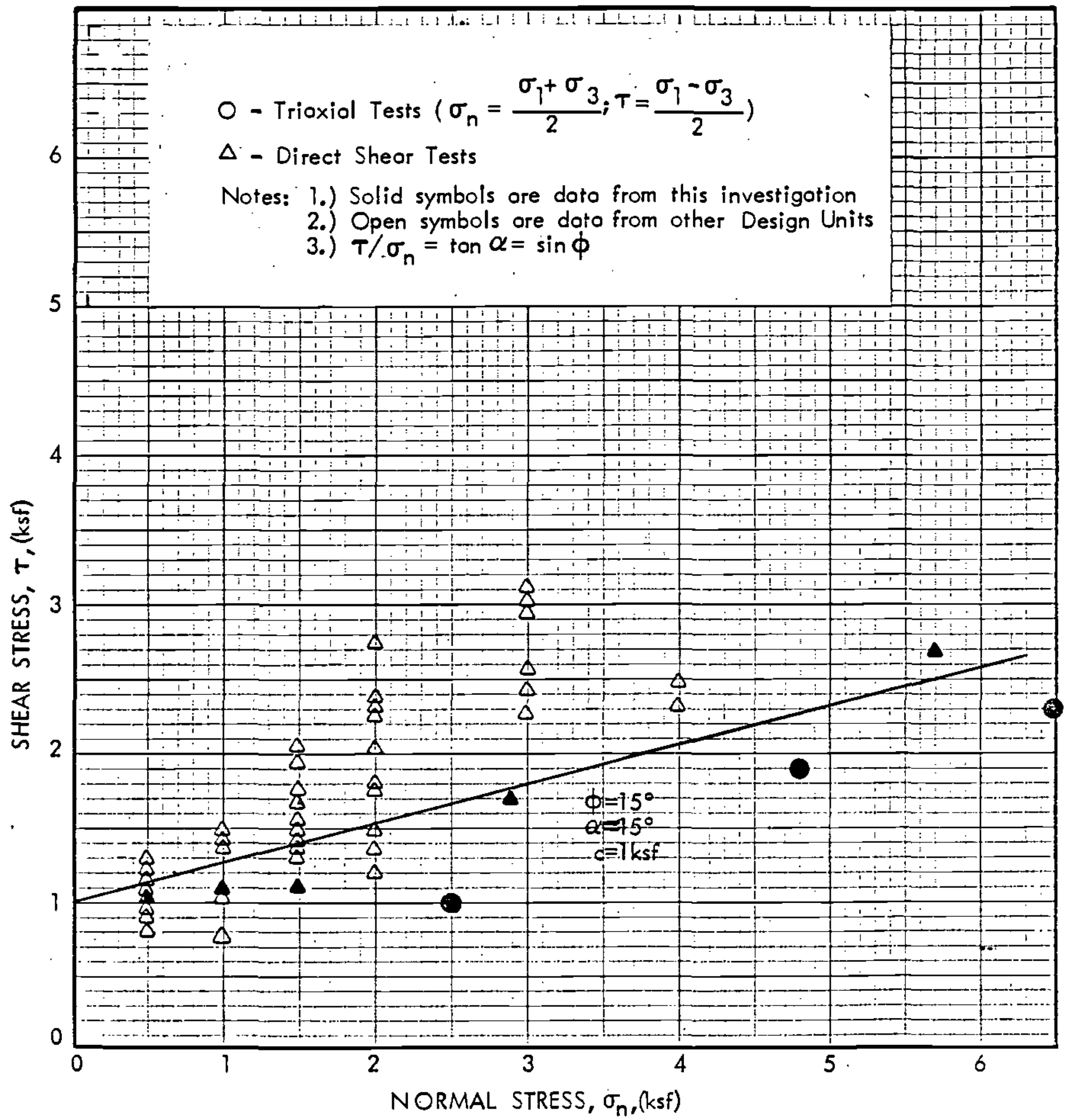
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
11-4	C-1	10	Silty Clay	A_2	103	23	40	22	3.4E-7(10)						X	X		X (2)
	C-2	20	Silty Clay	A_2	85	39	61	38							X	X		X
	PB-1	30	Silty Claystone	C	72	46				1.25								
	C-3	40	Clayey Siltstone	C	70	50				16.2								
	PB-2	51	Clayey Siltstone	C	84	37				8.81								
	C-4	60	Clayey Siltstone	C	71	49	70	27					.3 (.244)	1.50			X	
	PB-3	73	Clayey Siltstone	C	80	41				13.8								
11-5	C-3	40	Silty Claystone	C	95	23											X	
	PB-2	51	Silty Claystone	C	81	41				9.07								
	C-4	60	Silty Claystone	C	89	32				23.0								
	PB-3	73	Silty Claystone	C	88	34				9.13								
11-6	C-2	19	Silty Claystone	C	92	25				20.9								
	C-3	39	Silty Claystone	C	82	33				19.9								
	PB-2	51	Silty Claystone	C	74	46				13.6								
	C-4	59	Silty Claystone	C	78	40							.5 (.244)	1.10	X	X	X	

TABLE E-3
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)	Coefficient of Consolidation Cc (in/in per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear φ, deg	c, ksf	One-Dimensional % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
10	S1	32	Claystone, massive	C	84	38						7.0											
	S2	50	Claystone, massive	C	90	31						96.8											
	S3	60	Silty claystone	C	89	32						137									X		
	S6	76	Silty claystone, folded	C	90	31						135											
	S7	89	Claystone, massive	C	78	40						7.1											
	S8	93	Silty claystone, folded	C	93	29						83.6											
	S9	124	Claystone, massive	C	86	34						58.7											
	S10	139	Claystone, massive	C	89	31						112											
	S11	154	Claystone, massive	C	93	29						126											
	S12	156	Claystone, massive	C	92	29						79.9											
	S13	164	Claystone, massive	C	82	37						11.1											
	S14	170	Claystone, massive	C	94	29						99.1											
	S16	188	Silty claystone, folded, micaceous	C	98	26						124											
11	C1	20	Silty clay	A_4			47	25															
	S1	26	Silty claystone, bedded & folded	C	94	29	52	21	94	82	74												Q
	S2	42	Siltstone	C	95	32							4.E8-7			31.8							
	S2	42	Varved ML/CL/SP	C	90	32	44	13	100	93	78												cu
	S2	43	Varved ML/CL/SP	C	100	24																	cu
	S2	44	Varved ML/CL/SP	C	94	28																	cu
	S3	60	Fine sandy siltstone	C	89	30	42	16	100	95	68		4.6E-7			42.2					X		
	S3	60	Fine sandy siltstone	C	98	26						122											
	S3	61	Fine sandy siltstone	C	103	22																X	
	S4	77	Fine sandy siltstone	C	87	33	50	18	100	91	69												Q
	S5	88	Siltstone	C	93	28						52.6											
	S7	108	Siltstone, folded & bedded	C	90	30						8.9											
	S7	109	Sandy siltstone	C	98	20																	
	S8	124	Siltstone	C	93	29	43	13															Q

TABLE E-3
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)	Coefficient of Consolidation Cc (in/in per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear ø, deg	c, ksf	One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
11	S9	141	Siltstone, thin bedded sand	C	84	36						17.1											
	S10	158	Siltstone, thin bedded sand	C	95	29						59.5											
	S13	199	Siltstone, thin bedded sandstone	C	98	26						60.1											
12	C1	20	Brown silty clay	A_4	105	23							1.5E-8		2.71	37.5	5	1.0					
	C1	20	Brown silty clay	A_4	104	22																	
	S2	63	Claystone with folded siltstone	C	87	32						83.8											
	S3	77	Siltstone	C	86	33						5.0											
	S4	82	Claystone, lenses of siltstone	C	88	32						69.4											
	S5	99	Siltstone, folded & bedded	C	89	31						53.9											
	S6	116	Very fine sandstone & siltstone varves	C	92	29						23.6											
	S7	119	Very fine sandy siltstone	C	79	38						12.1											
	S8	136	Siltstone	C	83	36						67.8											
	S9	150	Claystone, folded & bedded	C	88	31						22.7											
	S10	157	Siltstone & claystone, folded & bedded	C	97	25						66.6											
	S13	187	Siltstone, layered with sandstone	C	94	27						109											



Approved for publication by

SUMMARY OF TOTAL STRENGTH DATA - FINE GRAINED ALLUVIUM

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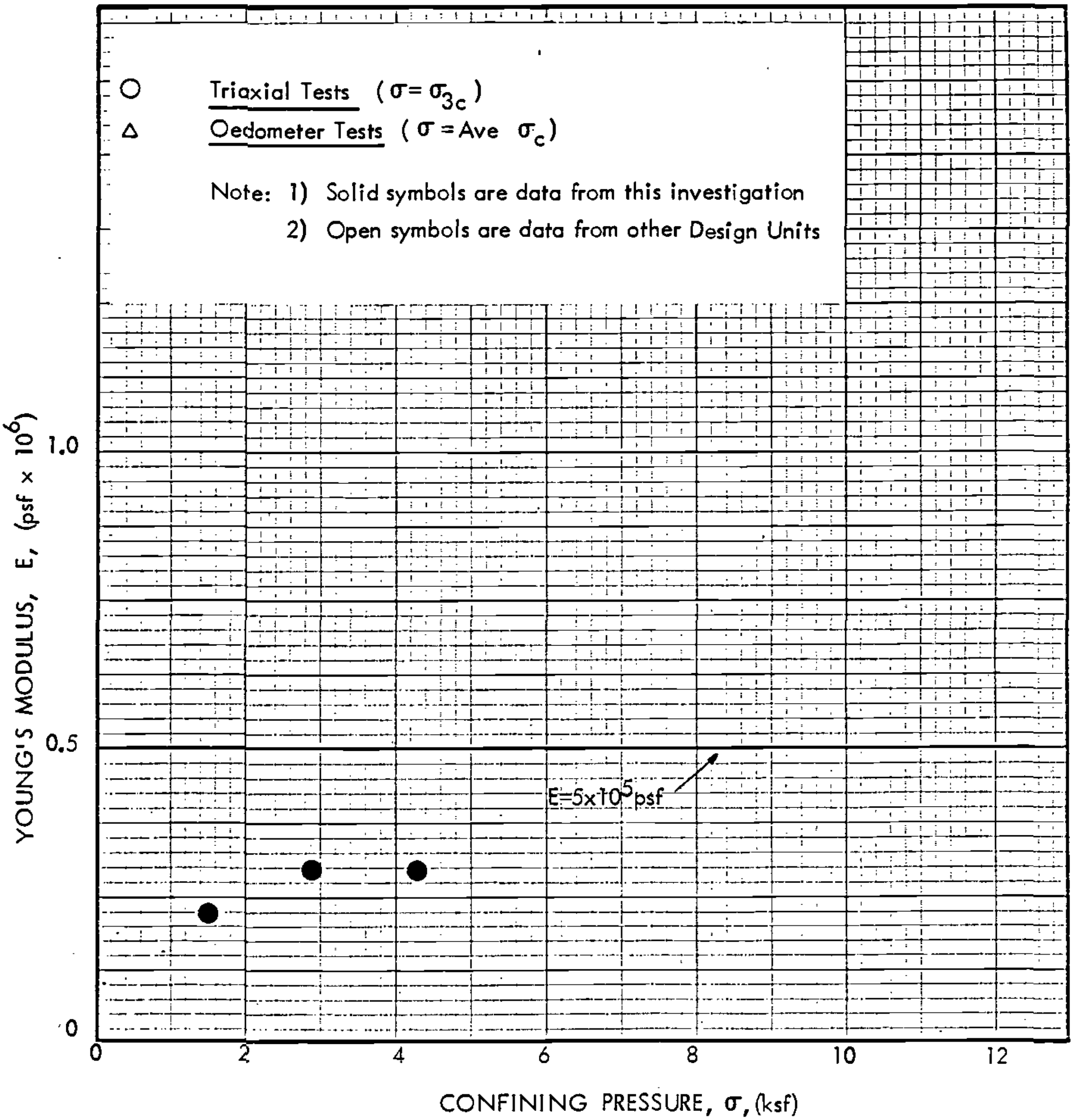
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. E-1



SUMMARY OF MODULUS DATA - FINE GRAINED ALLUVIUM

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Figure No. E-2

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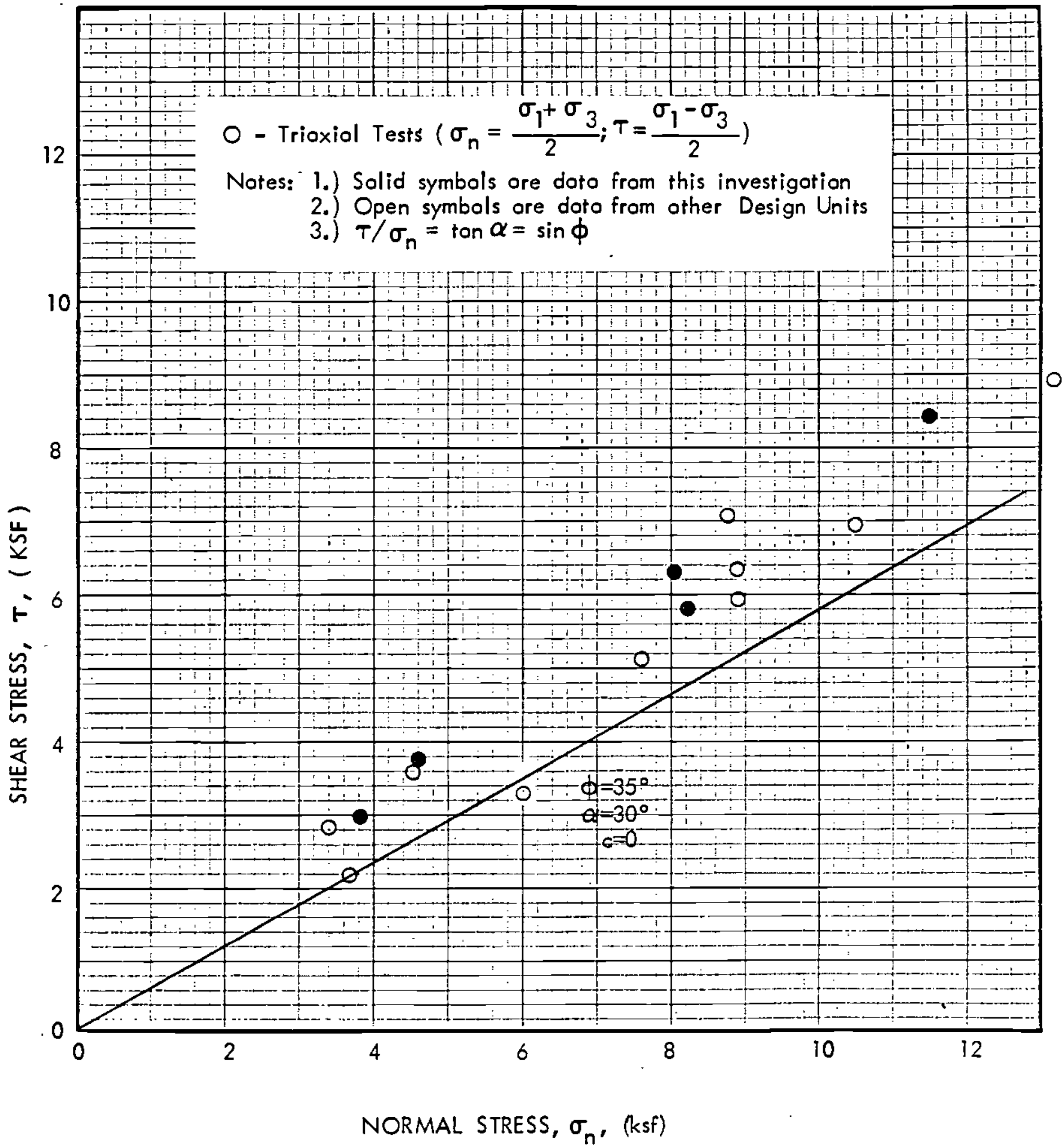

