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**Converse Consultants**
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# GEOTECHNICAL REPORT

## METRO RAIL PROJECT

## DESIGN UNIT A135

BY

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

SEPTEMBER 1983

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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."

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**Converse Consultants**
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September 15, 1983 83-1101-43

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 A135 Union Station and Crossover 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 A135.

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 Bob Plum, Howard Spellman, Julio Valera, Jim Doolittle and Roger Hail.

Respectfully submitted,

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

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## PROFESSIONAL CERTIFICATION




Robert M. Pride
Senior Vice President




Howard A. Spellman
Principal Engineering Geologist

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.

# E R R A T A

## GEOTECHNICAL REPORT

## DESIGN UNIT A 135

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1. OMIT TITLE AND TEXT OF SECTION 6.9.3, PAGE 35.

2. OMIT TEXT OF SECTION 6.11.1, PAGE 35. REPLACE WITH THE FOLLOWING TEXT:

> "DESIGN PROCEDURES AND CRITERIA FOR UNDERGROUND STRUCTURES UNDER EARTHQUAKE LOADING CONDITIONS ARE DEFINED IN THE SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT (SCRTD) REPORT ENTITLED "SUPPLEMENTAL CRITERIA FOR SEISMIC 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 EARTHQUAKE PRESSURE DIAGRAM IN FIGURE 6-6.

4. OMIT TITLE AND TEXT OF SECTION 6.11.3, PAGE 38.

5. OMIT TEXT OF PARAGRAPH 2, PAGE 198, APPENDIX G-2.

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# Section 1.0

# Executive Summary

A135

## 1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for the proposed Union Station and Crossover structure which will form the eastern terminus of the Metro Rail Project in the Los Angeles area. The proposed structures will be about 1000 feet long, 65 feet wide, and will require excavating 46 to 60 feet below the existing ground surface. The purpose of the investigation is to provide geotechnical information to be used by design firms in preparing designs for the project. Although this report may be used for construction purposes, it is intended primarily for design purposes and not to provide all the geotechnical information that may be required to construct the project.

The subsurface conditions encountered consist of 75 to 85 feet of Young Alluvium consisting primarily of gravelly sands with an upper sandy zone and a lower bouldery zone. The central area of the proposed construction is covered by about 14 feet of fill. Underlying the Young Alluvium, the explorations encountered weak claystone bedrock of the Puente Formation. Ground water was encountered within the gravelly sands at elevations ranging from 20 to 30 feet above the depth of the proposed excavation. During pumping of a test well, gas consisting of hydrogen sulfide mixed with hydrocarbons was released and exploded within the test well during a gas detector test. Oil is known to occur at shallow depths within the Puente Formation less than 2000 feet south of the site.

Construction of the Station and Crossover structures will involve making a 60-foot deep excavation through predominately dense granular soil to some 30 feet below the static water table. This will involve a major dewatering effort and may require special construction provisions due to the possible production of gases during dewatering. The permanent structures will bear on and retain dense granular soil.

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

- EXCAVATION DEWATERING: Since the excavation will extend 20 to 30 feet below the static water table in permeable sand and gravel, a major dewatering system will be required to construct the proposed excavation. The report presents ground water data and general criteria to be satisfied by the contractor designed, installed, and operated dewatering system.

- UNDERPINNING: The report presents general guidelines for assessing the need to consider underpinning.

- TEMPORARY EXCAVATION SUPPORT: We understand that 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 drilled soldier piles with either

tiebacks or cross-lot bracing for lateral support. Accordingly, we present design criteria for these types of systems. Other systems may also be appropriate and should be considered by the contractor.

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

- PERMANENT FOUNDATION SYSTEMS: The Station and Crossover Structure can be adequately supported on the dense, natural soils. The report presents allowable bearing pressures, and estimates of foundation settlements.

- LOADS ON PERMANENT SLABS AND WALLS: The report presents recommended lateral design loads on the permanent structures. These include hydrostatic uplift pressures on the bottom slab which equal about 2000 psf.

- LIQUEFACTION POTENTIAL: Since the site is underlain by 50 to 60 feet of saturated granular soils, a liquefaction analysis was performed. Based on the results of the analysis and engineering judgement, it is our opinion that the site would not be subject to liquefaction during the postulated design ground motion.

# Section 2.0
# Introduction

## 2.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Design Unit A135. The subject Design Unit includes the Union Station and Crossover Structure. 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 borings, a pump test, 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 preliminary geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experience 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-9 through A-18. These documents were prepared for SCRTD by Harry Weese & Associates, Tippetts-Abbett-McCarthy-Stratton, Environmental Collaborative, Inc. and Gin Wong Associates.

# Section 3.0

# Project and Site Description

## 3.0 PROJECT AND SITE PREPARATION

### 3.1 SITE DESCRIPTION

The proposed Union Station and Crossover structure, as shown on Drawings 2 and 3, will be located below the currently operating railroad yards east of the existing Union Station Terminal Building. This area is at the edge of the downtown core and is bounded by Alameda Street to the west, Macy Street to the north, Vignes Street to the east and the Santa Ana Freeway to the south. Located within this area is the Union Station Terminal Building, Bus Terminal Building, freight buildings, surface parking, tracks and a vacant, unimproved area east of the tracks. An existing Pedestrian Tunnel is located under the existing track yard. Immediately to the north of the site is the Post Office Terminal Annex which is a 4-story masonry structure.

The area around the existing Terminal Building and east of the Rail Yard adjacent to Vignes Street is relatively level at an elevation of about 278 to 280. The Rail Yard area has been filled to an elevation of about 292.

### 3.2 PROPOSED STATION AND CROSSOVER STRUCTURE

The proposed Union Station structure will be about 550 feet long and 65 feet wide. Adjacent to each end of the station will be a Crossover structure. The Crossover structure on the west side will be about 450 feet long. Design Unit A135, the subject of this report, includes both Union Station and the western Crossover structure. Thus, the total length of the cut-and-cover section discussed in this report is approximately 1000 feet. In addition, the station will include entry structures on both the west and east sides of the station. These entry structures will include some above-ground construction with the eastern entry connecting to the existing Pedestrian Tunnel.

The proposed top of rail varies from about Elevation 238 at the west end of the station platform to Elevation 239 at the east end of the station platform. Assuming that the bottom slab will be about 4 to 6 feet thick, the Station area will require an excavation to about Elevation 232. This will be some 60 feet below the existing Track Yard level and some 46 feet below the adjacent lower areas to the west. Generally, the full excavated depth will be required only in the track and platform areas. Portions of the entry structures will also extend below grade with required excavation depths on the order of 10 to 25 feet.

It is understood that the permanent structure will be designed as a rigid reinforced concrete box with one row of interior columns located along the longitudinal centerline of the station. Portions of the Station roof slab will support the existing Rail Track and a few feet of earth fill. The roof slab of the Crossover structure will support 40 feet of earth cover for a major portion of its length.

It is understood that all existing above-grade structures will be removed around the western section of the proposed Station to facilitate construction. The need to provide temporary decking over all or portions of the construction excavation has not been determined at the time of this writing.

# Section 4.0

# Field Exploration and Laboratory Testing

A 135

## 4.0 FIELD EXPLORATION AND LABORATORY TESTING

### 4.1 GENERAL

The conclusions and recommendations presented in this report are based primarily on the field and laboratory investigations performed in 1983 for this study and those performed in 1981 for the initial Geotechnical Investigation Report. In addition, the subsurface data compiled in 1977 by the USGS and by Kaiser Engineers in 1962 were reviewed. Comprehensive research of all existing boring data in the vicinity of the Project was beyond the scope of this report. Thus there may be additional boring data developed for specific buildings adjacent to the alignment which have not been reviewed.

In general the geotechnical studies include borings, a man-sized auger boring, a pump test, ground water observation wells, field geophysical measurements, field gas notations, water quality laboratory tests, and soil/rock laboratory tests.

### 4.2 BORINGS

The borings drilled in 1983 in this Design Unit include five borings (5-1 to 5-5) ranging in depth from 80 to 100 feet. The 1981 exploration program included 150-foot deep borings CEG-4 to CEG-6 in the general site area. Kaiser borings 114 to 116 are in the vicinity. The locations of the borings are shown on Drawings 2 and 3. Ground water observation wells were installed in borings CEG-4, CEG-6, 5-1 and 5-5. The observation wells included a perforated section within the lower 50 feet of the boring with gravel backfill placed to the surface seal. Section 5.4 presents a summary of ground water level measurements obtained from the observation wells. The logs of the 1981 and 1983 borings along with descriptions of the field procedures are included in Appendix A.

### 4.3 PUMP TEST

A pump test was performed adjacent to boring 5-1 as shown on Drawings 2 and 3. The well was 12 inches in diameter (I.D.), 82.5 feet deep, and perforated below the ground water table. Two 4-inch diameter observation wells were installed to evaluate water level drawdown at distances of 60 and 120 feet west of the test well. Boring 5-1 was also used as an observation well during the test.

The test well was pumped initially at a discharge rate of 175 gpm for about 8 hours. The test was terminated early due to problems of gas in the discharge affecting the flow meter measurements. Following a 15-hour recovery period a second test was performed at a reduced discharge rate of 150 gpm for 23 hours. The maximum drawdown at the 60-foot piezometer was 2.8 feet (after 23 hours at 150 gpm).

Appendix B provides a report of the pump test procedures and results. The well casing is capped but open at the time of this writing. It may be possible to re-use the casing in the future provided that effects of ground water corrosion do not prevent its use.

## 4.4 GEOPHYSICAL MEASUREMENTS

Downhole and crosshole shear wave velocity surveys were performed in boring CEG-5 (see Drawing 2). Downhole shear wave velocity measurements were made at 5-foot vertical intervals from ground surface to a depth of 130 feet. Crosshole shear wave velocity measurements were performed in Boring CEG-5 at 5-foot vertical intervals. An array of three borings, spaced about 15 feet apart, was used for the crosshole survey. The results of the downhole and crosshole surveys are presented in Appendix C.

## 4.5 OIL AND GAS ANALYSES

In the 1981 and 1983 borings, noxious sulfur odors ($H_2S$) were detected. Gas was ignited, causing an explosion, in the pump test well (see Appendix B). During the 1981 investigation gas chromatograph analyses (Appendix D) were performed in Boring CEG-2 located about 2000 feet to the east; however, no chemical analyses of the oil, nor gas chromatographic analyses, were performed at the of Union Station site in the 1981 or 1983 borings.

## 4.6 WATER QUALITY ANALYSES

Water quality analyses were performed on samples taken from Borings CEG-4, CEG-6, and man-size Boring 6A. The results are presented in Appendix E. The water was tested for basic cations, anions, conductivity, total dissolved solids, pH, turbidity and boron.

## 4.7 GEOTECHNICAL LABORATORY TESTING

Laboratory testing programs were performed on representative soil and rock samples for both the 1981 investigation and this investigation. 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. Since the bedrock formation encountered at the site is "soil-like", the bedrock samples were subjected to the same type of testing as the soil samples.

Appendix F summarizes the testing procedures and presents the results from both the 1981 and 1983 programs.

# Section 5.0

# Subsurface Conditions

## 5.0 SUBSURFACE CONDITIONS

### 5.1 GENERAL

The geologic sequence in the site area consists of Artificial Fill, Young Alluvium ($A_1$), and bedrock of the Puente Formation (C). The geologic units include:

- Artificial Fill: This includes man-made fills placed for construction of the existing Union Station and associated Rail Yard.

- Young Alluvium (Granular) - $A_1$: This deposit, which is of Holocene age, is primarily sands and gravels deposited in swift-flowing streams. Locally, the unit is known to contain boulders up to 2 feet in diameter and is likely to contain boulders up to 4 feet in diameter.