SUMMARY OF EFFECTIVE STRENGTH DATA - BEDROCK

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Figure No. E-3

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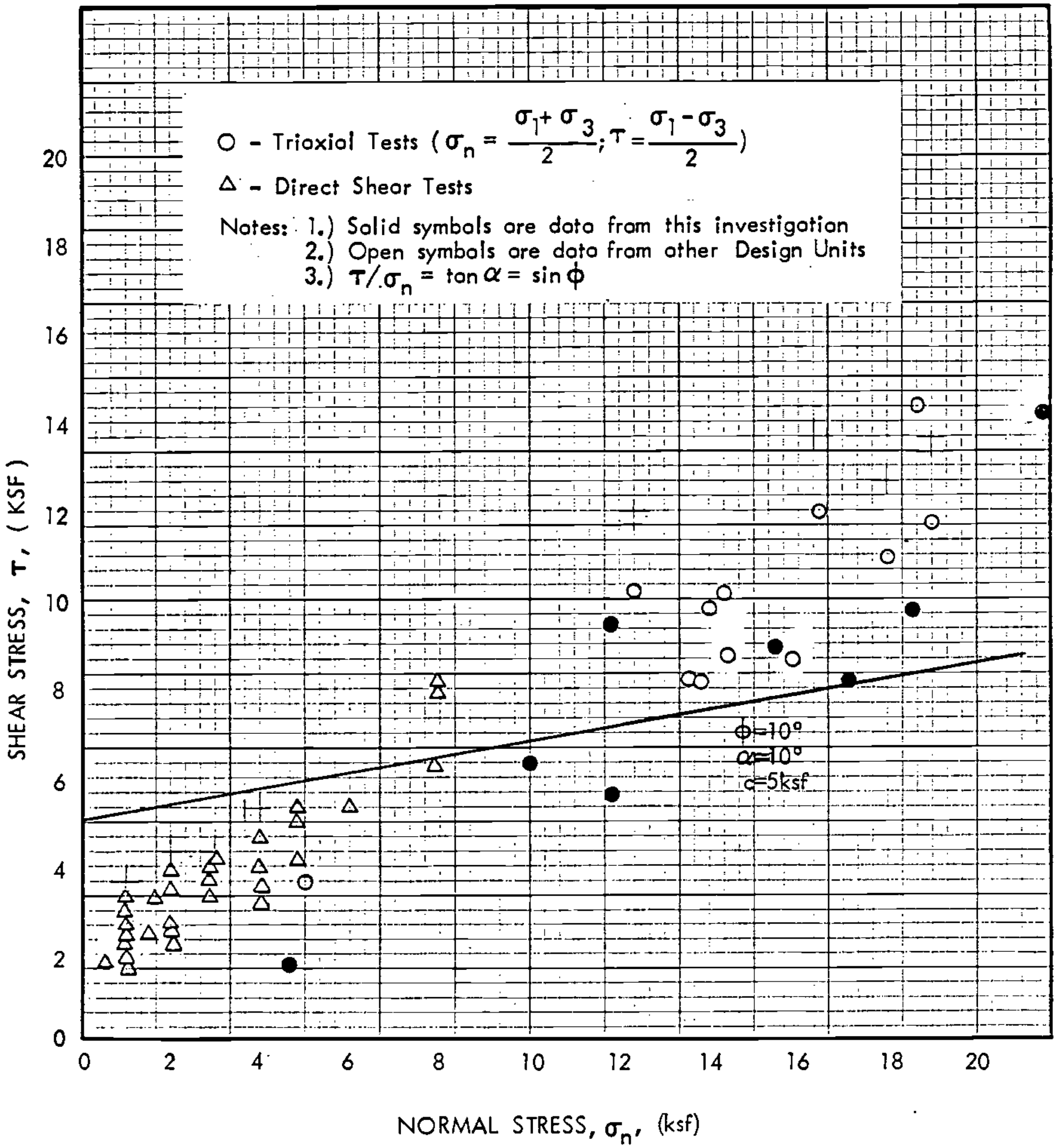

SUMMARY OF TOTAL STRENGTH DATA - BEDROCK

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METRO RAIL PROJECT

Project No. 83-1101

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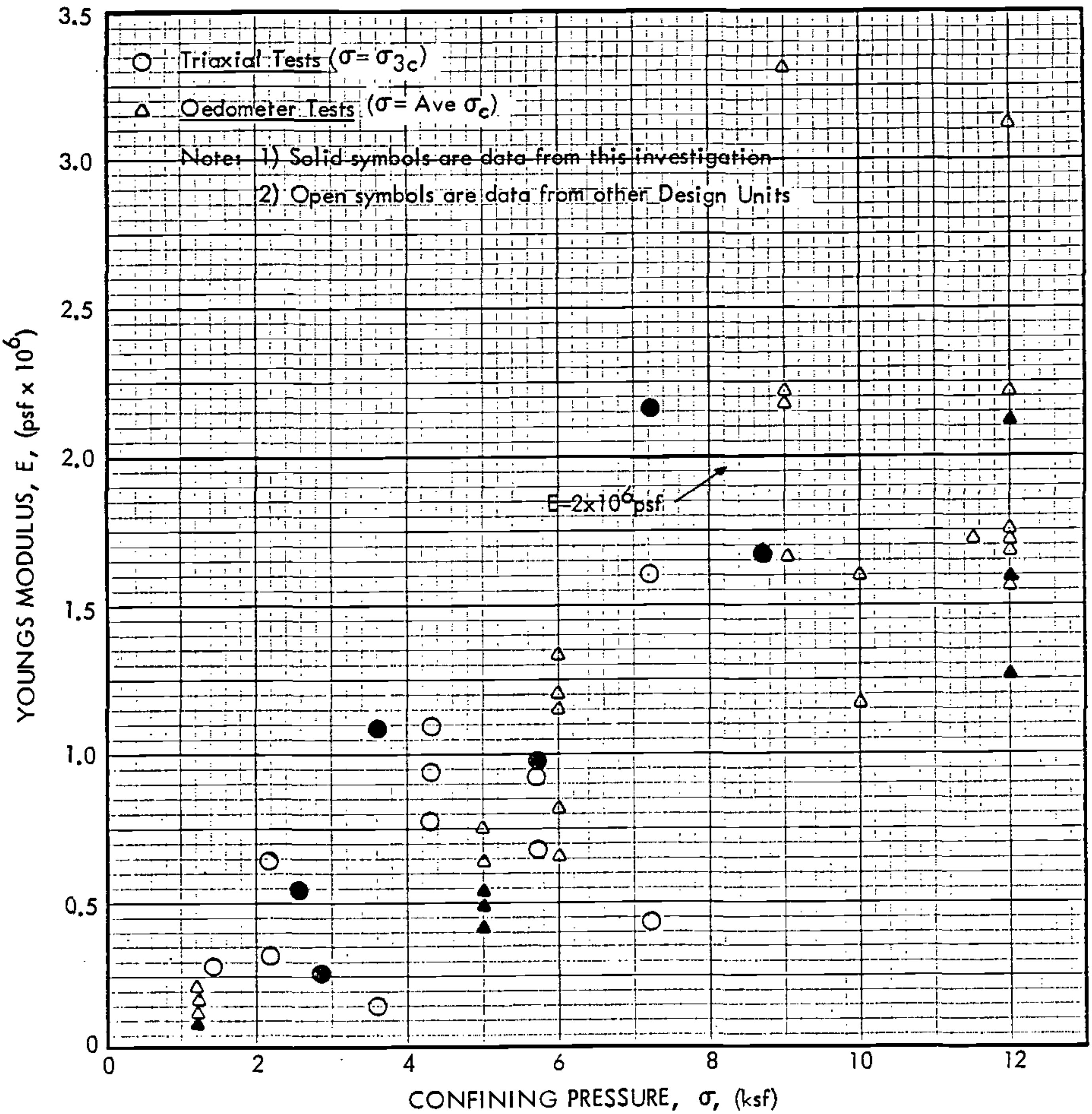
Figure No. E-4

Approved for publication by



SUMMARY OF MODULUS DATA - BEDROCK

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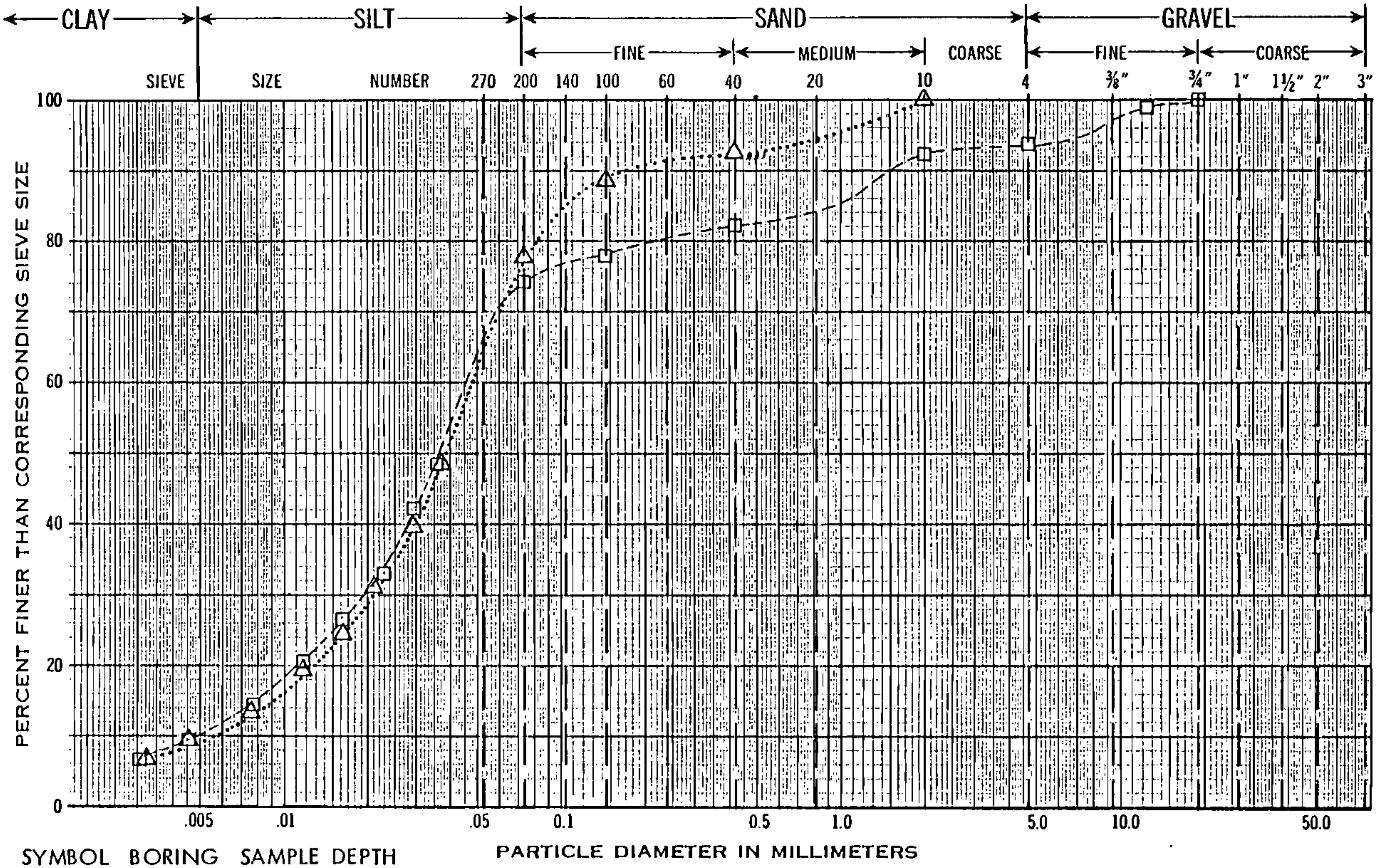
Figure No.
E-5

Approved for publication KDH by 12/1/83

FORM NO. Approved for publication ________ by ________ GRAIN-SIZE ANALYSIS OF SOILS A... DESIGNATION: D422



SYMBOL	BORING	SAMPLE	DEPTH
---□---	11	S-1	27'
·····△·····	11	S-2	42'