- Puente Formation - C: The bedrock underlying the area is of upper Miocene age and consists of well stratified, steeply dipping weak siltstones and claystones. Local hard, cemented sandstone beds ranging from less than 1 inch to 3 feet thick may be encountered. Based on boring data and cut slope exposures in the Puente Formation along Beverly Boulevard (1 mile west of the site), it is estimated these hard beds comprise less than 2% of the Puente Formation.

The Los Angeles anticline (upfold), a major geologic structure trending about N70W, influences the dip of the bedrock strata. There are no known active or potentially active faults identified in the Union Station area.

Drawings 2 and 4 show lines, grades and geologic sections through the site. In general, the subsurface conditions consist of some 75 to 85 feet of alluvium consisting primarily of gravelly sands with an upper sandy zone and a lower bouldery zone. Underlying the alluvium are weak claystones and siltstones. Underlying the existing Track Yard, some 14 feet of fill was encountered. Ground water was measured at elevations ranging from about 252 to 262 in the station area.

Assuming a bottom excavation elevation of about 232, the proposed excavation would extend through fill, sands and gravelly sands. The lower Station slab would be founded on gravelly sands about 20 to 30 feet below the measured static ground water levels.

### 5.2 SUBSOILS

#### 5.2.1 Fill

The fill underlying the existing Track Yard as encountered in Borings 5-2 and 5-3 consisted of stiff silty clay and clayey silt with varying amounts of sand and gravel to a depth of about 14 feet (see Drawing 4, Geologic Section A-A'). Boring 5-3 encountered gravel fill (up to 2½-inch diameter) from the ground surface to a depth of about 4½ feet. It is not known whether the gravel

layer, which may be railroad ballast, underlies much of the existing Track Yard or occurs only in localized pockets. The management of Union Station would not allow exploration within the Track Yard.

Borings 5-1, 5-4 and 5-5 located outside the Track Yard encountered about 2 to 6 feet of fill consisting of dense silts and sands. Borings CEG-5 and CEG-6 did not encounter fill.

Boring 5-2 encountered a boulder, or concrete debris, at a depth of about 13 feet. Boring 5-5 encountered brick debris within the 5-foot fill layer. Based on this data and the probable filling history of the area, bricks, timber, concrete blocks, and other building debris will probably be encountered within the fill.

### 5.2.2 Young Alluvium ($A_1$)

The variable layer of fill is underlain by Young Alluvium. Based on the borings, the Young Alluvium consists of a thin upper sandy unit, the main gravelly sand unit, a lower sand unit and a bouldery zone overlying the Puente Formation (see Drawing 4). Specific descriptions of these materials are as follows:

- Upper Sand Unit: Below the fill, the borings encountered a thin 3- to 6-foot layer of silty fine sand. Underlying this silty material, Borings 5-4 and 5-5 encountered 5 to 11 feet of clean fine sand. Boring CEG-5 did not encounter this unit, while CEG-6 encountered a silty sand at a depth of 24 to 38 feet. Based on the standard penetration resistance and unit weights, these upper sands are medium dense to dense. Typical grain size curves for these materials are shown in Appendix F.

- Main Gravelly Sand Unit: The majority of the Young Alluvium consisted of well graded gravelly sands with about 5% fines. The gravel content decreases with depth. Due to the gravel content, sample recovery was generally poor and limited to soil particles smaller than the inside diameter of the samplers (1.4 to 2.4 inches). It is believed that the material probably grades through the coarse gravel sizes (2 to 3 inches) with occasional cobbly zones. Occasional boulders 2 to 4 feet in diameter are likely to be encountered during excavation. These interpretations are based on the drilling action (such as rig chatter and drilling difficulty), observations in man-size auger Boring 6A and the swift water depositional environment. Grain size curves for samples recovered from this unit are shown in Appendix F.

  Thin layers/lenses of sand, silts and clays were occasionally encountered within the main gravelly sand unit in Boring 5-4 and man-size auger Boring 6A. Boring 5-4 encountered a zone of sands interbedded with thin beds of stiff silty clay between a depth of about 28 to 41 feet. The occurrences of finer units within the main gravelly sand unit are consistent with the depositional environment of the Young Alluvium. Thus, layers of sands, silts, and/or clays ranging in thickness from a few inches to more than a foot may be expected during excavation in the main

gravelly sand unit. These finer grained interbeds are expected to be of limited lateral extent.

Due to the gravel content, the high standard penetration resistances recorded through this unit have little meaning. However, based on the probable high energy of deposition (i.e., deposited in rapidly flowing streams) and standard penetration resistance measured in the intermediate sand layer in Boring 5-4, it is believed that the main gravelly sand unit is medium dense to dense.

- Lower Sand Unit: Boring 5-2 through 5-5 encountered 7 to 16 feet of dense, poorly graded fine and fine to medium sand with occasionally silty lenses. This layer was encountered at an elevation of about 215. Appendix F shows typical grain size curves for these materials.

- Bouldery Zone: At elevations ranging from about 200 to 210, the borings encountered a boulder zone some 10 to 15 feet thick overlying the Puente Formation. Due to the composition of this unit, the only samples obtained were cuttings from the drilling mud. These cuttings indicated that the boulders were predominately of granitic composition. It is likely this zone is composed of boulders 2 to 3 feet in diameter in a matrix of sands and gravels. Man-size auger Boring 6A encountered boulders up to 2 feet in diameter from a depth of 0 to 22 feet.

The following description of bouldery ground, applying to Young Alluvial deposits in the Los Angeles River, is excerpted from the 1981 Geotechnical Investigation, Volume I, page 29:

> "Bouldery Ground - Contains occasional boulders in the ancestral Los Angeles River channels (Reaches 1, 2 and 8) up to 2 ft diameter; boulders observed at the surface, prior to lining the Los Angeles River at the Macy Street crossing, were reported to be 4 ft diameter. The presence of boulders and cobbles is noted in the boring logs. However, boulders were noted only where encountered; their absence, therefore, cannot be assumed where not noted, especially near the Los Angeles River. The possibility of undetected irregular-shaped lenses of large and small boulders and cobbles should be assumed."

## 5.3 BEDROCK (C)

Borings 5-1 and 5-5 penetrated 5 to 10 feet, respectively, into the Puente Formation underlying the boulder zone (Drawing 4). In boring 5-1, the 5 feet penetrated consisted of a weak sandy claystone and clayey sandstone. The 10 feet penetrated in Boring 5-5 consisted of weak claystone laminated with thin silty claystone and sandstone beds. Based on the results of the 1981 geotechnical investigation, the Puente Formation is believed to contain local hard, cemented sandstone beds ranging from less than 1 inch to 3 feet thick. The top of the bedrock appears to vary from about Elevation 190 at the east end of the station to about 200 at the west end of the west crossover structure.

Borings CEG-4, CEG-5 and CEG-6 drilled during the original 1981 geotechnical investigation were advanced some 70 feet into the Puente Formation (Drawing 2). In boring CEG-6, the bedrock consisted primarily of siltstone and silty sandstone. Borings CEG-4 and CEG-5 encountered primarily claystone. Thus, it appears that the Puente Formation may grade from sandy claystone and siltstone on the west side of the station to claystone on the east side.

## 5.4 GROUND WATER

The proposed Union Station and Crossover structure lies within the Los Angeles Forebay portion of the Central Ground Water Basin. The term "forebay" refers to a recharge area where substantial infiltration of surface water can occur. Due to heavy urban development, the infiltration has probably been significantly reduced. The Los Angeles River flows in a concrete-lined channel some 1600 feet southeast of the site. Ground water elevations and fluctuations measured in Borings CEG-4, CEG-5, CEG-6, 5-1, 5-5 and man-sized auger Boring 6A are shown below.

GROUND WATER ELEVATIONS (ft*)

| BORING | 3/10/81 | 3/17/81 | 6/17/81 | 4/28/82 | 2/08/83 | 2/14/83 | 4/27/83 | 6/08/83 | 9/02/83 |
|---|---|---|---|---|---|---|---|---|---|
| CEG-4 | 249 | | 249 | 245 | | | | | |
| CEG-5 | 255** | | | | | | | | |
| CEG-6 | | 262 | 260 | 260 | | | | | 262 |
| 5-1 | | | | | 263 | | 264 | 265 | 260 |
| 5-5 | | | | | 252 | | 253 | 254 | 254 |
| 6A | | | | | | 270 | | | |

*Rounded to the nearest foot.
**Piezometer tubing not installed.

It appears that the ground water varies across the site ranging from about Elevation 264 in boring 5-1 (located west of the western crossover to about Elevation 249 in boring CEG-4 (located east of the eastern crossover). This represents a ground water gradient of about .006 foot per foot (vertical/horizontal) in a southeast direction. The regional water table aquifer reportedly slopes to the south at a gradient of about .009 (Los Angeles County Flood Control, 1976). Thus, the measured local gradient appears to be in reasonable agreement with the reported regional gradients. Fluctuations were about 4 feet in Boring CEG-4, and 2 feet in Borings CEG-6, 5-1 and 5-5 during the observational period.

Water quality in Boring 6A was generally good when compared to the very poor water quality in Borings CEG-4 and CEG-6. In our opinion, oil field brine contained in the Puente Formation was penetrated by Borings CEG-4 and CEG-6 and is not representative of ground water quality in the overlying Young Alluvium. Man-sized auger Boring 6A bottomed in Young Alluvium and did not penetrate the Puente Formation. Therefore, in our judgement, chemical constituents in Boring 6A are more representative of ground water quality in the

Union Station area. Selected water quality parameters are shown below, and complete chemical analyses are in Appendix E.

SELECTED WATER QUALITY PARAMETERS

| Boring No. | Depth Water Sampled (ft) | Date Sampled | pH @ 25° C | Total Dissolved Solids (ppm) | Sulfate $SO_4$ (ppm) | Chloride (ppm) |
|---|---|---|---|---|---|---|
| CEG-4 | 30 | 02-19-81 | 7.6 | 5,085 | 79 | 2,800 |
| CEG-6 | 19 | 02-20-81 | 7.5 | 20,230 | 27 | 12,255 |
| 6A | 20 | 02-11-83 | 8.3 | 688 | 108 | 74 |

The hydraulic properties of the Young Alluvium were investigated by performing a pump test in a well located 5 feet east of Boring 5-1 (see Drawing 2). The testing procedures and test results are presented in Appendix B. The general hydraulic characteristics of the Young Alluvium are discussed in Section 5.6.3.

## 5.5 GAS

Hydrogen sulfide odors were detected during drilling in borings CEG-4 (trace), CEG-6, and 5-1 through 5-5. During the pump test, considerable gas was released causing discharge flow measurement problems. The gas appears to have been hydrogen sulfide mixed with hydrocarbons including methane. Appendix B discusses the observed gas release and resulting impacts on the pump test results. In man-sized auger Boring 6A, located some 500 feet west of the pump test well, no gas was detected, even though the bottom of the hole was extended in free air to a depth of some 35 feet below the static ground water table.

The Union Station Oil Field is located some 2000 feet south of the proposed station site. Little is known about the oil field, but it does produce from the Puente Formation at very shallow depths.

Boring CEG-2 located some 2000 feet east of the proposed station site penetrated the Young Alluvium/Puente Formation contact at a depth of 38 feet. Oil was encountered at this contact as well as in sandstone layers within the Puente Formation from 38 to 100 feet (bottom of hole). Gas was first detected at a depth of 37 feet in Boring CEG-2. Gas chromatograms run in Boring CEG-2 (Appendix D) indicate 100 ppm methane and 500 ppm ethane, thereby resulting in a classification in the lower explosive limit (5% LEL).