GRAIN-SIZE DISTRIBUTION CHART

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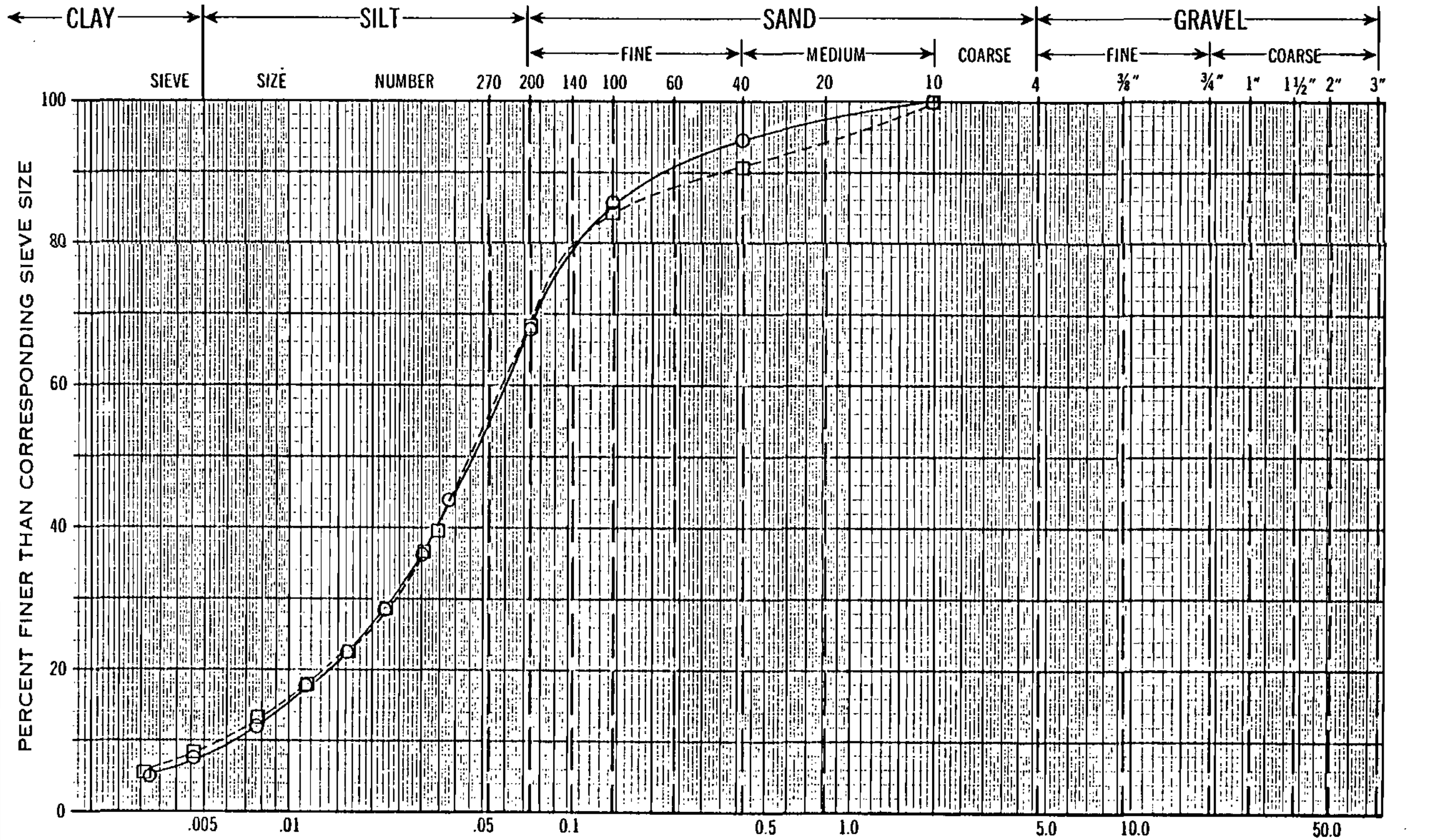
Figure No. E - 6

FORM NO. 7 Approved for publication ______ by ______ GRAIN-SIZE ANALYSIS OF SOILS A... SIGNATION: D422



SYMBOL	BORING	SAMPLE	DEPTH
—○—	11	S-3	60'
---□---	11	S-4	78'

GRAIN-SIZE DISTRIBUTION CHART

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Project No. 83-1101

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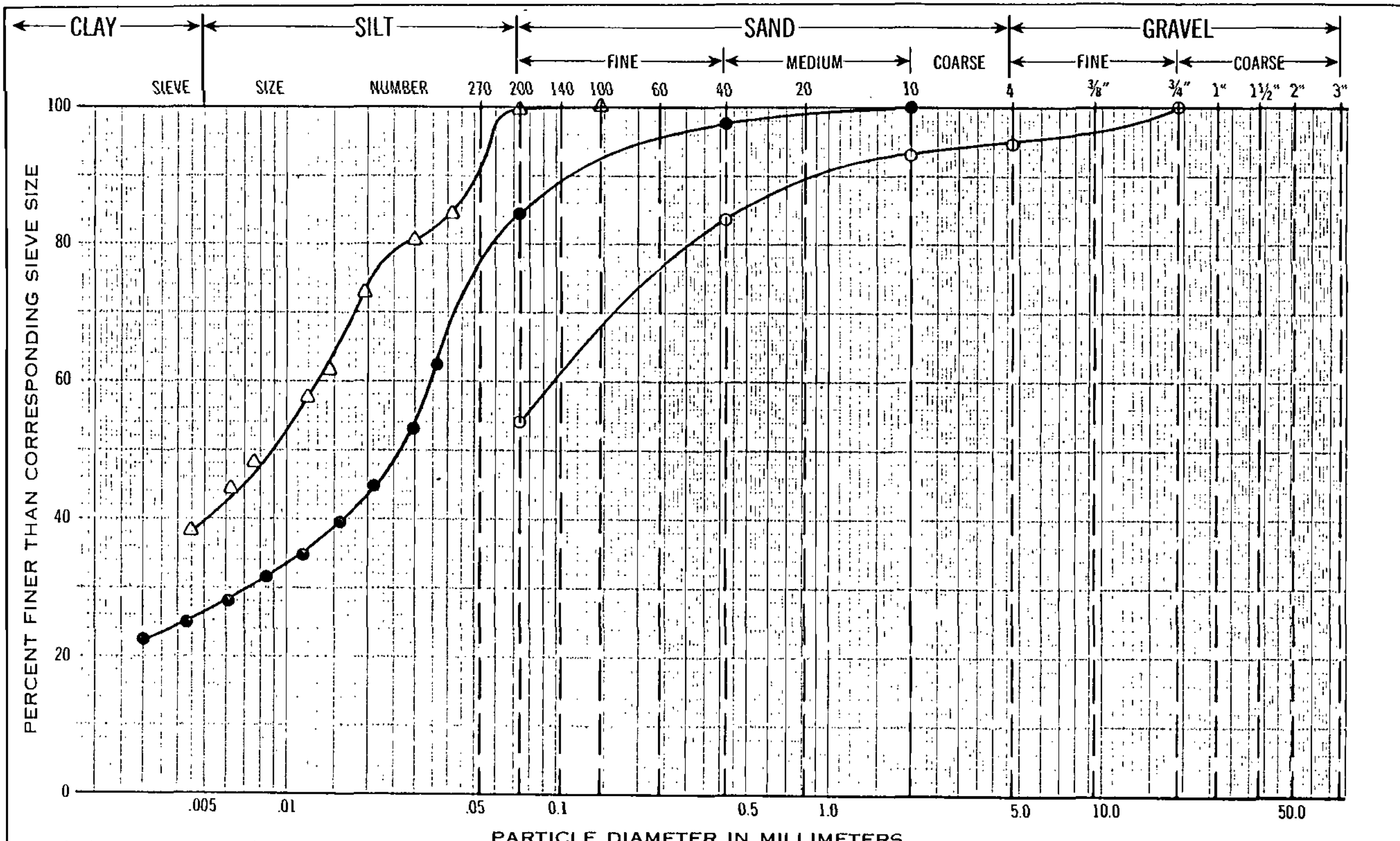
Figure No. E - 7

FORM NO. Approved for publication ______ by ______



Symbol	Boring	Sample	Depth
●	11-2	C-2	20'
○	11-2	J-5	27'
△	11-3	PB-1	40'

GRAIN-SIZE DISTRIBUTION CHART

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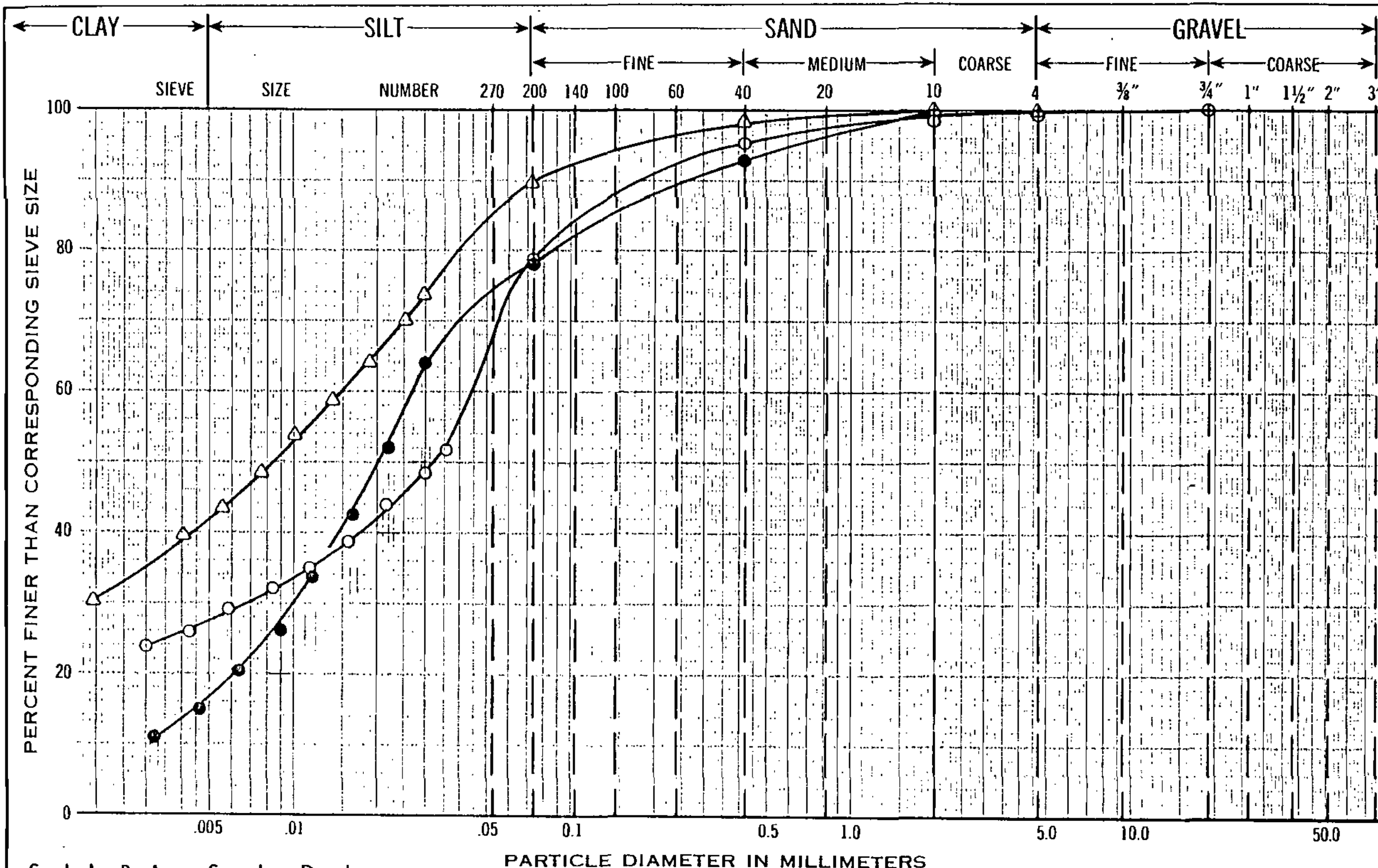
Figure No. E - 8

FORM NO. D Approved for publication ______ by ______



Symbol	Boring	Sample	Depth
○	11-4	C-1	10'
△	11-4	C-2	20'
●	11-6	C-4	59'

GRAIN-SIZE DISTRIBUTION CHART

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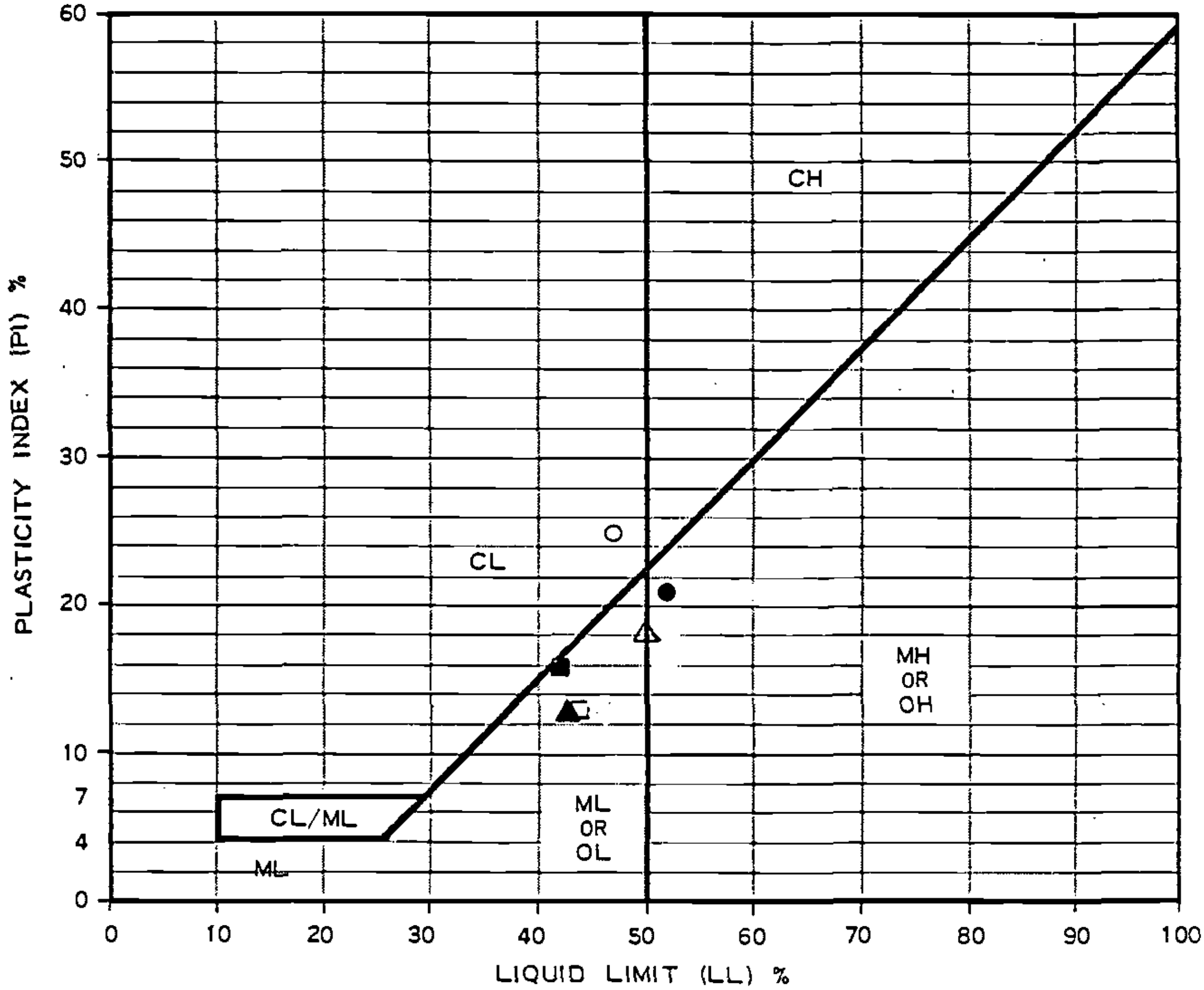
Figure No. E - 9



PLASTICITY CHART

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing No. 200 Seive
○	BH 11, 19' (CL)	47	32	25	-
●	BH 11, 26' (MH)	52	31	21	74
□	BH 11, 41' (ML)	44	31	13	78
■	BH 11, 61' (ML)	42	26	16	68
△	BH 11, 77' (ML/MH)	50	32	18	69
▲	BH 11, 125' (ML)	43	30	13	-

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Project No.
83-1101

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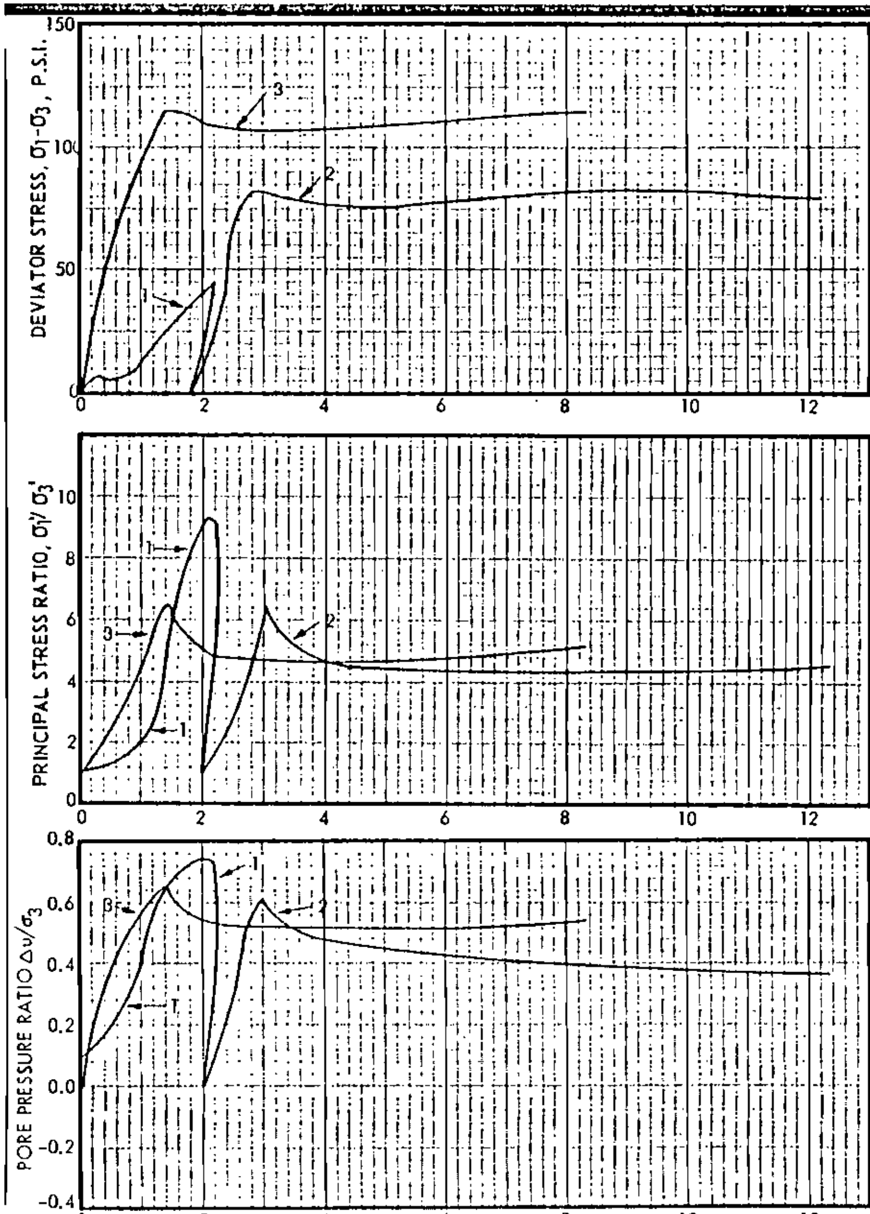
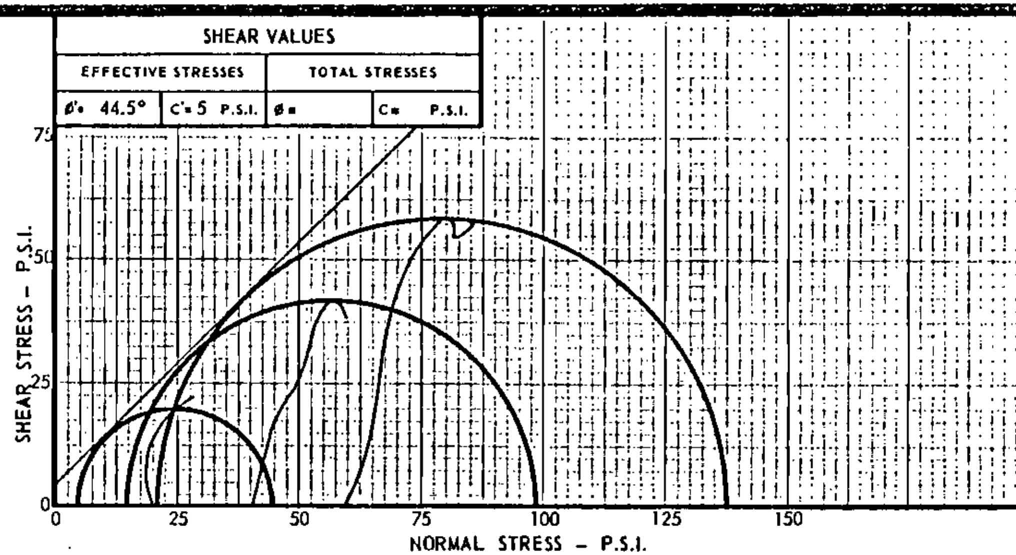
Figure No.
E - 10

Approved for publication 12/21/83 by [illegible]





SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
$\phi' = 44.5°$	C' = 5 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	
PB-1	B-11-3	39	CL	7.06	2.86	64	51	PITCHER

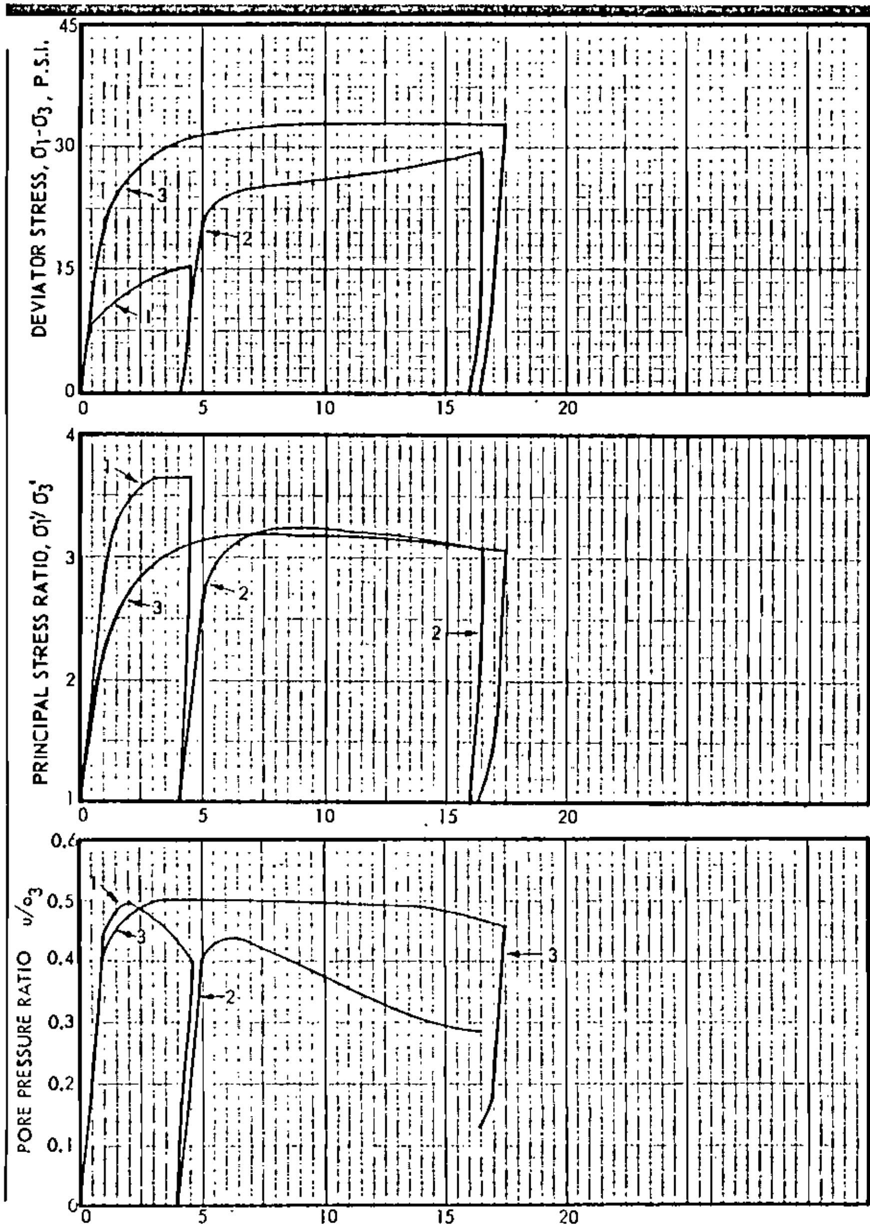
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	PB-1	20	--	4.9	46.7	40	
2	PB-1	40	82.6	15.2	97.8	40	
3	PB-1	60	116.2	21.3	137.6	40	

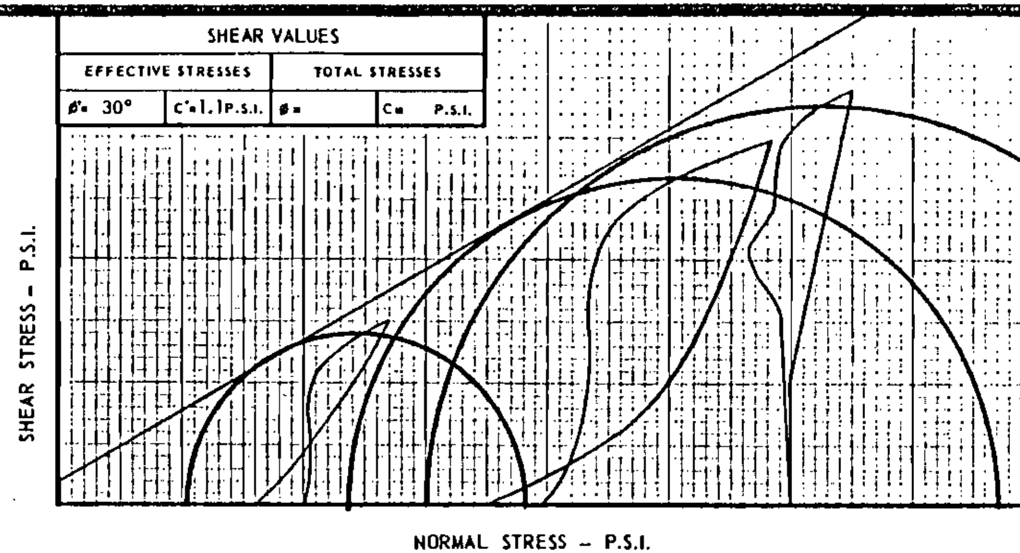
TRIAXIAL COMPRESSION TEST

DESIGN UNIT A170
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale As Shown
Date 10-11-85
Prepared by TPZ
Checked by JDH
Approved by JAD
Project No. 83-1101
Figure No. E – 11





SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 30°	C'=1.1 P.S.I.	ϕ=	C= P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSIFICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C1	B11-4	10	CL	4.97	2.42	103	23	UNDISTURBED
C1	B11-4	10	CL	4.97	2.42	103	23	UNDISTURBED
C2	B11-4	19	CL	5.01	2.42	85	39	UNDISTURBED

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C1	10	15.4	5.3	19.1	80	CONSOLIDATED UNDRAINED
2	C1	20	29.8	11.9	38.6	80	WITH PORE PRESSURE
3	C2	30	33.1	15.0	47.8	80	MEASUREMENTS