It is believed that free gas may be contained in the Young Alluvium or underlying Puente Formation and/or a portion of the ground water underlying the site may contain gas in solution which originated from the underlying Puente Formation. During the pump test, there was a considerable drop in pressure

head near the well as water flowed into the pump. This pressure drop may have resulted in release of the gas into the well. Additional data would be required to confirm these concepts and delineate the problem. The engineering implications of these observations are discussed in Section 6.2

## 5.6 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

### 5.6.1 General

For purposes of our engineering evaluations and development of design recommendations, the geologic units at the site were grouped as fill, Young Alluvium, (upper sands, main gravelly sand unit, lower sand and boulder zone) and Puente Formation bedrock. This section includes engineering descriptions of each geologic unit and, in Table 5-1, presents the engineering parameters used in our analyses. These parameters are based on the laboratory test results (Appendix F), field test results (Appendices A, B and C), data from other investigations, and published data of observed and recorded field behavior on recent construction projects. Therefore, the parameters are based on factual data and engineering judgement.

TABLE 5-1

MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

| MATERIAL PROPERTY | FILL | YOUNG ALLUVIUM | PUENTE BEDROCK |
|---|---|---|---|
| Moist Density Above Ground Water (pcf) | 125 | 125 | 120 |
| Saturated Density (pcf) | - | 132 | 120 |
| Effective Stress Strength | | | |
| $\phi'$ (degrees) | 30 | 37 | 35 |
| c' (psf) | 0 | 0 | 0 |
| Total Stress Strength | | | |
| $\phi$ (degrees) | - | - | 0 |
| c (psf) | - | - | 5000 |
| Unconfined Compressive Strength (psf) | - | - | 10,000 |
| Permeability (cm/sec) | - | $3 \times 10^{-2}$ | $10^{-6}$ to $10^{-7}$ |
| Initial Tangent Modulus (psf) | $200 \cdot \sigma_v'$* | $350 \cdot \sigma_v'$* | $2 \times 10^{6}$ |
| Poissons Ratio | 0.3 | 0.35 | 0.35 |

* $\sigma_v'$ is the effective overburden pressure (psf) equal to effective density times over-burden depth. Moist density should be used to determine $\sigma_v'$ above the water table and submerged density (saturated density minus water density) was be used for the effective density of soils below the water table.

### 5.6.2 Fill

The fill soils were variable and included soil types such as gravel, clayey silts and silty sands with some building debris. Consistency of these materials is considered to be generally dense and stiff based on sampling resistance and density measurement. However, due to its variability and possible occurrences of soft/loose zones, relatively conservative strength parameters were used. Strength and modulus tests in these materials consisted

only of unconfined compression tests on the fine-grained soil samples and, therefore, engineering judgement was required in selecting the parameters. It was our opinion that drained (effective strength) properties should be utilized in design since these materials are generally above the water table and only partially saturated.

5.6.3 Young Alluvium

The Young Alluvium encountered consisted primarily of silt-sand-gravel mixtures, and these materials were generally classified as silty sand, sand and gravelly sand. Standard Penetration Test (SPT) results and laboratory densities indicate the Alluvium is dense to very dense. Much of these materials lie near or below the water table. Selected moist and saturated densities are presented in Table 5-1.

The hydraulic properties of the Young Alluvium were investigated by performing a pump test in a well located 5 feet east of Boring 5-1 (see Drawing 2). The testing procedures and test results are presented in Appendix B. The general hydraulic characteristics of the Young Alluvium determined on the basis of the tests results are as follows:

- Transmissivity: Computed to vary between 21,500 to 43,000 gpd/ft
- Storage Coefficient: Estimated to be about 0.1 to 0.15
- Boundaries: Boundaries were not observed during the pump test but may have been observed had the test duration been longer
- Saturated Thickness: Estimated to be about 65 feet
- Average Formation Permeability: Computed to be about $2 \times 10^{-2}$ to $4 \times 10^{-2}$ cm/sec. However, individual layers may have widely varying permeabilities. Laboratory tests on a silty sand sample indicate permeabilities less than $4 \times 10^{-4}$ cm/sec, and clayey interbeds are probably less than $1 \times 10^{-6}$ cm/sec.

Strength tests performed on the sand soils included both direct shear and triaxial compression tests. Considering the relatively high permeability of the sands, drained (effective) strength parameters are considered appropriate for static design. Effective stress strength data for granular alluvium is summarized in Figure F-1 of Appendix F. Figure F-1 includes test data obtained on similar soils from other Sections of the Rail Project.

Elastic properties for the sands were based on the laboratory triaxial and consolidation tests combined with published data and engineering judgement. The modulus data on soil samples from this Project Section and similar soil samples from other Rail Sections are summarized on Figure F-2 of Appendix F. The data indicate the modulus increases linearly with confinement. This characteristic is consistent with published data. The modulus value is presented in terms of the effective overburden pressure (effects of submergence must be considered in determining the effective overburden pressure).

### 5.6.4 Puente Formation Bedrock

The weak Puente Formation claystone and siltstone materials were considered to act as very stiff to hard highly overconsolidated fine-grained soils for the purpose of our engineering evaluations.

Bedrock elastic properties of Table 5-1 were based on field performance data, published data, and engineering judgement. A constant modulus value is considered appropriate for the highly overconsolidated bedrock materials. The selected constant modulus value presented in Table 5-1 is consistent with observed bedrock heave in the Los Angeles area (Evans, 1968).

The undrained strength parameters (0° friction and 5000 psf cohesion) were selected to represent the expected cohesive characteristics of the in situ bedrock. Measured permeability of intact samples of the bedrock was very low ($10^{-7}$ to $10^{-8}$ cm/sec). However, the mass permeability of this unit including effects of fractures, joints and sandstone beds would be higher. The permeability value of Table 5-1 is intended to represent the mass permeability of the bedrock.

# Section 6.0

# Geotechnical Evaluations and Design Criteria

## 6.0 GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA

### 6.1 GENERAL

Construction of the Station and Crossover structures will involve making a 60-foot deep excavation through predominately dense granular soil to some 30 feet below the static water table. This will involve a major dewatering effort and may require special construction provisions due to the occurrence of gas in the ground water. The permanent reinforced concrete structure will bear on and retain dense granular soil.

The primary geotechnical considerations at the site consist of:

- Construction dewatering and subsidence considerations;
- Design and construction of the temporary shoring system;
- Identifying construction problems and solutions relating to the possible production of gas and oil during dewatering;
- Recommendations for soil and water loads on the permanent structure;
- Earthquake design criteria.

### 6.2 EXCAVATION DEWATERING

#### 6.2.1 General

Assuming an excavation depth at Elevation 232, the proposed excavation will extend some 20 to 30 feet below the static ground water table (as measured in 1981 and 1983). Since the soils consist primarily of permeable sand and gravel, a dewatering system will be required. Section 5.6.3 summarizes the hydraulic characteristics of the soils to be dewatered. This section provides dewatering criteria, presents a possible dewatering scheme to aid in evaluating project feasibility, and identifies potential operating problems.

A deep slurry wall could be installed through the permeable soils into the underlying Puente Formation. The deep slurry wall would provide an effective cut-off and essentially stop ground water flow into the excavation. This would eliminate the need for an extensive dewatering system although excavation sumps would still be required initially to dispose of the ground water in the excavated soils. However, the slurry wall would need to extend some 30 to 40 feet below the minimum wall depth required simply for excavation support. In our opinion, the additional cost of the deep slurry wall would not be cost competitive with the required dewatering system. However, if properly designed and installed, the slurry wall would provide a technically acceptable alternative to dewatering.

As discussed in Section 5.5, gases including methane and hydrogen sulfide were released from the ground water during pumping. Significant volumes of gas released during dewatering could cause several problems including impacts on pump operation, offensive odors in the construction area and downstream along the discharge disposal route, and safety hazards. Several methods to mitigate these problems are discussed below including gas separaters at the pump, well venting, gas removal systems at the surface, and use of construction provisions applied to gassy formations. No oil was detected in the borings or during the pump test. However, the Union Station Oil Field, which reportedly produces oil in the Puente from very shallow depths, is located about 2000 feet to the south of the site. In addition, Boring CEG-2, located some 2000 feet to the east, encountered oil and gas at a depth 38 feet. Sustained dewatering conceivably could draw oil into the area and result in pumping oil.

The impacts of water quality, as discussed in Section 5.4, also need to be incorporated into the design, installation, and operating procedures.

### 6.2.2 Criteria for Dewatering Systems

Where the contractor will be responsible for designing, installing, and operating a suitable construction dewatering system, it will be subject to review and acceptance by the Metro Rail Transit Consultants (MRTC). The system should satisfy the following criteria:

- The dewatering system should be installed and in operation for a sufficient period prior to the excavation reaching the level of static ground water level to adequately drawdown the ground water table. This period depends on the maximum installed pumping capacity of the system but could be several months.

- The system should maintain the ground water levels at least 2 feet below the lowest excavation level in the center and 4 feet along the excavation perimeter.

- The system should be operated continuously. Emergency power and backup pumps should be required to insure continual excavation dewatering. This is essential at the Union Station site since even a temporary disruption in dewatering could result in rapid flooding of the excavation and the possible development of localized "quick" conditions.

- The contractor should be responsible for disposing of the well discharge. He should be made aware of the potential environmental and operational problems caused by noise, poor quality ground water, strong gas odors from dissolved gases, and the possibility of pumping oil. He should be made responsible for resolving these potential problems and satisfying applicable codes and ordinances.

- The wells must be designed and developed to eliminate loss of ground due to piping. The well operations should be constantly monitored for evidence of piping.

### 6.2.3 Preliminary Analysis of a Dewatering Installation

It is not the intent of this report to recommend a specific dewatering system. However, to evaluate feasibility, a possible dewatering scheme was developed. It is believed that an appropriate dewatering design would include about ten 24-inch diameter wells penetrating into the Puente Formation. Penetration of the wells into the Puente will maximize the capacity of the wells. This will require drilling the wells through the boulder zone. Alternatively, more wells could be installed with the wells terminated above the boulder zone. The test well discussed in Appendix B was drilled through the boulder zone. The wells would be located outside the proposed excavation and spaced at about equal intervals around the excavation. The wells should be perforated throughout the saturated zone with a bottom blank section to minimize piping of the fine-grained bedrock. It is estimated that the maximum drawdown at the well would have to be on the order of 40 feet. The initial total pumping rate might be on the order of 2000 to 4000 gpm, depending on how quickly it was necessary to drawdown the water table. The long term "steady state" maintenance total flow would be about 750 to 1500 gpm.

As discussed above, dewatering is expected to release gas and possibly oil. Additional subsurface information obtained prior to construction and/or monitoring data obtained during dewatering would be required to determine their occurrence, composition and concentrations. Based on the available data, gas may have to be separated from the water before disposal. There are methods available for separating gas from liquid at the pump inlet. One method involves the installation of a gas shroud on the pump to separate much of the gas and prevent air-locking of the pump. Special construction procedures may also be required due to accumulation of potentially explosive gases in the excavation and wells.

### 6.2.4 Subsidence

A major dewatering program operated for the extended construction period will lower the ground water table over a relatively large area. Depending on the dewatering system used, it is estimated that the drawdown will be on the order of 20 feet at a distance of 500 feet, 15 feet at a distance of 1000 feet, and 5 feet at a distance of 2000 feet from the station. This drawdown will increase the effective stress in the subsurface formations and, theoretically, result in some surface settlement. Assuming that the subsurface conditions within 1000 feet of the site are similar to those encountered in the borings, the theoretical settlement caused by dewatering was computed. These calculations indicate that the surface settlement would be less than 1/2 inch for 20 feet of drawdown and less than 1/5 inch for 5 feet of drawdown. The ground would rebound to essentially its original elevations after dewatering is terminated as the effective stresses reduce to their original levels. In addition, the subsidence slope would be very gentle with differential settlement along any single structure being considerably less than the total.