TRIAXIAL COMPRESSION TEST

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METRO RAIL PROJECT

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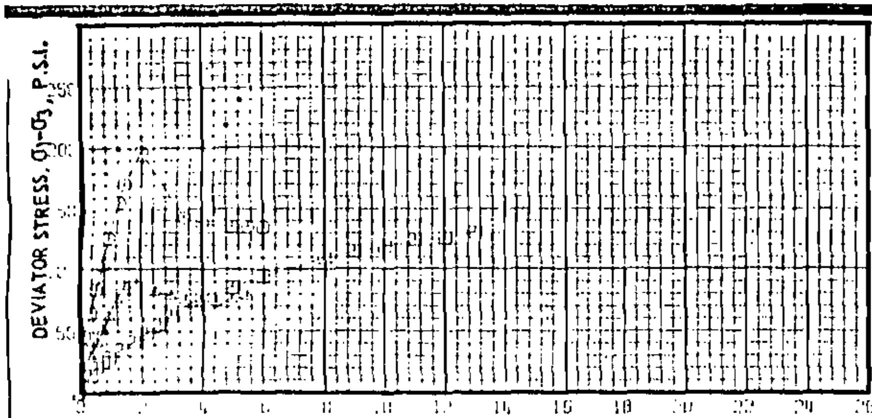
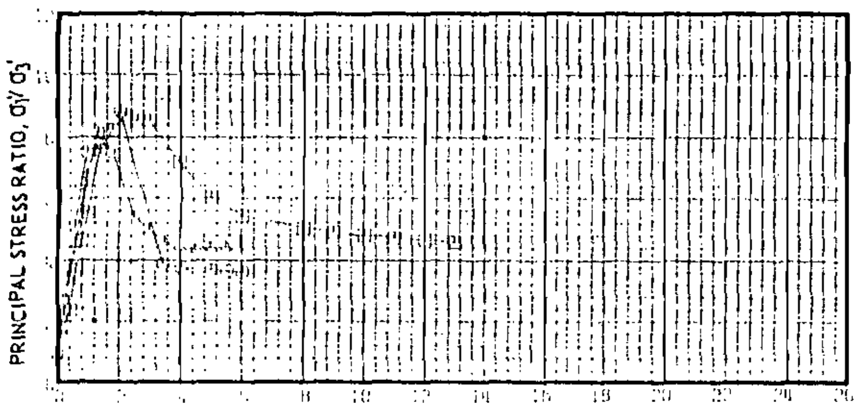
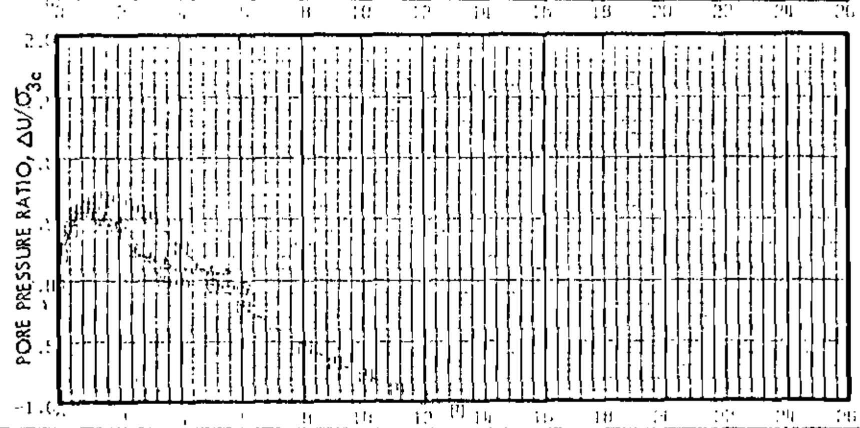
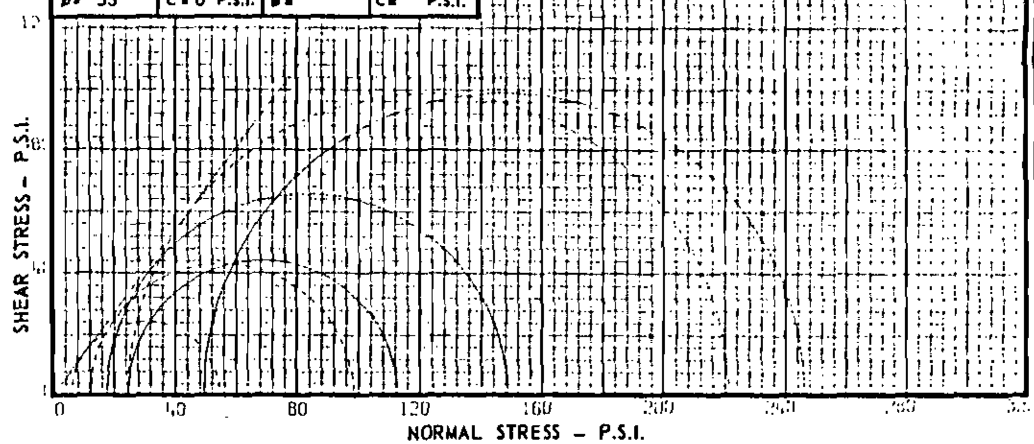
Scale	As Shown	Project No	83-1101
Date	10-11-85		
Prepared by	TPZ	Figure No	E - 12
Checked by	JDH		
Approved by	JAD		







SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ' = 53°	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	
S2	11	[illegible]	ML/CL/SP	4.99	2.42	93.80	[illegible]	UNDISTURBED
S2	11	[illegible]	ML/CL/SP	4.97	2.42	89.60	31.60	UNDISTURBED
S2	11	[illegible]	ML/CL/SP	4.98	2.42	99.60	[illegible]	UNDISTURBED

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3': EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	BACK PRESSURE (P.S.I.)	TYPE OF TEST
	S2	18.0	131.5	5.3	[illegible]	87.0	CONSOLIDATED UNDRAINED
	S2	50.0	196.9	25.0	[illegible]	75.0	CONSOLIDATED UNDRAINED
	S2	25.0	86.4	12.5	99.0	80.0	CONSOLIDATED UNDRAINED

TRIAXIAL COMPRESSION TEST

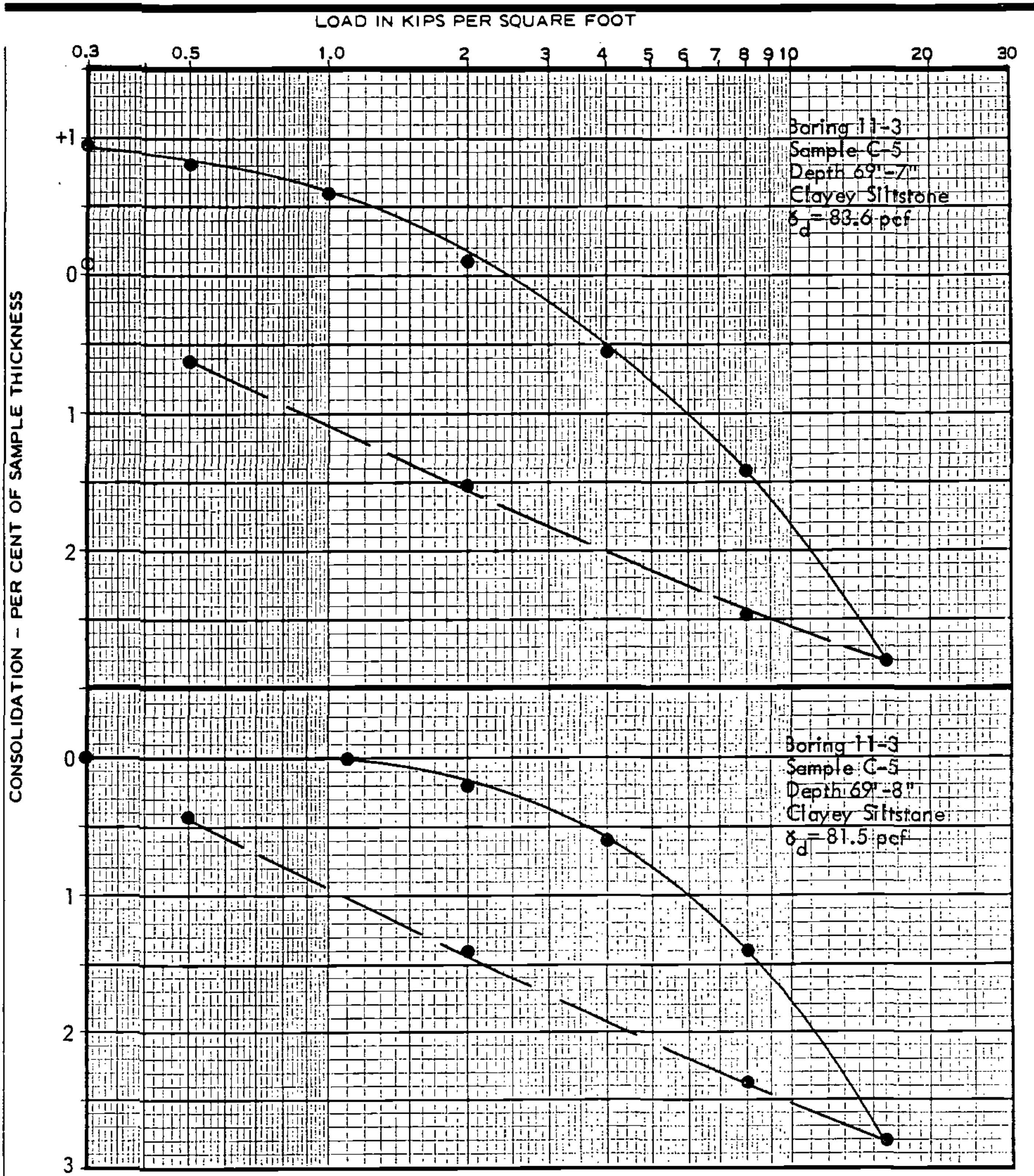
DESIGN UNIT A170
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Consultants

Scale: As Shown
Date: 10-11-83
Prepared by: TPZ
Checked by: JSL
Approved by: JSL
Project No. 83-1101
Figure No. E-13

FORM NO. D60/77

11-945



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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METRO RAIL PROJECT

Project No.
83-1101

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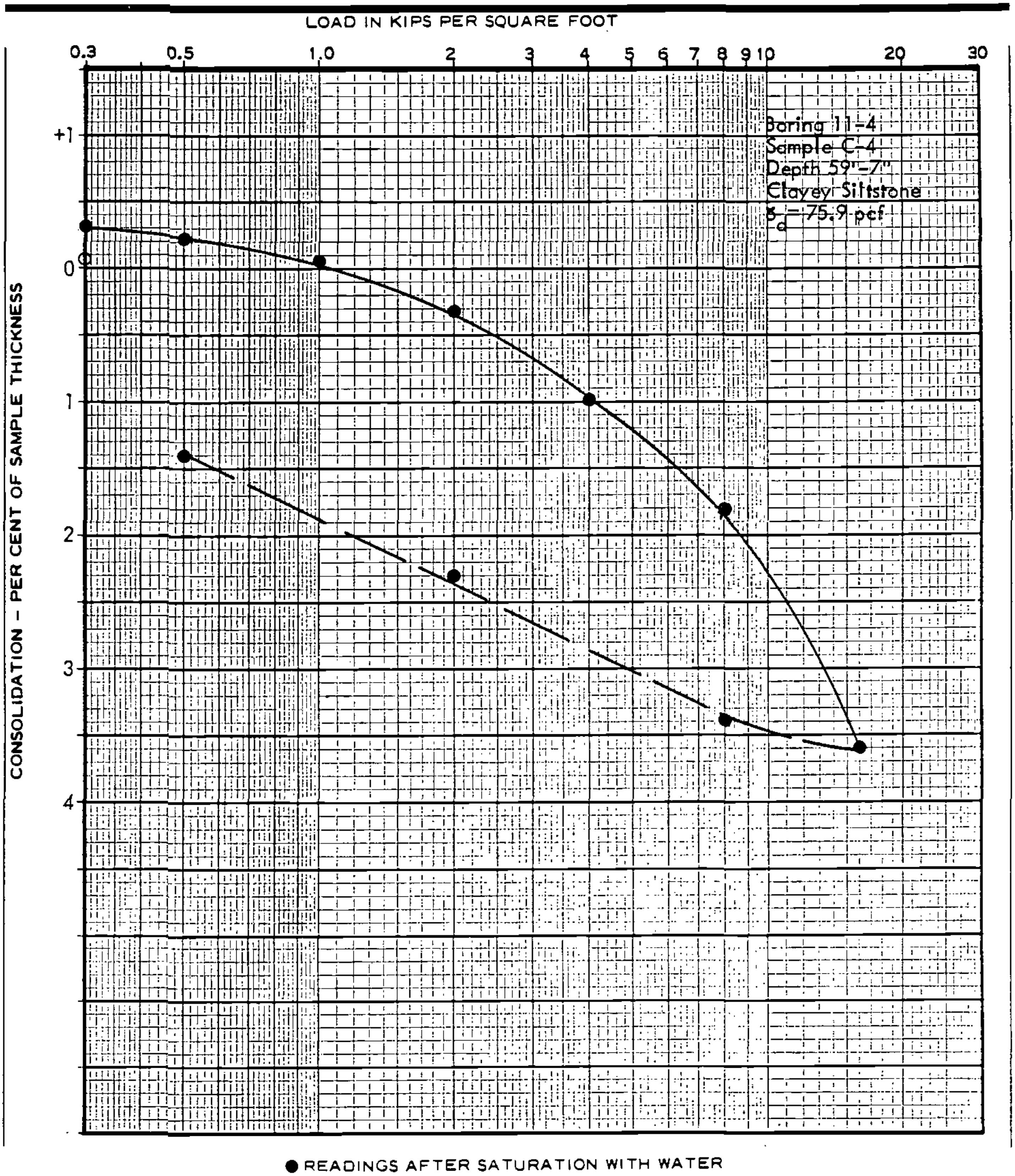
Figure No.
E-14

Approved for publication ______ by ______



CONSOLIDATION TESTS

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Project No.
83-1101

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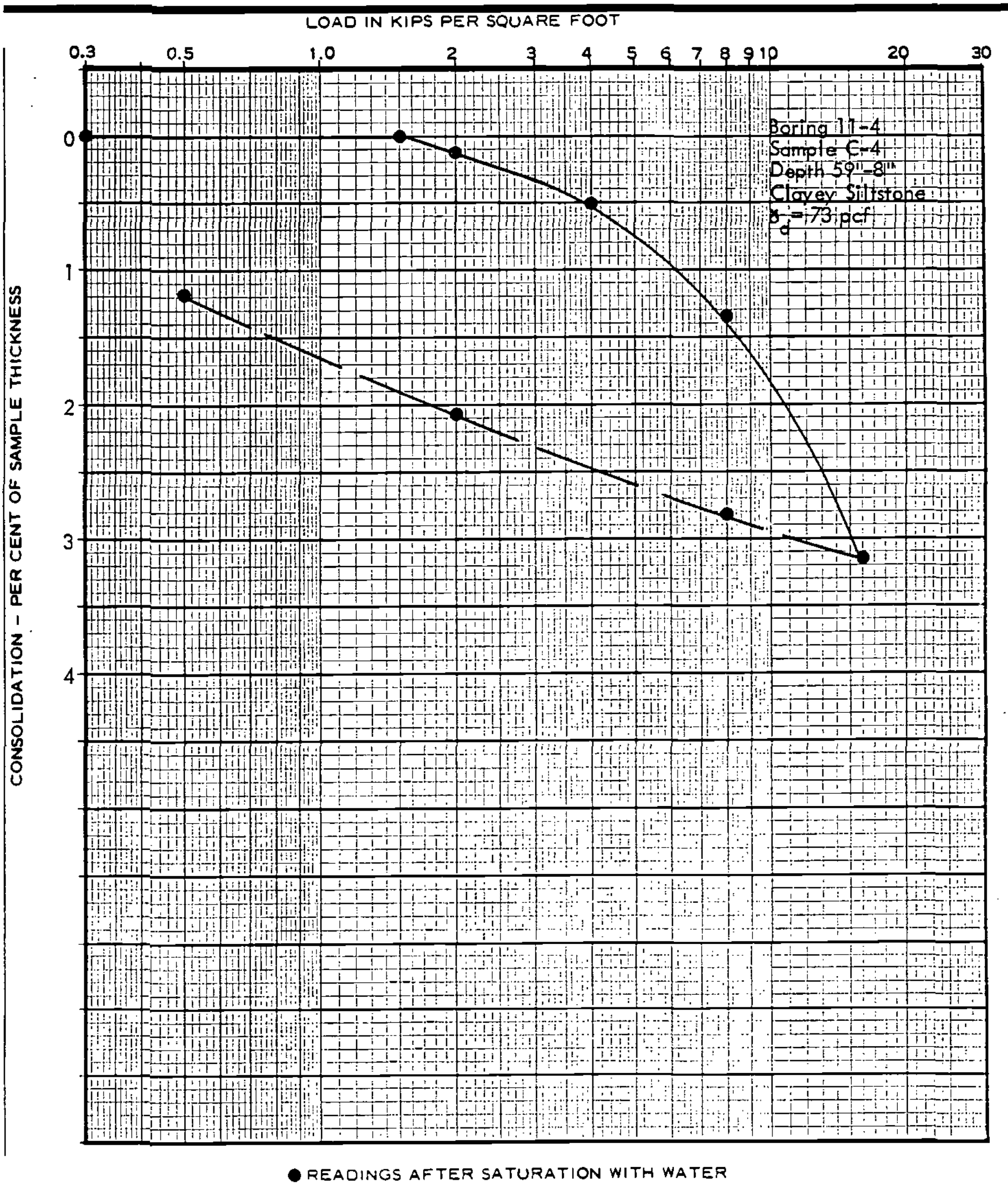
Figure No.
E-15