A well located immediately adjacent to the existing pedestrian tunnel could induce a ground subsidence on the order of 3/4 inch. However, we believe that the angular distortion of the tunnel due to this subsidence would be small, on the order of 1/4 inch over a 50-foot distance. To minimize this problem, it

may be appropriate to locate dewatering wells at least 50 feet away from the existing tunnel.

Due to the proximity to existing facilities (buildings, pedestrian tunnel, rails, streets, utilities, etc.) the dewatering system must be designed and maintained to minimize loss of ground due to piping. Loss of ground due to piping could lead to ground subsidence, particularly near the wells. The existing pedestrian tunnel would be particularly sensitive to this type of problem.

## 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. Both the existing Post Office and Union Station buildings are located beyond the 1H:1V guideline shown on Figure 6-1. However, the contractor must consider potential problems associated with maintaining the existing pedestrian tunnel due to dewatering and excavation. Anticipated ground movements adjacent to the excavation are discussed in Sections 6.2.4 and 6.4.6.

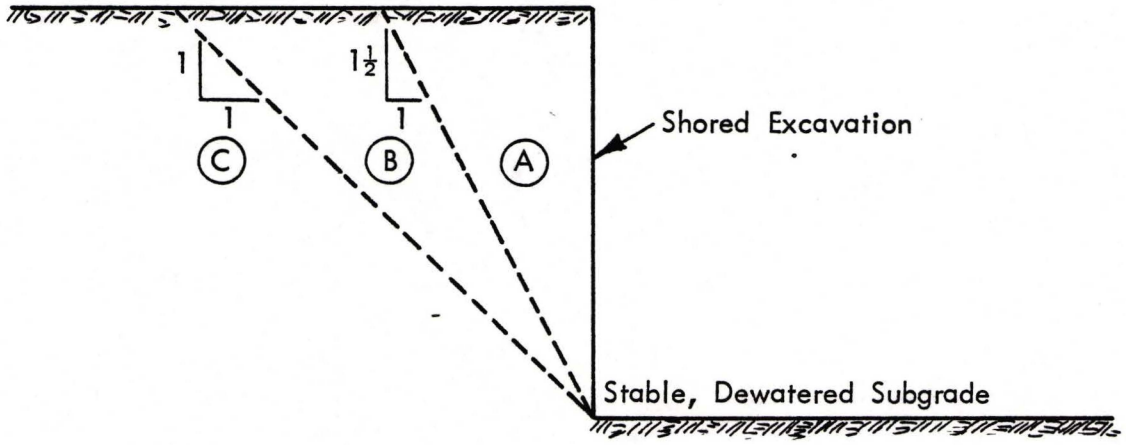
If underpinning is proposed, careful consideration should be given to the method used due to the high caving potential of the gravelly subsoils at the site.

## 6.4 TEMPORARY EXCAVATIONS

### 6.4.1 General

The required excavation will extend some 60 feet below the existing Rail Yard and some 46 feet below the lower area to the west. We understand that the excavation system will be chosen and designed by the contractor in accordance with specified criteria and subject to the review and acceptance by the SCRTD.

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 pertains to this specific excavation method. Other systems may also be appropriate and should be considered by the contractor. In our opinion, driven sheet piles are not considered feasible since the composition of the subsoils would make driving difficult if not impossible. Installation of drilled piles and tiebacks may also be difficult due to caving of the coarse granular subsoils.




NOTE: These guidelines are applicable only for stiff or dense stable ground conditions.

(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.

(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.

(C) No Special Provisions

Approved for publication 4-12-83 by PHP

## UNDERPINNING GUIDELINES

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Figure No. 6-1

Converse Consultants Geotechnical Engineering and Applied Sciences

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### 6.4.2 Sloped Excavations

Portions of the required excavation could be made with a sloped excavation, particularly the shallower cuts at the entry structures. To minimize slope instability and potential loss of ground due to occurrences of perched water which may persist after dewatering, sloped excavations should be limited to depths above the static ground water table. A sloped excavation seems suitable at this site provided easements can be obtained to remove additional width of track. Sloped excavations extending to the static ground water table would reduce the height of the temporary shoring by 50%. Construction of a wide bench at the toe of the cut slope would provide access to the shored excavation but would increase the volume of excavated soil.

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, ground water conditions, the weather (i.e., dry or heavy rain), construction procedures and scheduling. Applicable governmental safety codes must also be complied with (refer to CALOSHA Article 6, 1540 d).

Based on previous experience in similar soils, temporary construction slopes of 1.5H:1V through the fill and upper silty sands and 1H:1V through the dense gravelly sands would probably be suitable. This assumes suitable site dewatering, no heavy loads at the top of the slope, protection of the slope surface and some slope maintenance. 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 predrilled holes and braced with tiebacks or internal struts is a common method of shoring deep excavations in the Los Angeles area. Appendix G.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 granular soils similar to those encountered at the proposed Union Station site. In these types of soils, particularly below the ground water table, caving can be a problem. This may be more pronounced at the Union Station site due to the gravelly composition of the subsoils and the occurrences of cobbles and boulders. These conditions have been successfully resolved by maintaining a head of water or slurry in the hole.

The granular soils to be supported at the Union 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. At the Union Station site the granular soils may be unsuitable for guniting due to sloughing, particularly below the static ground water table.

### 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 there is not a significant difference between the maximum movement of properly designed and constructed internally 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 an internally 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 competent 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 anchors, high pressure grouted anchors, high pressure regroutable anchors, and others. Generally in the Los Angeles area, high capacity straight shaft anchors have been used. Due to the gravelly composition of the subsoils and the occurrences of cobbles and boulders, there may be tieback installation problems requiring use of casing, slurry, hollow stem auger and/or other procedures to maintain hole stability and minimize caving. Belled anchors are considered impractical at the Union Station site due to the potential caving problems. Due to the granular nature of the soils underlying the Union Station site, belled anchors are not appropriate. Driven, high pressure grouted anchors may be a feasible alternative but there is little experience with such anchors in the Los Angeles area, and anchor capacities would need to be demonstrated in the field.

### 6.4.5 Design Criteria

This section provides design criteria for a shoring system consisting of soldier piles, wooden lagging, and tiebacks. The soldier piles are assumed to consist of steel W- or H-sections installed in predrilled circular holes. It is assumed that the drilled hole will be filled with lean mix concrete. Thus for computing the allowable vertical and horizontal pile loads on the soil, a circular concrete section was assumed.

Specific shoring design criteria include:

° Design Wall Pressure: Figure 6-2 presents the recommended design lateral earth pressure on the temporary walls. The figure includes the case of full depth shoring and the case of partial sloped cuts. The full loading diagram should be used to determine the design loads on tieback anchors or struts and the required depth of embedment of the soldier piles. For computing design stresses in the soldier piles, the computed values can be multiplied by 0.8. For sizing lagging, the earth pressures can be reduced by a factor of 0.5.

The recommended design static lateral earth pressures are based on standard design practices and experience in the Los Angeles area. Appendix G.1.5 summarizes the design shoring pressures used on eight typical projects in the Los Angeles area. Several of these involved excavation into Young Alluvium, similar to but not as coarse-grained as the Alluvium underlying Union Station.

Appendix G.2 and Section 6.11.2 provide technical support for the recommended design seismic pressures.

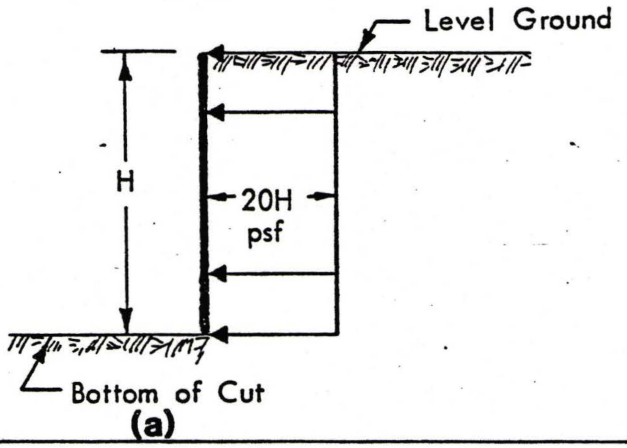
° Depth of Soldier Piles: The depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to safely carry both the lateral and vertical loads under static and dynamic loading conditions. The vertical loads on tieback-supported soldier piles can be substantial. In general the vertical load is equal to the imposed vertical load on top of the soldier pile plus the vertical component of tieback anchor loads (if used). For example, anchors installed at 20° below the horizontal will develop a vertical load on the soldier pile equal to about 35% of the total anchor tension plus any superimposed loads.

The required depth of embedment to safely carry vertical loading should be computed based on both end bearing resistance and shaft friction of the soldier pile penetrating the dense gravelly sands. Allowable vertical loads for several typical pile diameters are shown on Figure 6-3.

The imposed lateral load on the embedded portion of the soldier pile should be computed based on the contributing area of the earth pressure envelope diagrams (Figures 6-2a to 6-2d, and 6-2f) below the lowest tieback or strut. The allowable passive resistance developed by the pile should be based on the passive pressure minus the active pressure below the bottom of the excavation, as shown on Figure 6-2e. 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, whichever is less. Figure 6-2 indicates the recommended method to compute passive resistance.

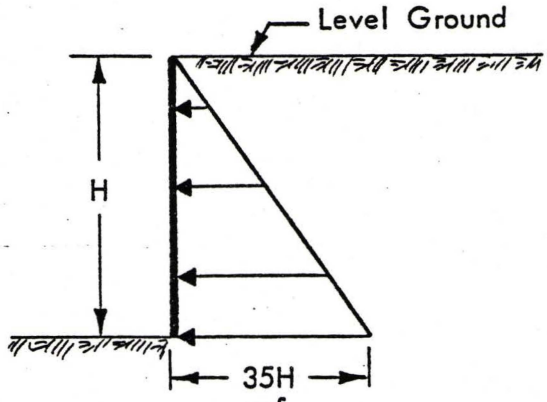
The required depth to safely carry both vertical and lateral loads should be compared with the greatest value controlling the design.

## BRACED WALL



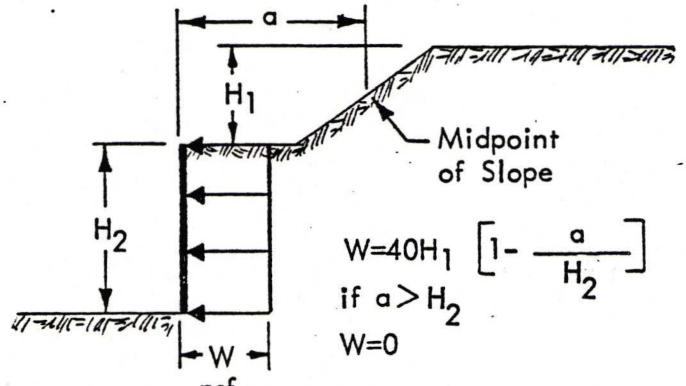

(a)

## CANTILEVER WALL




(b)

## DUE TO BENCH EXCAVATION

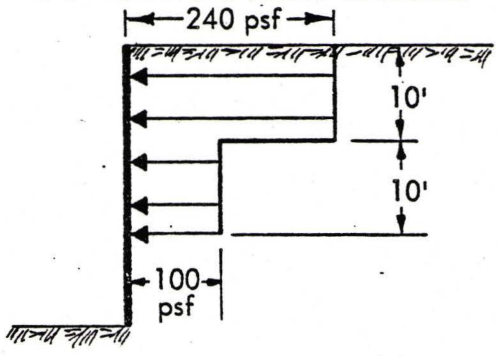



$W = 40H_1 \left[1 - \frac{a}{H_2}\right]$ if $a > H_2$

$W = 0$

(c)

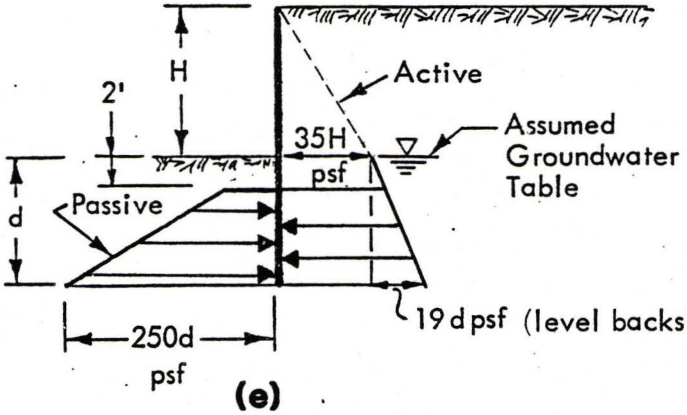
## CONSTRUCTION LOADS




(d)

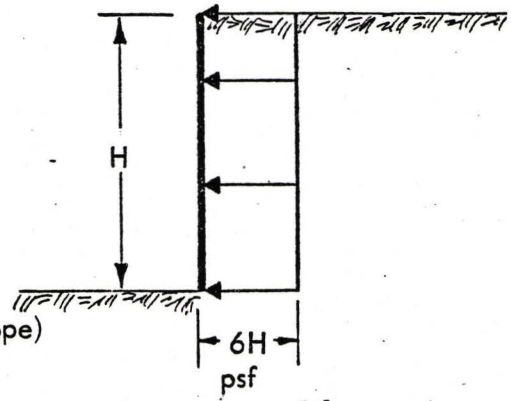
## PASSIVE RESISTANCE

NOTE: Passive acts on 1.5 times the pile diameter or $\frac{1}{2}$ pile spacing (which ever is less).




(e)

## EARTHQUAKE LOAD




(f)

# LOADS ON TEMPORARY FLEXIBLE WALLS (WITH DEWATERING)

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Figure No. 6-2

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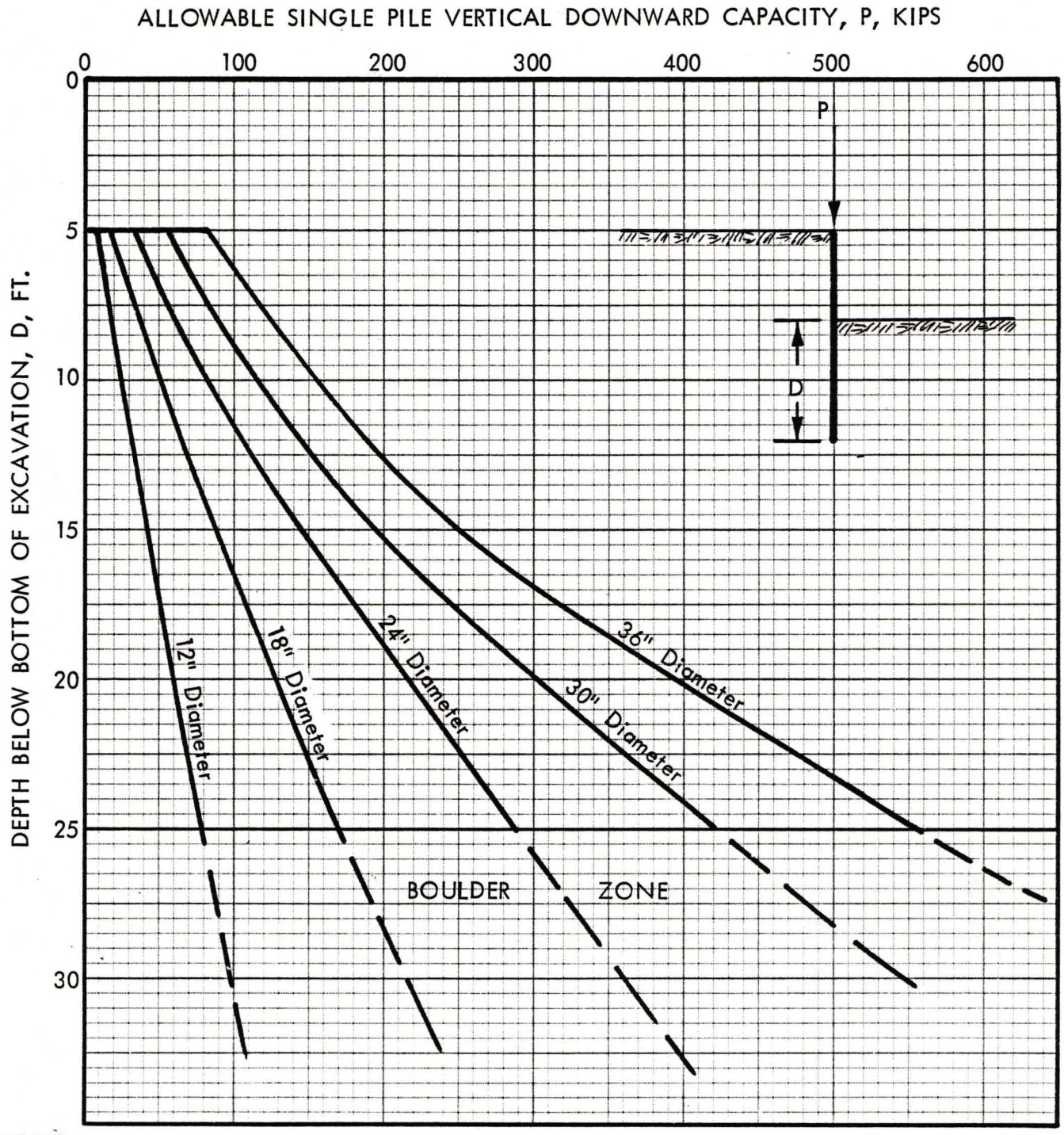



NOTES:

1.) Bottom of excavation assumed at elev. 232 ft.

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

3.) Proper excavation clean-out assumed for End Bearing Support.

Approved for publication by 9/23/83

# VERTICAL CAPACITY OF SOLDIER PILES

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
6-3

° Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil loads, member sizes and costs, anchor load, and the ability for soils to arch. At the Union Station site, the soils are granular and may contain pockets of water not fully drained by the construction dewatering system. Thus, it is recommended that the pile spacing be limited to about 6 feet and that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles. Use of geotextiles and/or limiting temporary exposed soil height should be implemented by the contractor to control ravelling problems.

° Tieback Anchor Design: Tieback anchor capacity can be determined only in the field based on anchor load tests. For estimating purposes we recommend that the allowable design capacity of straight shaft friction anchors installed in drilled shafts be computed based on the following equation:

$$P = \pi DLq$$

where:

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

The average soil resistance (q) can be assumed equal to:

$$q = 20d \leq 1000 \text{ psf}$$

where:

d = average depth of the anchor beyond the potential failure plane; measured vertically from the ground surface.

For design purposes, the potential failure surface behind the shored excavation should be determined by a plane drawn at 45° with the vertical through the bottom of the excavation. Only the frictional resistance developed beyond the assumed failure plane should be assumed effective in resisting lateral loads. In addition no resistance should be assumed within the fill. This criteria may lengthen some of the upper anchors in the Rail Yard area.

The anchors may be installed at angles between 20° to 50° below the horizontal. The contractor should be required to use a tieback anchor installation method which will minimize loss of ground due to caving. Uncontrolled caving not only causes installation problems but could result in surface subsidence of the Rail Yard. To minimize caving, casing should be installed as the hole is advanced but must be pulled as

the concrete is poured. Alternatively, a hollow stem auger could be used. Structural concrete should be placed in the lower portion of the drilled shaft up to the assumed failure plane. Pouring of the anchors should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the failure plane and the face of the shoring must be backfilled with a sand slurry or equivalent after concrete placement.

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. At 150% of the design 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.

- Internal 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:

- 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).

- Provide 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.

- Provide 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.

- Increase the load carrying capacity of the struts (larger members and/or intermediate supports) such that the bracing can safely support the maximum anticipated temperature-induced loads combined with the earth pressure loads.

### 6.4.6 Anticipated Ground Movements

Based on review of published data on shoring performance, we believe that the maximum horizontal deflection for a properly designed and installed wall will be equal to about 0.1% to 0.2% of the excavation depth. Thus for the proposed 46- to 60-foot deep excavation, the maximum wall deflection should be about ½ to 1½ inches.

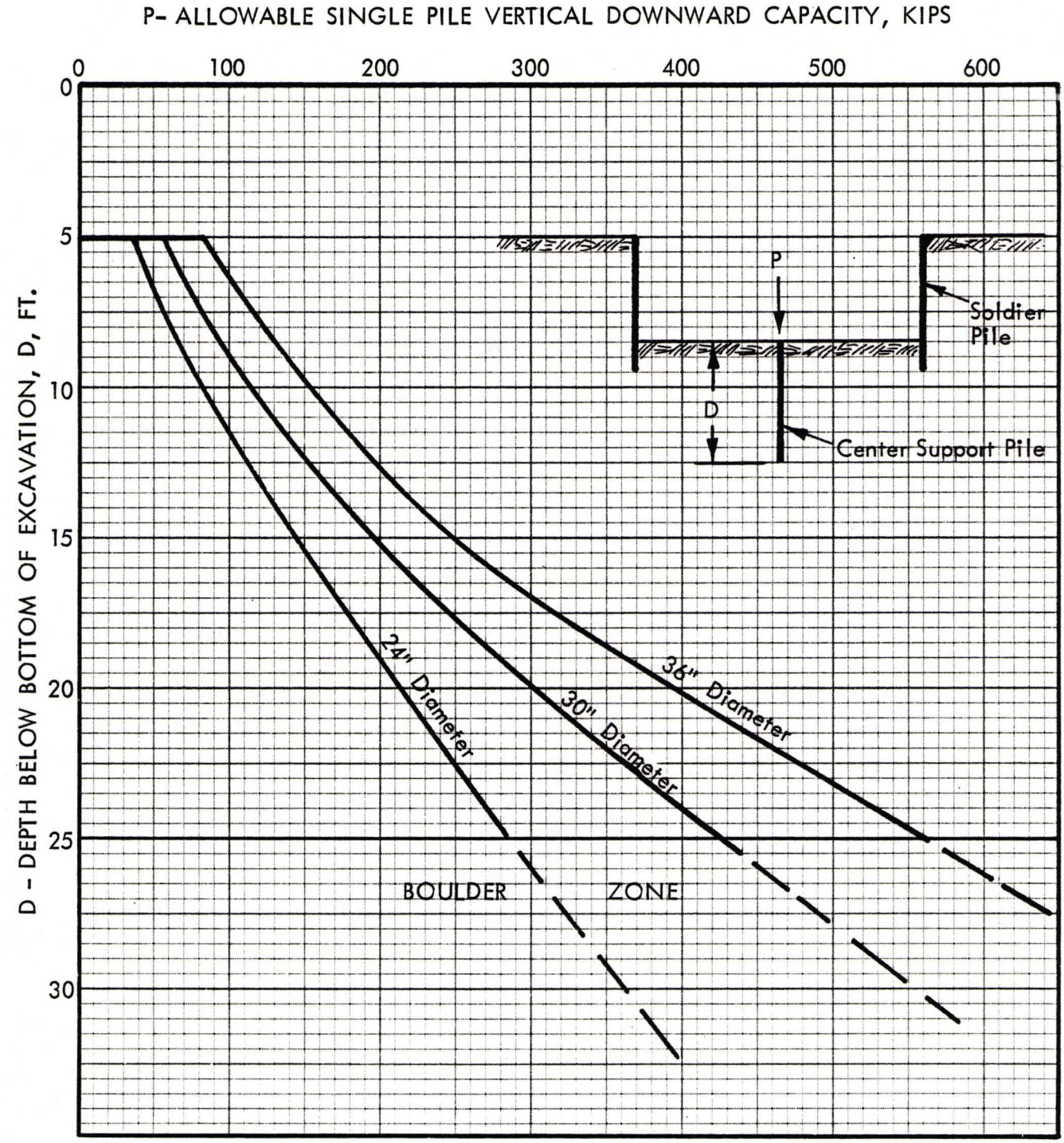
The maximum horizontal movement should occur near the top of the wall and decrease with depth if tiebacks are used. The maximum horizontal movement should occur near the base of a cross-lot braced shoring wall. The maximum settlement behind the wall should be equal to about 50% to 100% of the maximum 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

Depending on the Rail Yard operation requirements, it may be necessary to construct a temporary decking over portions of the shored excavations. To minimize the deck span, it may be desirable to install piles in the center of the excavation. Piles would need to extend below the maximum proposed excavation level for support. The natural dense granular soils encountered below the proposed excavation level are suitable for supporting pile loads. To minimize installation problems, piles should be designed to develop the required capacity at depths above the bouldery zone encountered in the borings. Thus, the tip elevation should be above about Elevation 210.