Approved for publication ________ by ________



CONSOLIDATION TESTS

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METRO RAIL PROJECT

Project No.
83-1101

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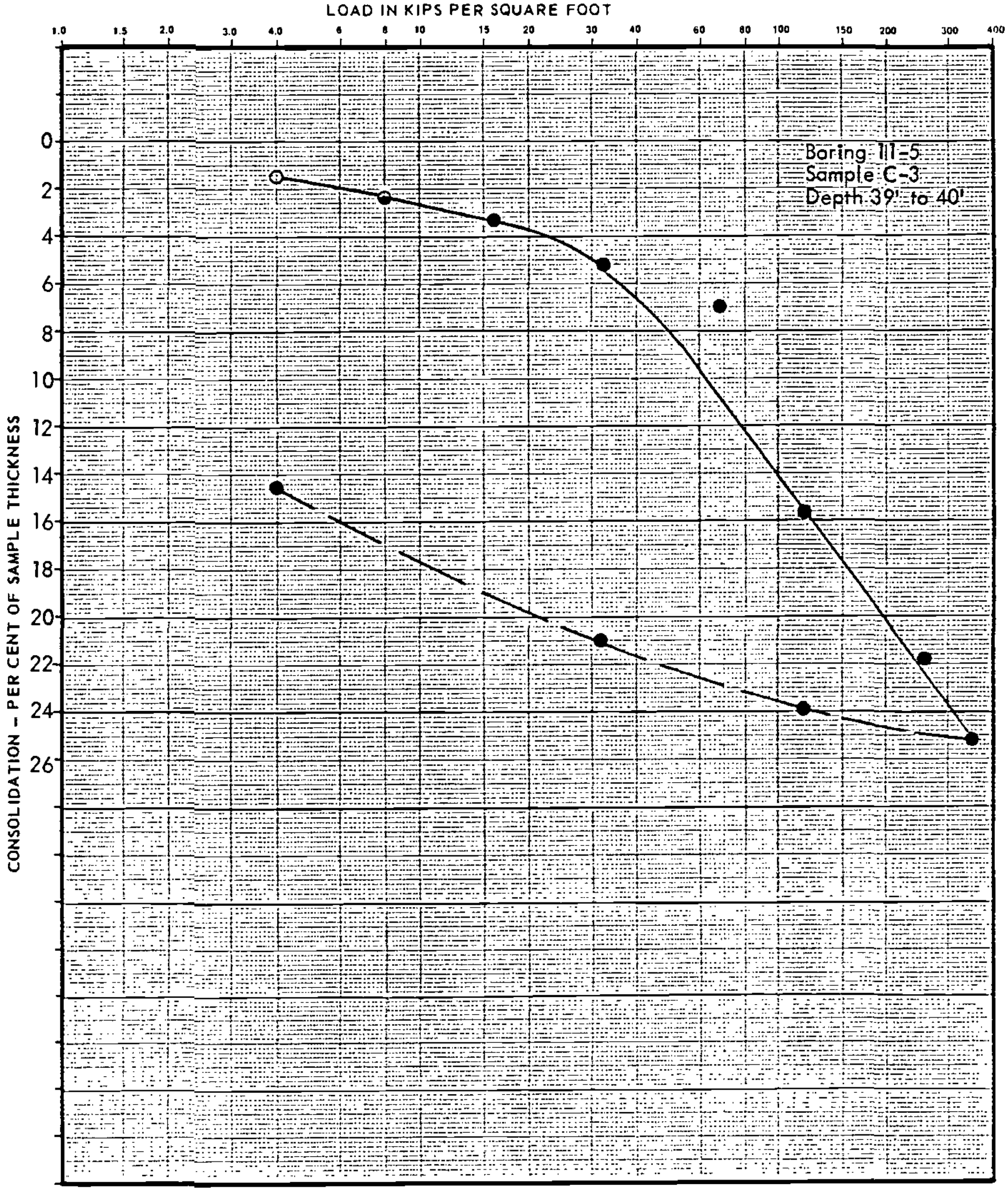
Figure No.
E-16



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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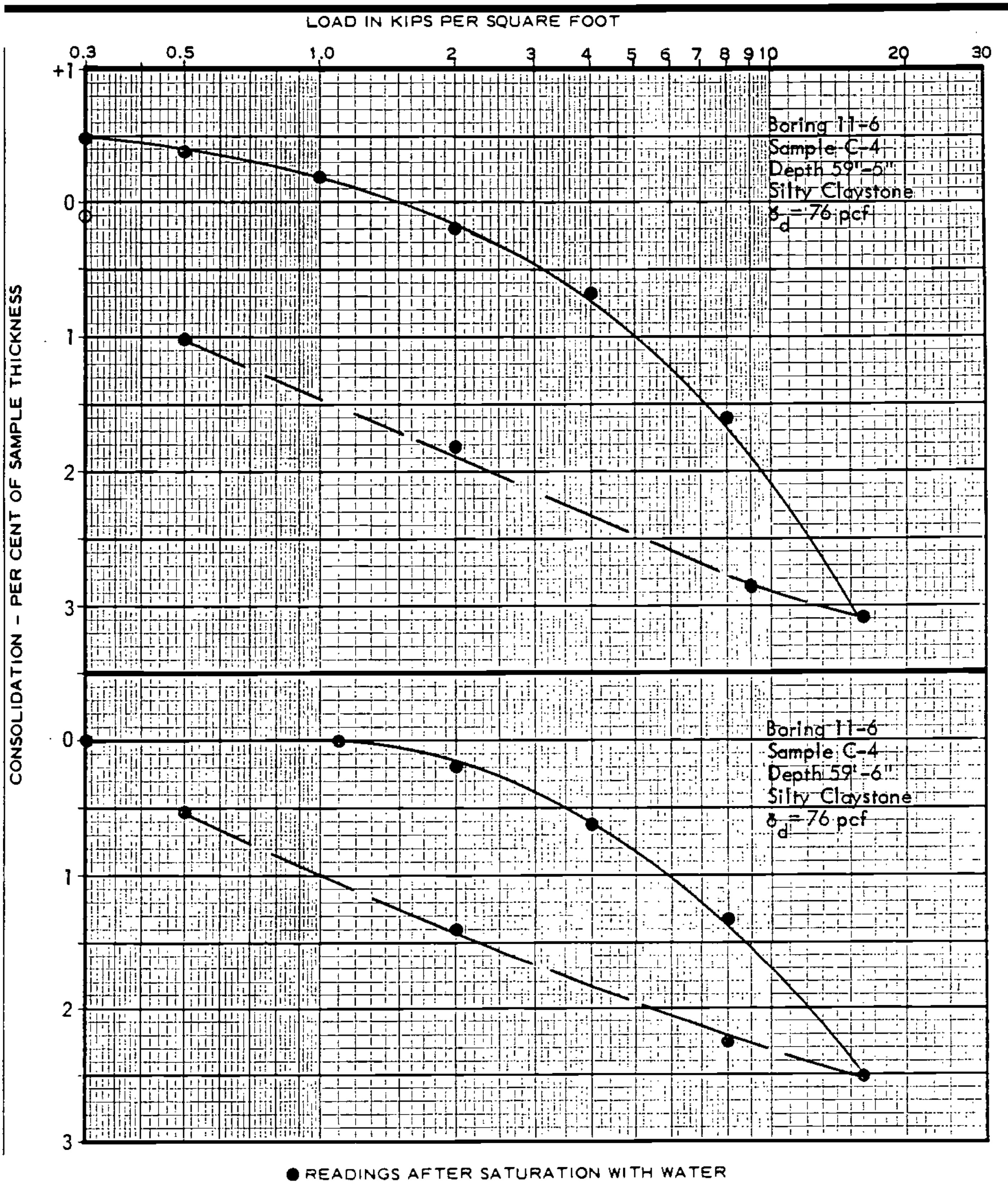
Figure No.
E-17

Approved for publication ______ by ______



CONSOLIDATION TESTS

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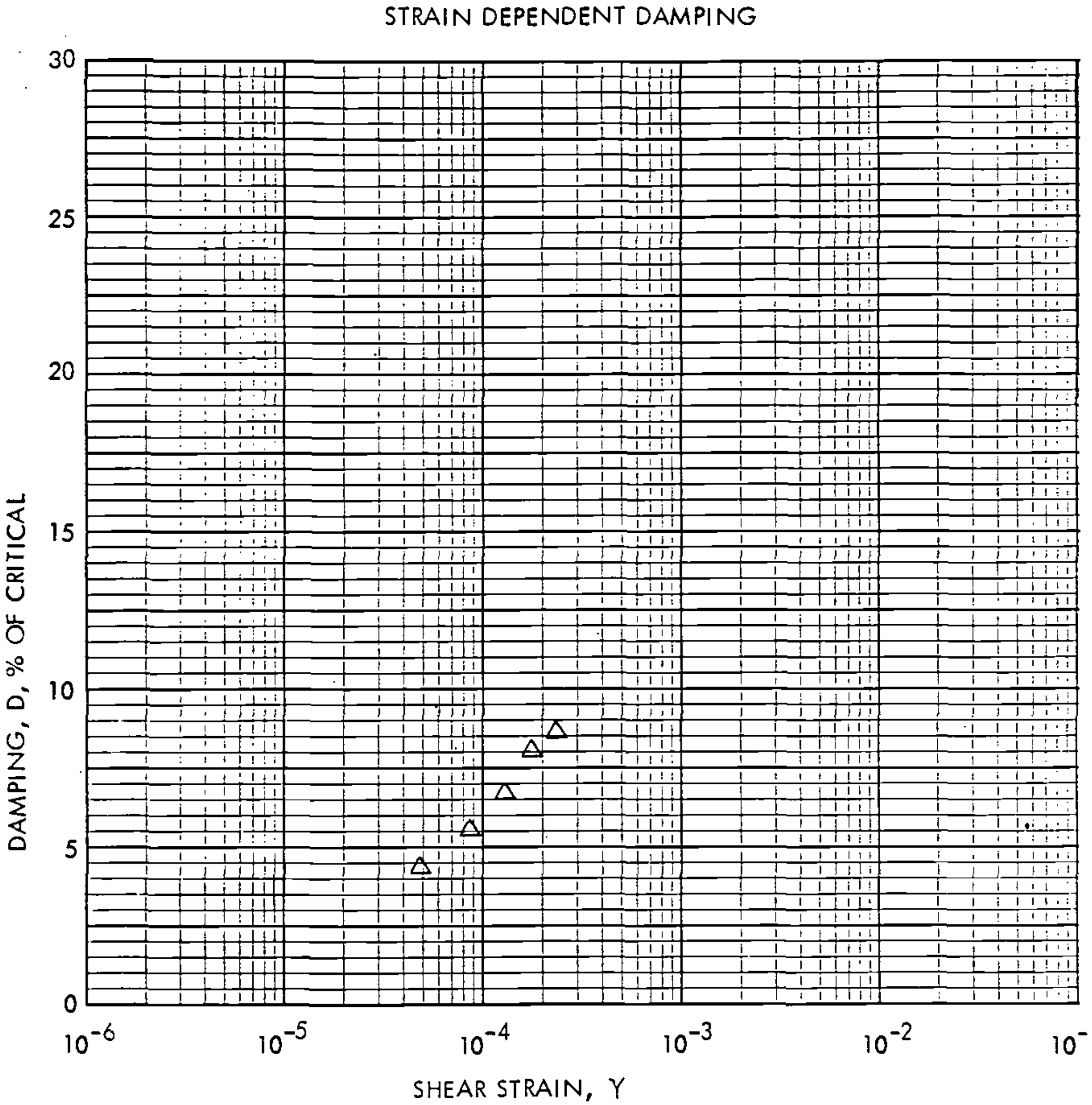
Figure No.
E-18