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that he may use similar piles to support the center decking. Accordingly, we evaluated the allowable loads on these types of piles for several typical diameters. The recommended allowable design loads are shown on Figure 6-4. 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 occurrences of cobbles and boulders within the alluvium, 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 settlements in the soil due to driving vibrations, particularly within the fill layer. Thus we believe that driven piles would probably not be used. Accordingly, we have not developed design loads for driven piles.




NOTES:

1.) Bottom of excavation assumed at elev. 232 ft.

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

3.) Proper excavation clean-out assumed for End Bearing Support.

# VERTICAL CAPACITY OF CENTER SUPPORT DRILLED PILES

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
6-4

Converse Consultants Geotechnical Engineering and Applied Sciences

Approved for publication by 9/23/83

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## 6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed excavation at the Union Station and Crossover Structure 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:

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

° Surface Survey Control: It is recommended that several locations around the excavation and on any nearby structures be surveyed prior to any construction activity and then periodically 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.

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

  We recommend that 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.

° Inclinometers: 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.

- 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 measurement of the instrumentation devices be under the direction and control of the Engineer. Experience has shown when the instrumentation program has been included in the bid package as a furnish and install item, the quality of the work has often been inadequate such that the data is 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 60-foot excavation (assuming 30 feet above the GWT and 30 feet below the GWT) will decrease the vertical ground stresses by about 6000 psf. This stress reduction will cause the soils below the excavation to rebound or swell. This response is not due to the occurrence of any swelling type of soils but simply the response to stress unloading. In addition, even with a suitable shoring system, shear stresses

will develop tending to cause the soil adjacent to the excavation to heave into the excavation. The net effect will be to cause the bottom of the excavation to heave or deform upward. 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 soil. We estimate that the Station and Crossover loads will be about 4000 to 5000 psf. This load will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil exceeds the weight of the final structure, the structure will cause ground settlement.

We estimate that the maximum heave at the center of the excavation will be on the order of 1½ to 3 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. Due to the dense granular composition of the alluvium and consistency of the bedrock, the majority of the heave deformation will be elastic rebound. Monitoring of the actual heave is recommended as discussed in 6.6.

It was computed that the imposed loads from the structure and backfill will induce settlements on the order of 1 to 2 inches. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/3 inch over half the structure width. This correlates to an angular rotation of only about 1:1200. 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. The anticipated differential settlements and distribution of the bottom slab bearing pressures could be estimated based on a soil-structure interaction analysis; however, such an analysis is beyond the scope of this study.

## 6.8 FOUNDATION SYSTEMS FOR PERMANENT STRUCTURES

### 6.8.1 Main Station and Crossover Structures

It is understood that the proposed Union 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 dense sand and gravels. Locally interbeds of dense silty sands and/or stiff clay may be encountered at the foundation level. We believe these interbeds will be too limited in extent to affect the behavior of the foundation. To minimize construction related problems, it may be advantageous to excavate any zones of clay encountered at the foundation level. We understand that the average foundation bearing pressure will be about 4000 to 5000 psf for both the Union Station and the Crossover Structure. After construction, the ground water table will recover to the original static levels. This will impose hydrostatic uplift pressures on the bottom slab equal to about 1750 to 2000 psf. Thus, the permanent average bearing pressures on the soil will only be about 2000 to 3000 psf. In our opinion the station and crossover structure can be

adequately supported on the dense natural soils with minimal settlements. Section 6.7 presents estimated settlements for the proposed structures.

### 6.8.2 Support of Surface Structures

Major surface structures should not be supported on the existing fills. Existing fill should be excavated from below the structures and backfilled with compacted structural fill for structural support. Figure 6-5 presents recommended maximum bearing pressures and anticipated settlements for footings bearing on either dense, undisturbed granular alluvium or properly compacted structural fill. The figure is 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%.

In the Rail Yard area, the depth of fill is about 12 to 14 feet. As an alternative to excavating the fill for support of shallow footings, the structures could be supported on drilled piles or piers penetrating into the underlying Young Alluvium. Figure 6-4 can be used to determine the allowable vertical loads on drilled piers using the depth of penetration into the undisturbed alluvium.

Minor lightly loaded structures (such as light standards, signs, etc.) may be supported on the existing fills. The footing bearing area should be over-excavated by at least 12 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, the passive pressure can be computed based on an equivalent fluid density of 250 pcf, and a base friction value of 0.4 can be assumed.

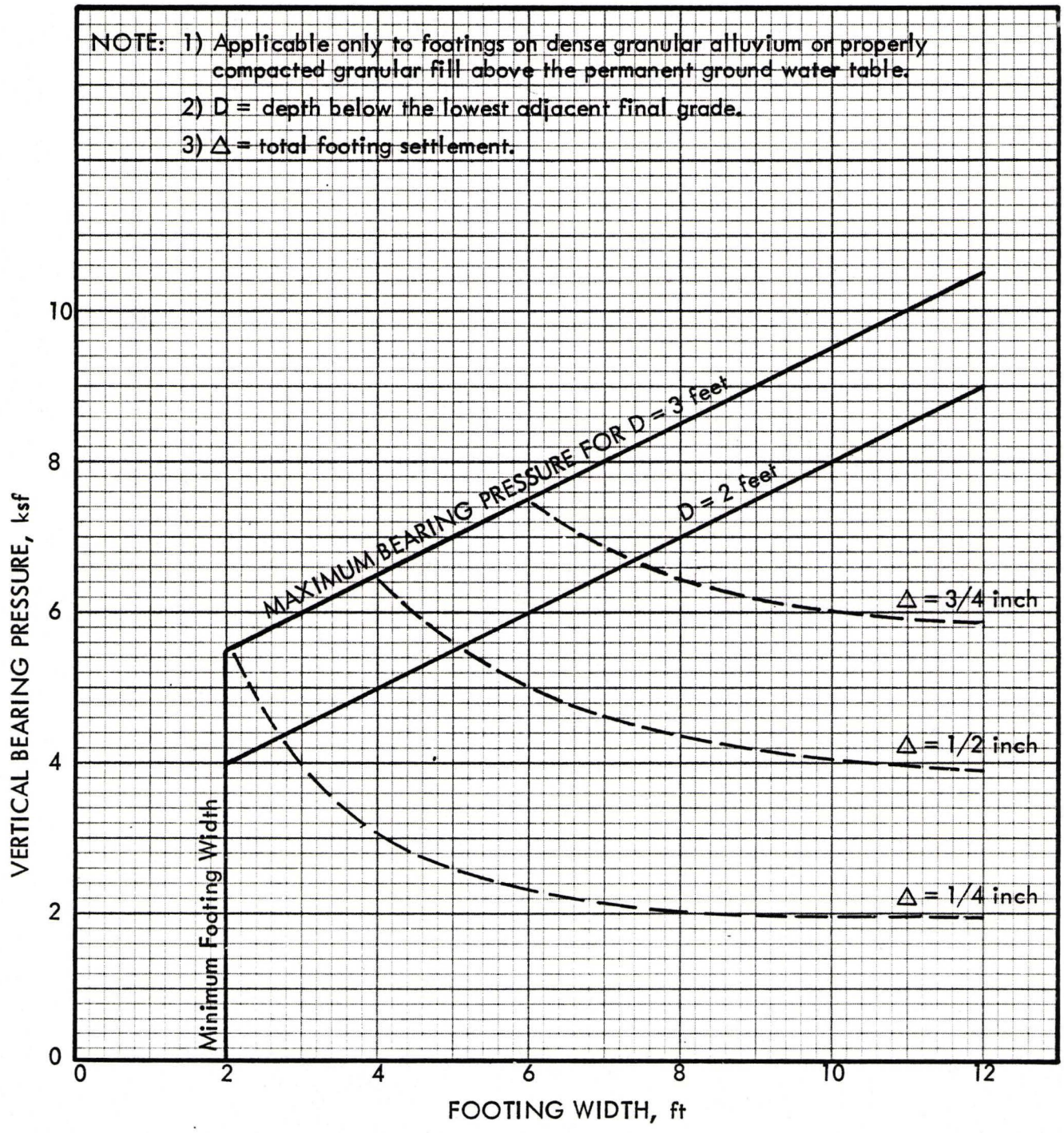
## 6.9 LOADS ON PERMANENT SLAB AND WALLS

### 6.9.1 Hydrostatic Pressures

The ground water table varies across the site and appears to range from about Elevation 262 at the west end of the Crossover to about Elevation 255 at the east end of the Station, as measured during 1981-1983. The winter of 1983 was one of the five wettest years in the past 100 years. We recommend that the following maximum ground water levels be assumed for design:

| Location | Elevation | Hydrostatic Pressure Uplift On the Slab* (psf) |
|---|---|---|
| West End of Crossover Structure | 265 | 2060 |
| East End of Station | 260 | 1750 |

*Assuming bottom of slab at Elevation 232.

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# SURFACE STRUCTURE SPREAD FOOTING DESIGN

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No.
6-5

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

### 6.9.2 Permanent Static 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 50 pcf above the ground water table and 30 pcf below the ground water table.

The pressures on the roof should be assumed equal to the full weight of the overburden soil plus surcharge for static loading conditions. In addition, surcharge loads due to surface rail or vehicular traffic should be considered as well as loads due to earthmoving equipment during station backfill operations.

### 6.9.3 Seismic Earth Pressures

Based on the discussions in Section 6.11.3 and the analysis presented in Appendix G.2, an equivalent rectangular pressure distribution of 8 times the depth of the structure is recommended.

### 6.9.4 Recommended Earth Pressure Diagrams

Figure 6-6 presents recommended design earth pressure diagrams.

## 6.10 PERMANENT DRAINAGE CRITERIA

We understand that the Station and Crossover Structure will be water-tight below the maximum anticipated static ground water table.

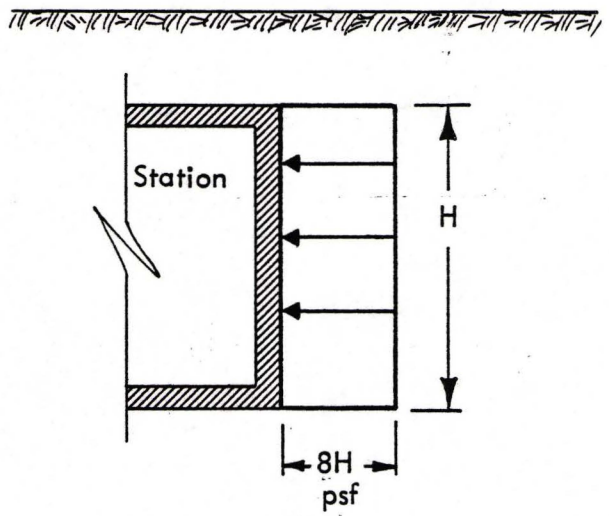
## 6.11 SEISMIC DESIGN CRITERIA

### 6.11.1 General

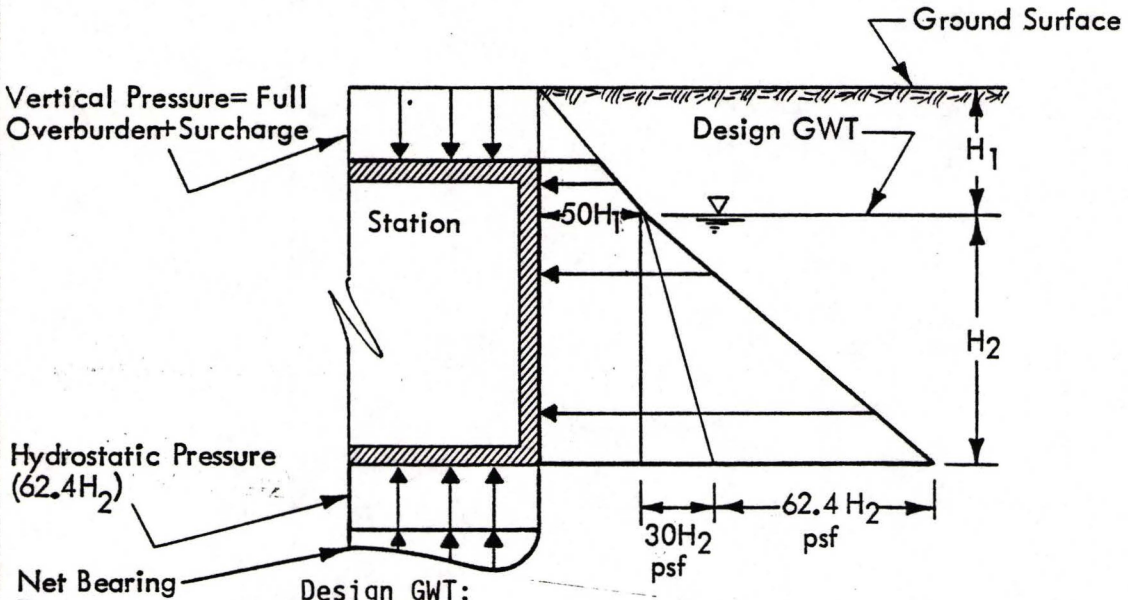
Detailed seismic criteria for design of the Southern California Rapid Transit Metro Rail Project have been previously developed and are presented in the "Seismological Investigation and Design Criteria" report dated May, 1983. The Part I investigation of this report contains an evaluation of the seismological conditions which may affect the project, and selection of 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

## EARTHQUAKE PRESSURE




## STATIC PRESSURE




Design GWT:

1. End of Construction: Below Floor Slab

2. Maximum Anticipated:
   - West End of Crossover Structure: El 265
   - East End of Station Structure: El 260
   - Locations Between: Interpolate

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## LOADS ON PERMANENT WALLS

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Figure No. 6-6

function required to maintain public safety, preventing catastrophic failure and loss of life. Design ground motion parameters for these two earthquakes are presented on Table 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 or 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 required that the variation of dynamic shear modulus and soil hysteretic damping with the level of shear strain be known.

Average values of compression and shear wave velocities based on interpretation of limited downhole and crosshole geophysical surveys performed in Boring CEG-5 and other borings in similar materials during the 1981 investigation (see Appendix C) are presented at the top of Table 6-1. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish appropriate modulus values at low strain levels. Computed moduli values for the Young Alluvium and Puente bedrock are tabulated in Table 6-1.

TABLE 6-1

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

| | | YOUNG ALLUVIUM | PUENTE BEDROCK |
|---|---|---|---|
| Average Compression Wave Velocity, $V_c$ (ft/sec) | | 5300 | 5700 |
| Average Shear Wave Velocity, $V_s$ (ft/sec) | | 1100 | 1300 |
| *Poisson's Ratio | | 0.35 | 0.35 |
| **Young's Modulus, E, (psi) | - moist | 474,000 | 530,000 |
| | - saturated | 211,000 | |
| **Constrained Modulus, $E_c$, (psi) | - moist | 760,000 | 850,000 |
| | - saturated | 800,000 | |
| **Shear Modulus, $G_{max}$, (psi) | - moist | 32,600 | 45,000 |
| | - saturated | 34,500 | |

* For saturated Young Alluvium, use value of 0.45.

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

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 G.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 judgement.

Based on the analysis presented in Appendix G.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. This value is based on a peak ground acceleration of 0.30g corresponding to the Operating Design Earthquake (ODE).

### 6.11.4 Liquefaction Potential

The site is underlain by 50 to 60 feet of saturated granular alluvium. Since saturated granular soils can be susceptible to liquefaction, a liquefaction analysis of the site was performed using data obtained from the 1981 and 1983 investigations. The details of the analysis are presented in Appendix G.3. The analysis was based on:

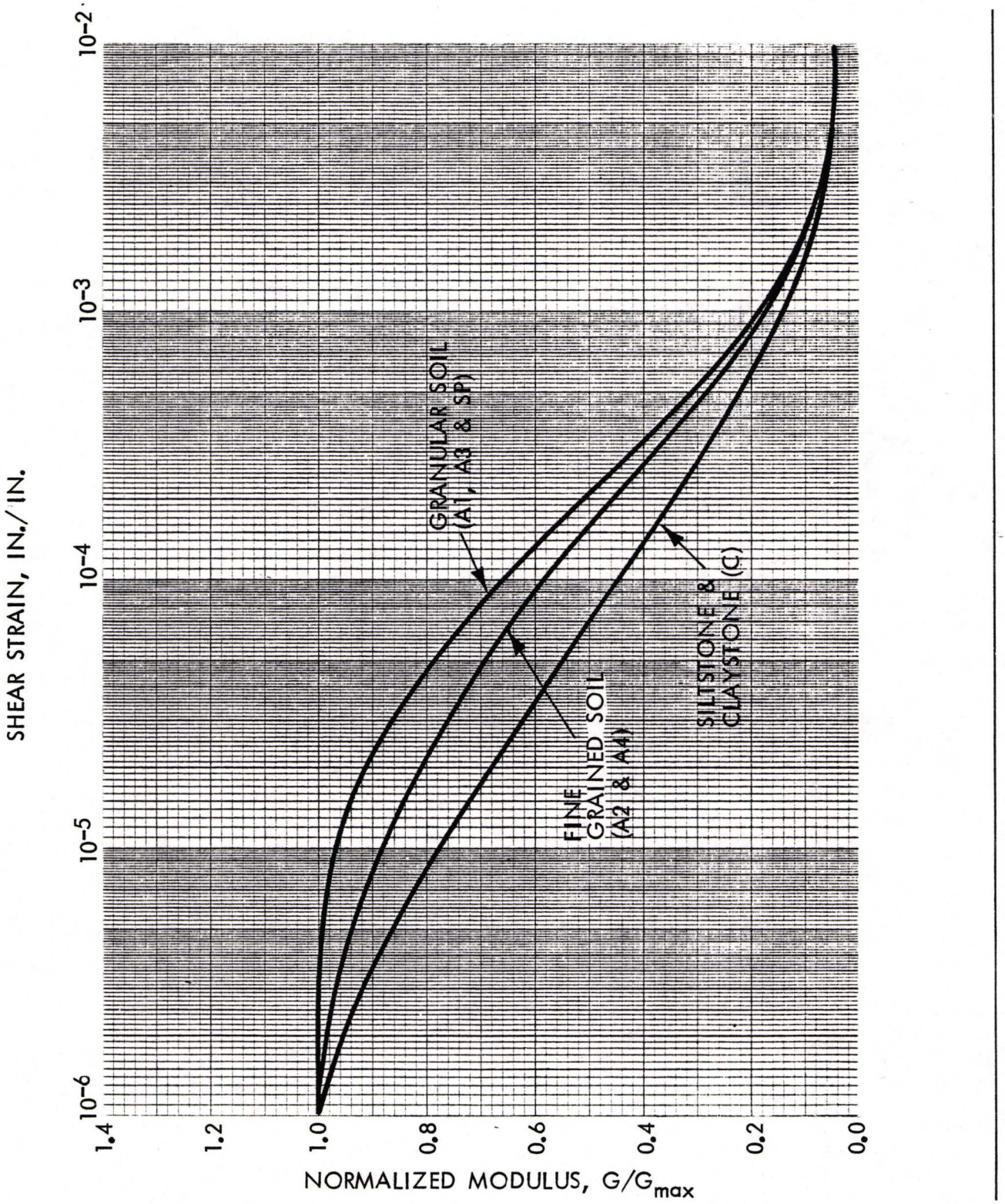
- Recent procedures developed by Seed et al., 1983, which relate observed field behavior with common soil properties. The main soil properties examined at this site included Standard Penetration Test (SPT) resistances, field shear wave velocity measurements and gradation.

- Observations made in man-sized Boring 6A.

The analysis indicated that the alluvium has a low risk of liquefaction due to its relatively high density and coarse gradation. Based on the results of the analysis presented in Appendix G.3 and engineering judgement, it is our opinion that the alluvium underlying the site would not be subject to liquefaction during strong ground shaking produced at the site by the postulated earthquake motions.