Approved for publication ________ by ________



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
11	S-3	61	103	22	50	△

Sample Description: Gray Siltstone; trace clay; moist

RESONANT COLUMN TEST

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83-1101

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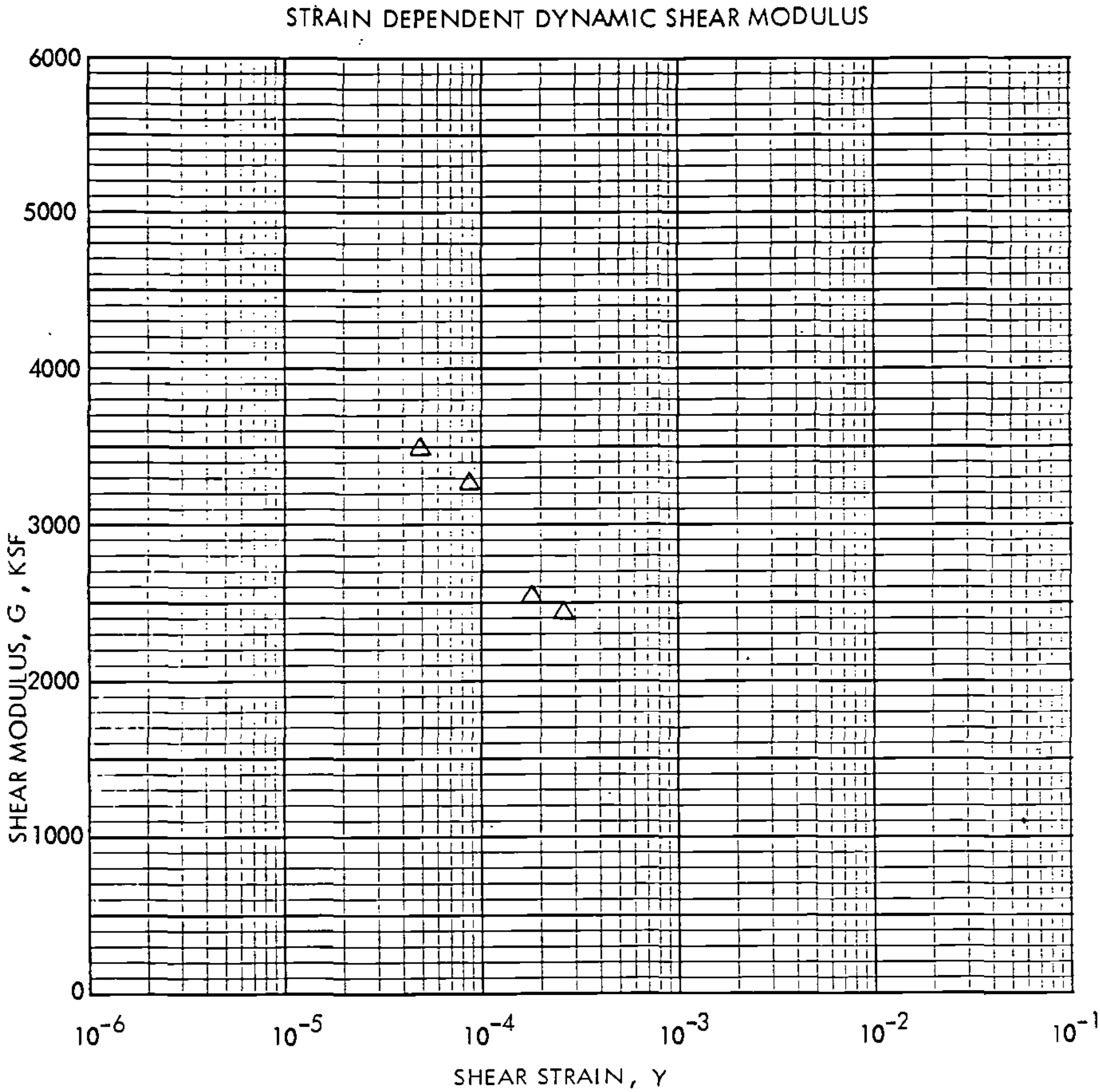
Figure No.
E-19



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
11	S-3	61	103	22	50	△

Sample Description: Gray Siltstone; trace clay; moist

RESONANT COLUMN TEST

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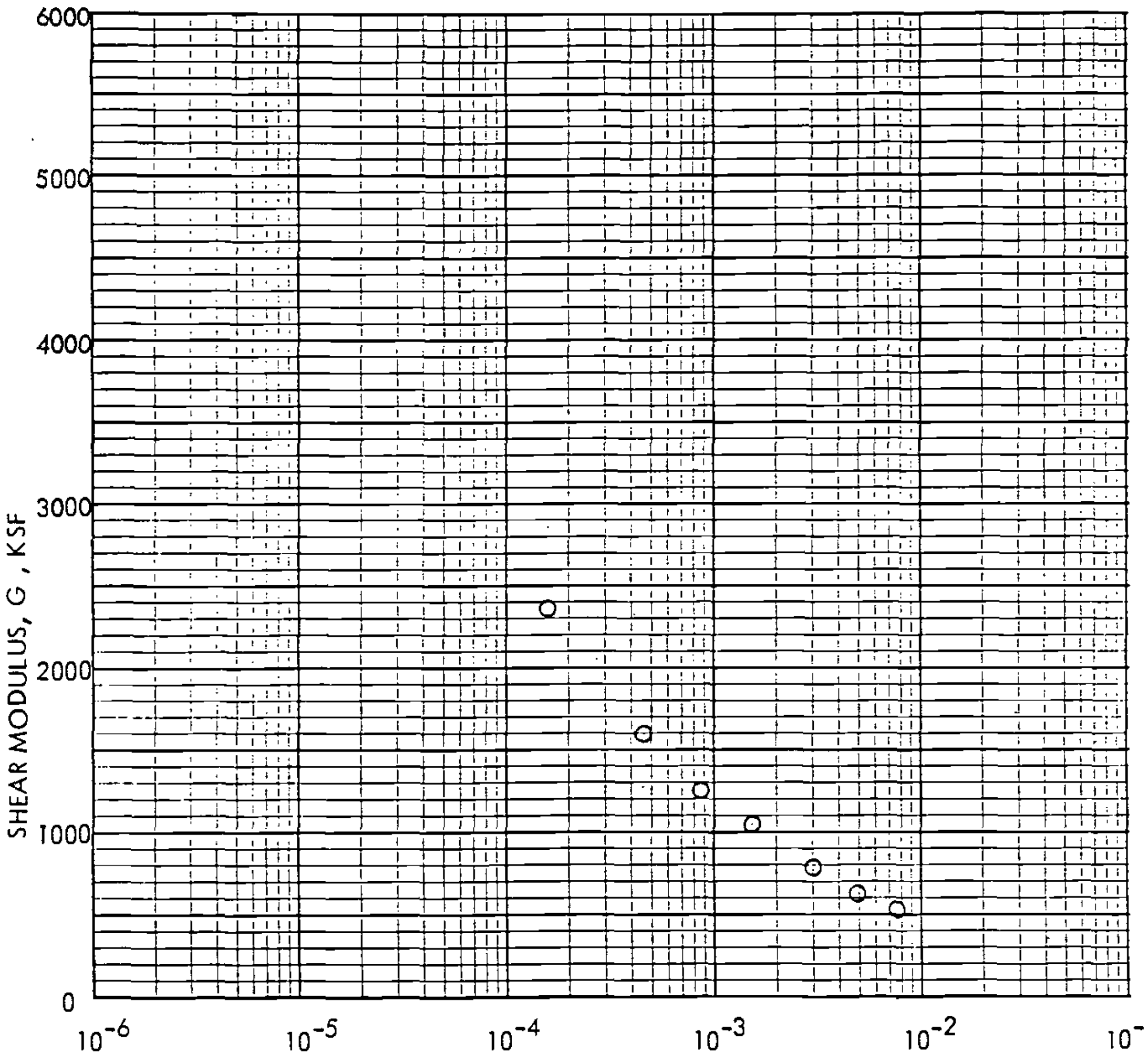
Figure No.
E-20

Approved for publication by

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
10	S-3	60	89	32	30

Sample Description: Gray Clayey Siltstone; stiff

DYNAMIC TRIAXIAL TEST

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Project No.
83-1101

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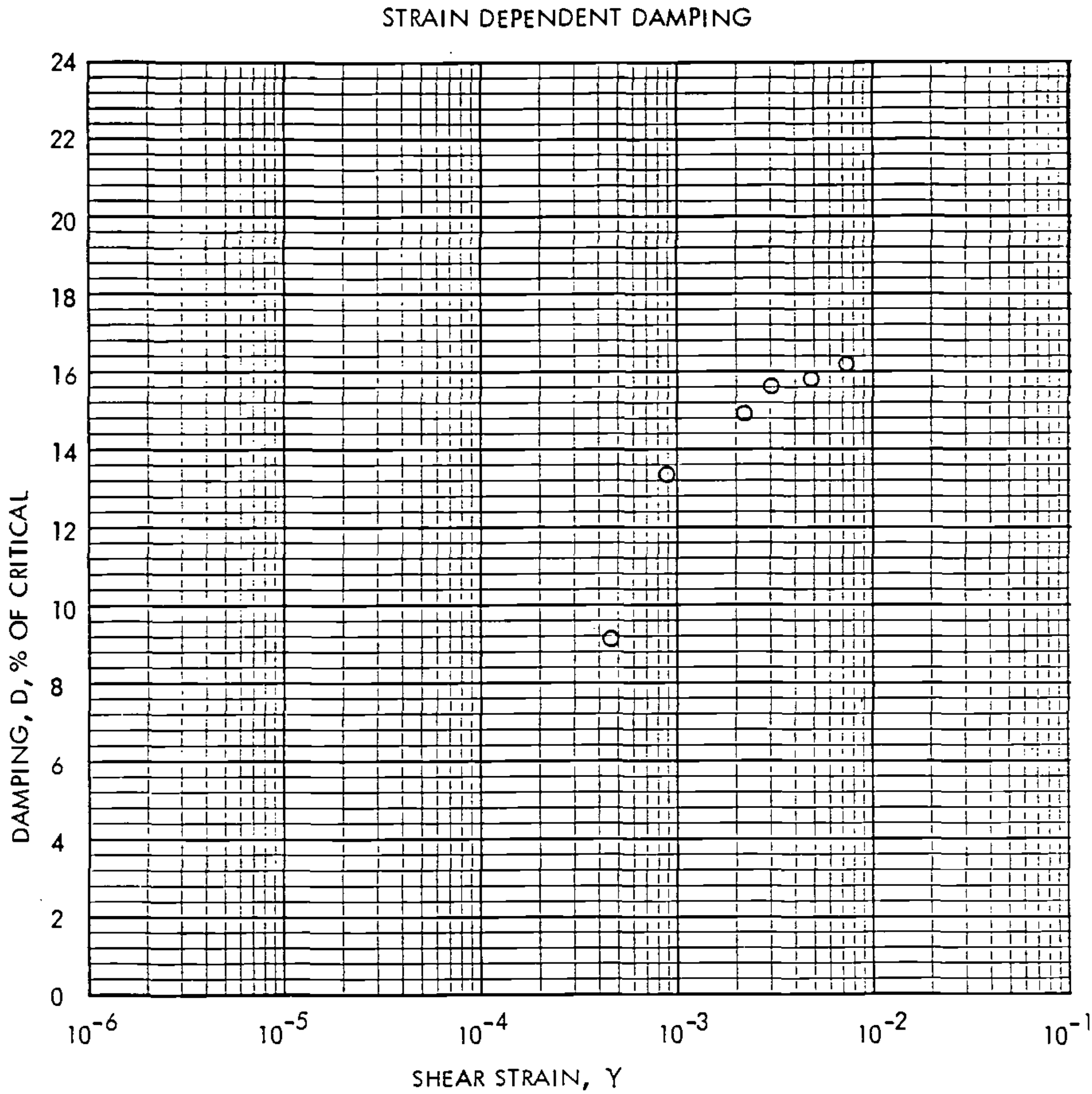
Figure No.
E-21

Approved for publication by



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
10	S-3	60	89	32	30

Sample Description: Gray Clayey Siltstone; stiff

DYNAMIC TRIAXIAL TEST

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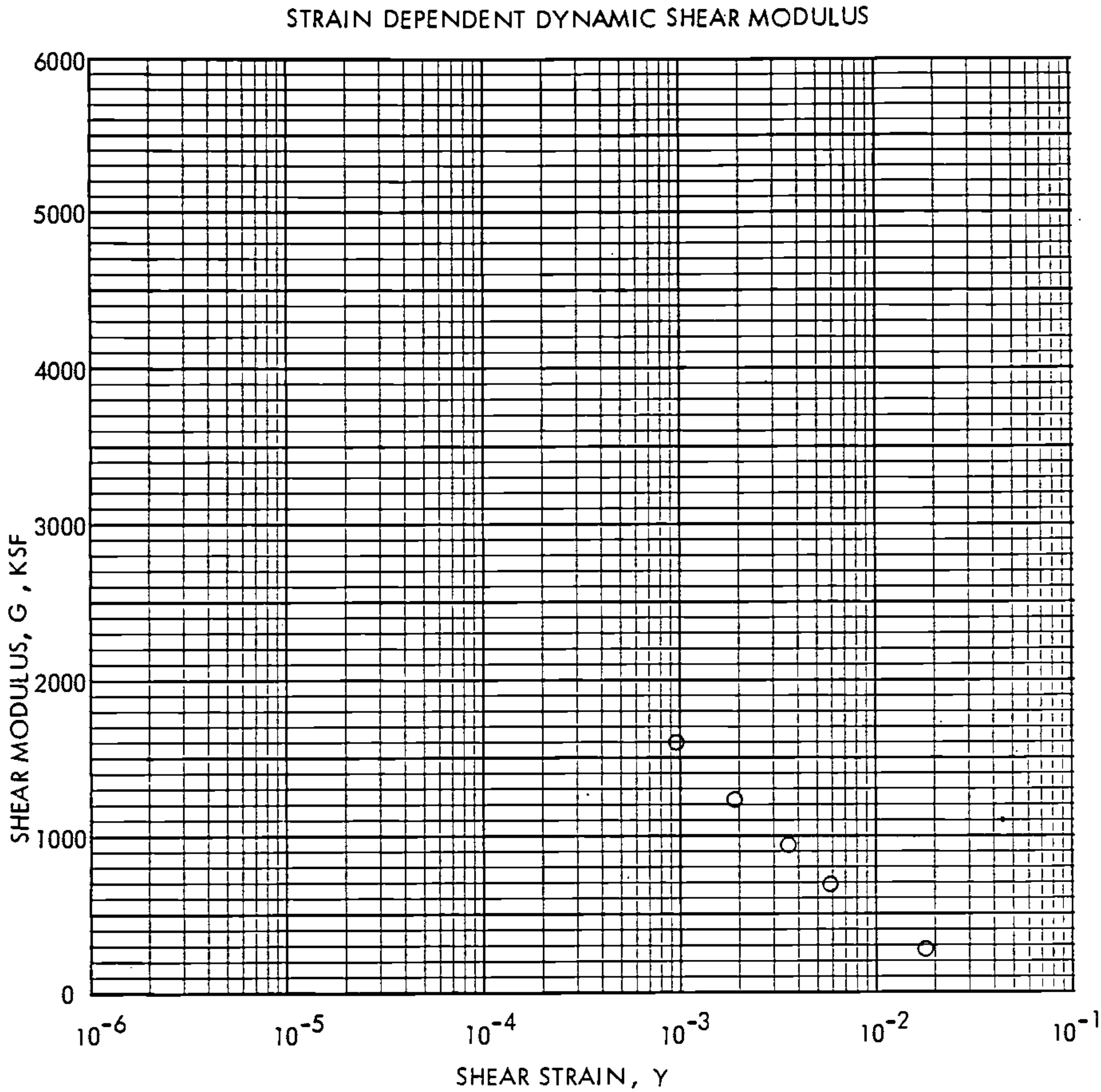
Figure No. E - 22

Converse Consultants Geotechnical Consultants



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
11	S-3	60	89	30	50	O

Sample Description: Stiff Dark Gray Silt with pockets of fine Sand

DYNAMIC TRIAXIAL TEST

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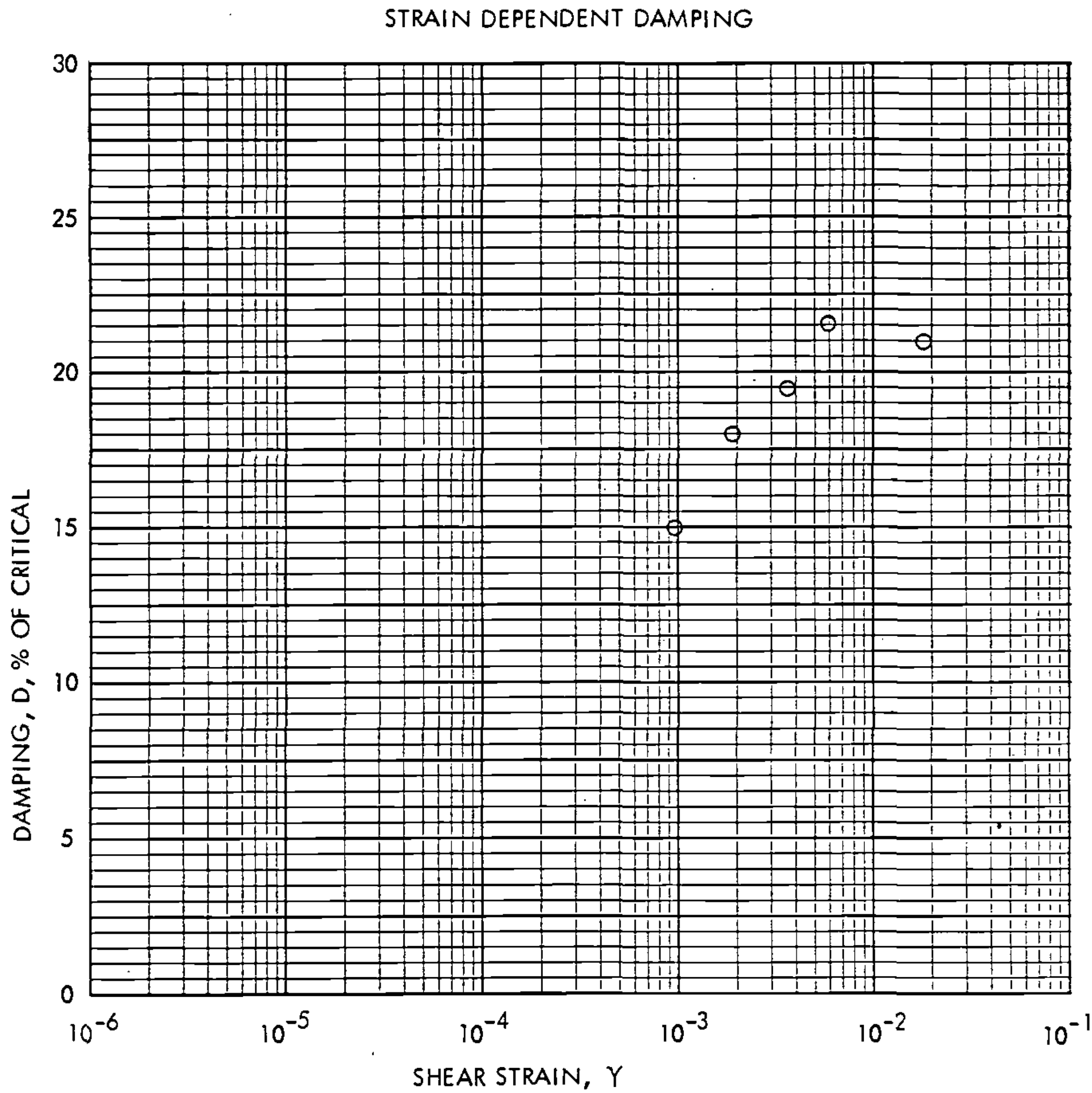
METRO RAIL PROJECT

Project No. 83-1101

Converse Consultants Geotechnical Consultants

Figure No. E - 23

Approved for publication ______ by ______



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
11	S-3	60	89	30	50	O

Sample Description: stiff Dark Gray Silt with pockets of fine Sand

Approved for publication 10/11/83 by [illegible]

DYNAMIC TRIAXIAL TEST

DESIGN UNIT A170
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Figure No. E-24

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Appendix F

Technical Considerations

APPENDIX F: TECHNICAL CONSIDERATIONS

F.1 SHORING PRACTICES IN THE LOS ANGELES AREA

F.1.1.1 General

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

F.1.2. Atlantic Richfield Project (Nelson, 1973)

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

o 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.

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

o 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 contrete mix above. The soldier pile spacing was typically 6 feet.

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

o 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.

o 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.

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

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

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

- o 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.
- o Shoring system consisted of steel WF soldier piles placed in 36-inch-diameter drilled holes spaced 6 feet on center.
- o 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.
- o Tieback anchors consisted of high-capacity 12- and 16-inch-diameter friction anchors.
- o Actual load imposed on the wall by the adjacent bridge was computed and added to the design wall pressures as a triangular pressure distribution.
- o Maximum horizontal deflection at the top of the wall was 3 inches, while the typical deflection was less than 1 inch. Adjacent to the exiting bridge, the deflections were essentially zero, with the tops of most of the soldier piles actually moving into the ground due to the high prestress loads in the anchors.
- o Survey of the bridge pile caps indicated practically no movement.

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

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

- o Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20^{o} to 40^{o}. Although the permanent ground water level was below the excavation level, perched zones of significant water seepage were encountered.
- o Shoring system consisted of steel WF soldier piles placed in 20-inch-

diameter drilled holes spaced at 6 feet on center.

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

F.1.5. Design Lateral Load Practices

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

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

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

TABLE F-1
SHORING LOADS IN LOS ANGELES AREA

PROJECT LOCATION	EXCAVATION DEPTH(ft)	SOIL CONDITIONS	ACTUAL DESIGN PRESSURE (P)	EQUIVELENT DESIGN PRESSURE (P')
Broadway Plaza Near 7th/Flower Station	15-30	Fill over Alluvium Sands	19.0H	15.2H
500 S. Hill	25	Fill over Sands and Gravel	22.0H	17.6H
Tishman Bldg. Near CEG-14	25	Alluvium-Clays, Sand, Silt	19.0H	15.2H
Equit. Life Near CEG-14	55	Alluvium Sand/ Siltstone	20.0H	17.5H
Arco Near CEG-9	70-90	Alluvium over Claystone	16.0H	12.0H
Century City Near CEG-20	70-110	Alluvium-Clays and Sands	18.0H	14.4H
St. Vincent's Near 3rd & Lk.	70	Thin Alluvium over Puente	15.0H	12.0H
Oxford Plaza Near 7th/Flower	40	Fill & Alluvium over Siltstone	21.0H	16.8H

Note:

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

F.2 SEISMICALLY INDUCED EARTHPRESSURES

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

During an earthquake the inertia forces are cyclic in nature and are constantly changing throughout its duration. It is unrealistic to replace these inertia forces by a single horizontal (and/or vertical) force acting only in one direction. In addition, the selection of an appropriate value of the hrozontal 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 ofapplication. 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,

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

Θ = $\tan^{-1} \frac{Kh}{1-Kv}$

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

γ = angle of wall friction

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes,

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

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

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

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

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

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

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

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

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

The recommended permanent wall uniform earthpressure (8H) presented in Figure 6-5 gives a seismic load $P_{AE} = 8H^2$. This value of P_{AE} was based on a peak ground acceleration of 0.3g (k_h = 0.2g) corresponding to the Operating Design Earthquake (ODE). Results of the Seismological Investigation (Part I) indicate the probability of excedence of 0.3g peak ground acceleration during an average 100 year period is on the order of 20%. This is an average recurrence of about 500 to 1000 years.

The allowable Building Code stress increase for seismic loading (33%) translates into an allowable uniform seismic earthpressure on the temporary shoring of about magnitude 6H. This earthpressure corresponds to a seismic coefficient (K_h) of about 0.15g and a peak ground acceleration ofabout 0.23g (using the recommended procedures). Data from the Part I Seismological Investigation indicates the 0.23g peak acceleration to have a probability of exceedence less than 5% during an average 2 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 6-2).

F.3 PREVIOUS TUNNELING EXPERIENCE - LACFCD SACATELLA TUNNEL

F.3.1 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

F.3.2 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.

F.3.3 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).

F.3.4. 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 F-1). Also note in Figure F-1 Puente Formation bedding (Unit C) and lack of ground water inflow.

F.3.5 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.

Digger Excavator
(LACFCD Sacatella Tunnel)

Figure F-1

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:

- 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.

F.3.6 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.

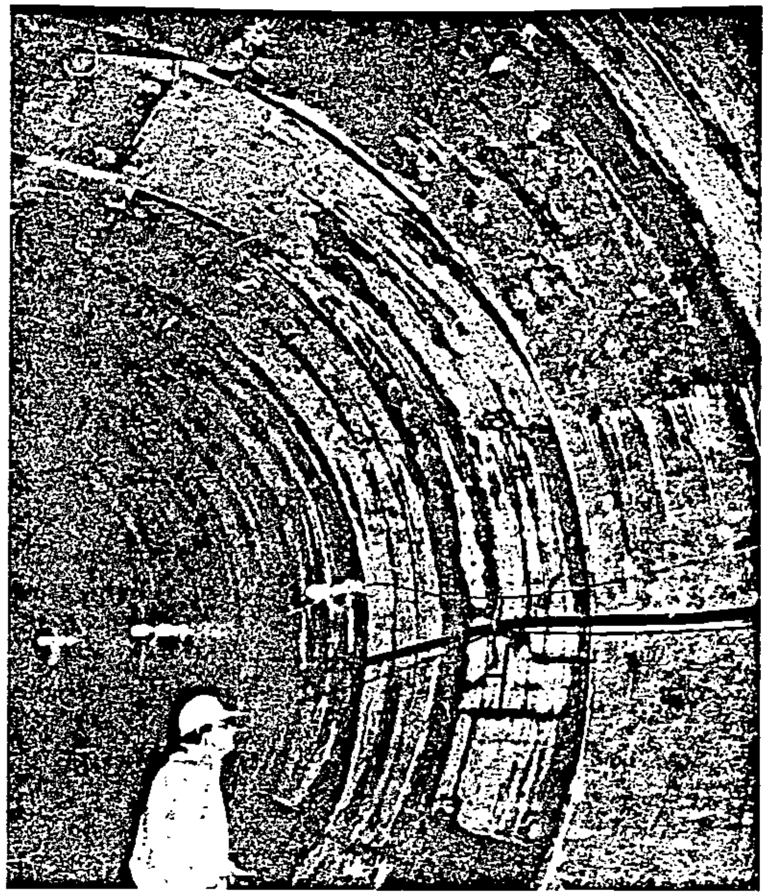
- The greatest apparent risk is where folding and a suspected fault may form significant traps (Wright, 1975, p. 8). Explosive-proof equipment was installed (although arc welding was permitted in the tunnel).
- The face was continuously monitored by a gas "sniffer" that automatically set off an alarm 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 F-2.

F.3.7 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.

F.3.8 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



Oil Seeps
(LACFCD Sacatella Tunnel)
Figure F-2

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."

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 F-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 F-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.

TABLE F-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)	EST. YIELD (gpm)	EST.[b] T	EST.[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).

F.3.9. 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.

F.3.10. 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.

F.3.11. 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.

F.3.12. 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.

F.3.13. 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 machine-bored. 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 G

Earthwork Recommendations

APPENDIX G - EARTHWORK RECOMMENDATIONS

The following guidelines are recommended for earthwork associated with site development. Recommendations for dewatering and major temporary excavations are presented in the text Sections 6.3 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 1.5:1 (horizontal to vertical).

- Structural Fill and Backfill:

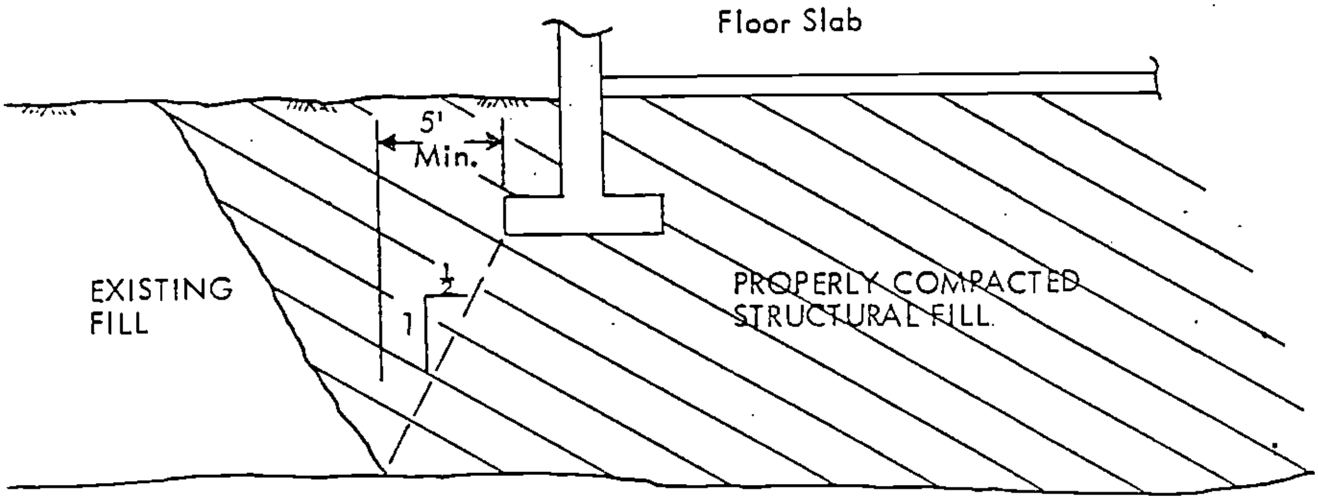
  Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed prior to backfilling the walls. Onsite soils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

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

If granular backfill or pea gravel is placed in an area of surface drainage, the backfill should be capped with at least 18 inches of relatively impervious type soil; i.e., soils containing at least 40 percent passing the No. 200 sieve.

- Foundation Preparation

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



- Subgrade Preparation:

Concrete slabs-on-grade at the subterranean levels may be supported directly on undisturbed dense alluvium. The subgrade should be proof rolled to detect soft or disturbed areas and such areas sould 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 structural fill. 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 sould be placed and compacted in accordance with "Structural Fill and Backfill."

- Recommended Specifications for Fill Compaction:

The following specifications are recommended to provide a basis for quality control during the placement of compacted fill.

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

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

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

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

5. The soils engineer shall observe the plac ement 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.