## 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,

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## RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Southern California Rapid Transit District
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83-1101

Figure No.
6-7

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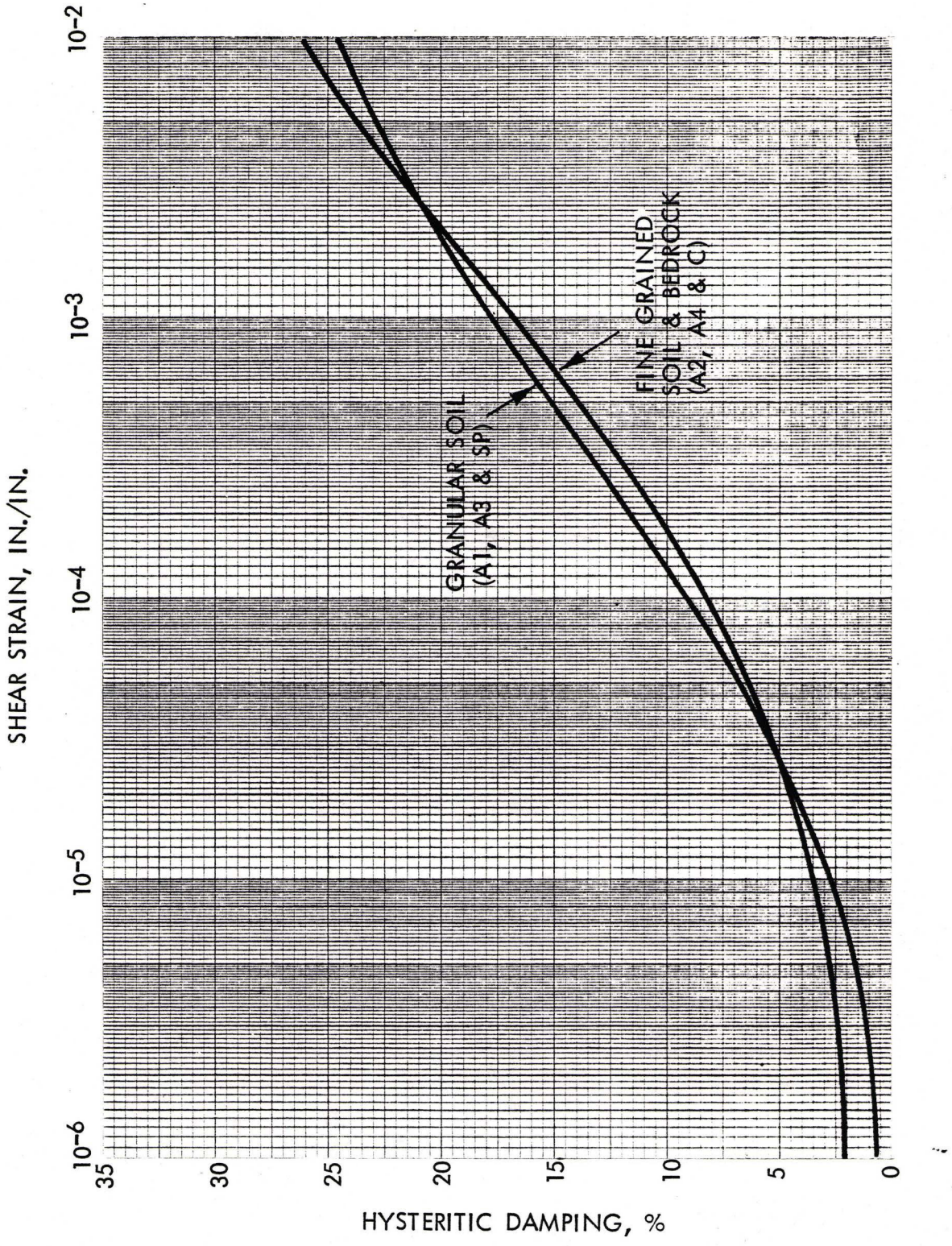


# RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
6-8

**Converse Consultants** Geotechnical Engineering and Applied Sciences

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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. Excavated onsite granular alluvium is considered suitable for use as backfill material provided the moisture content is adjusted as necessary for proper compaction to the required densities. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill are presented in Appendix H. Recommended specifications for compaction of fill are also presented in Appendix H. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

## 6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

- Observation Well Monitoring: 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.

- Additional Oil and Gas Explorations: 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.

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

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

- Construction Observations: A qualified geotechnical engineer should be on site full time during installation of the 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.

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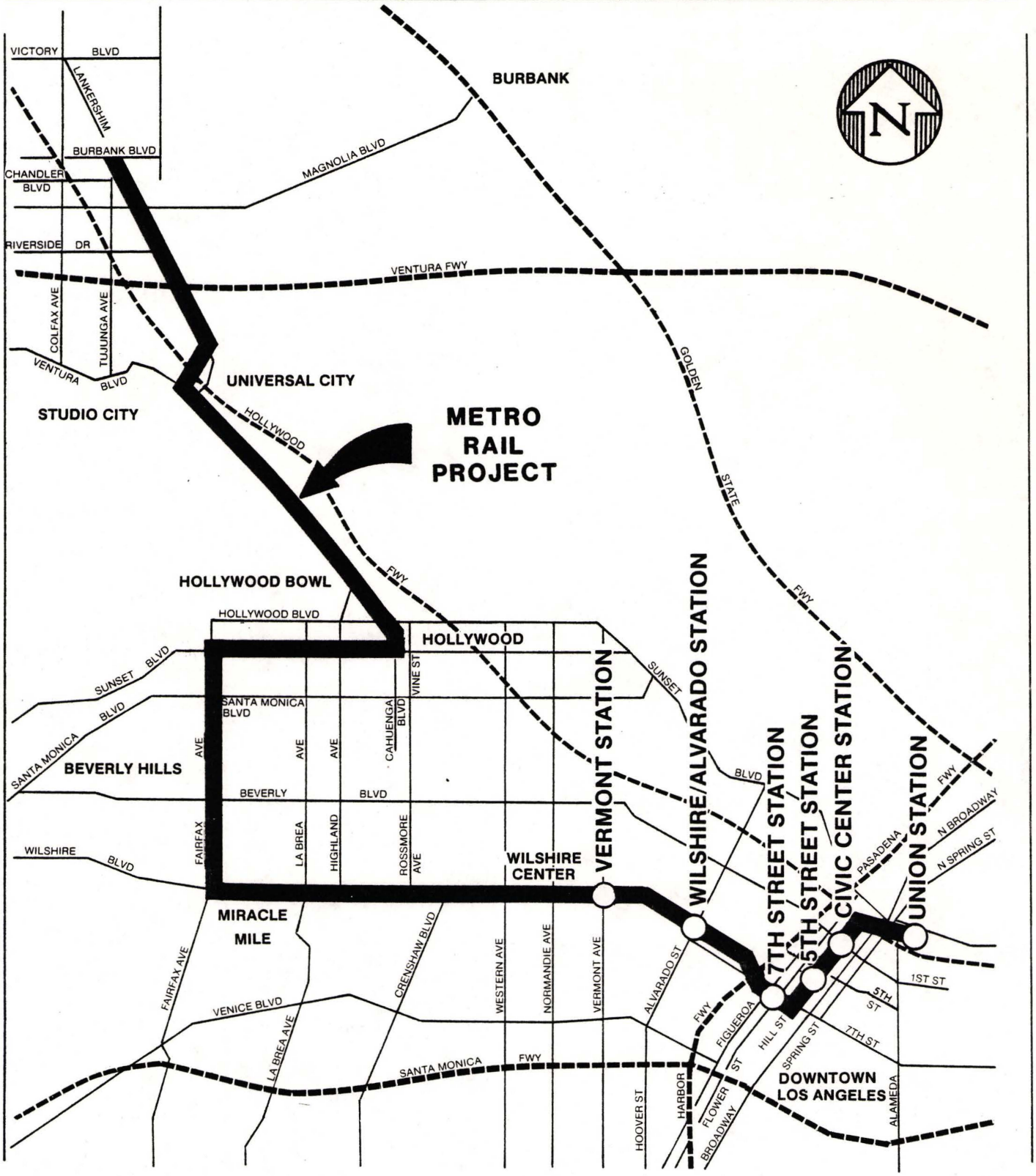
# Drawings




# VICINITY MAP

**DESIGN UNIT A135 - UNION STATION**
**Southern California Rapid Transit District**
**METRO RAIL PROJECT**

Project No.
**83-1101**

Drawing No.
**1**

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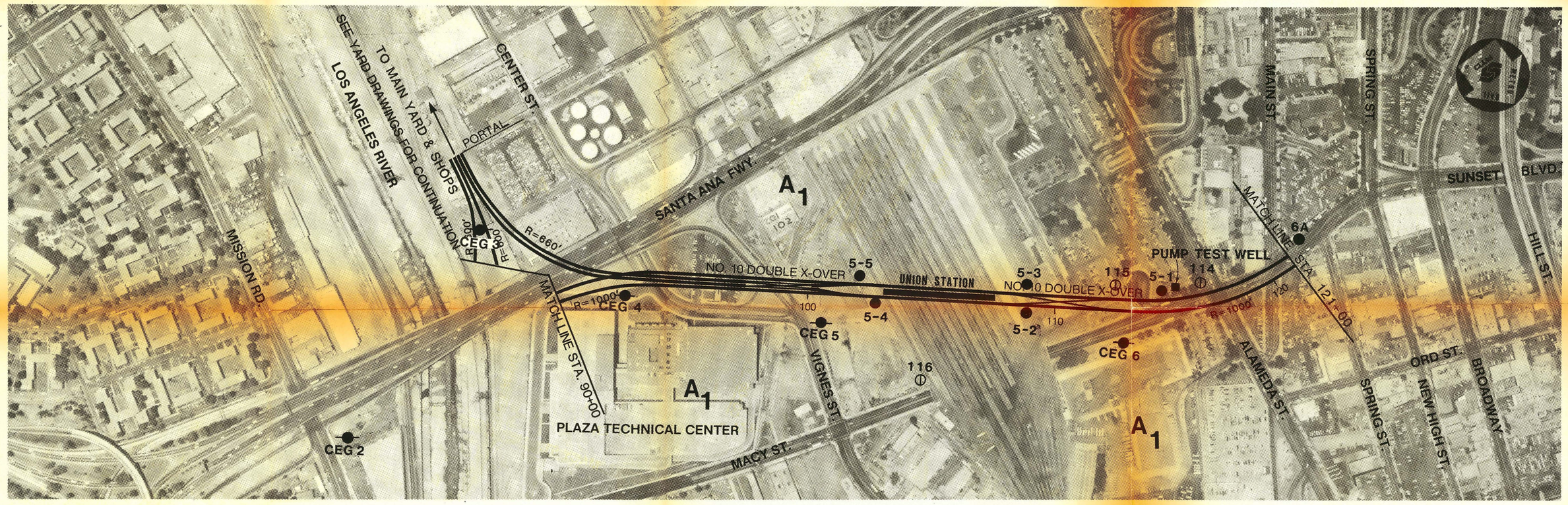
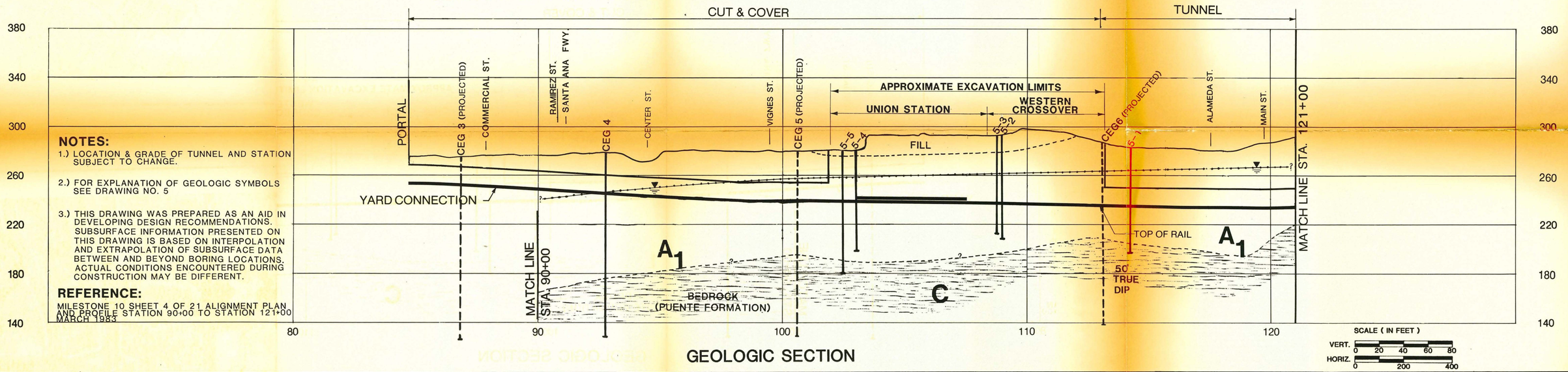
63




## LOCATION OF BORINGS




**NOTES:**

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

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

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

**REFERENCE:**

MILESTONE 10 SHEET 4 OF 21 ALIGNMENT PLAN AND PROFILE STATION 90+00 TO STATION 121+00 MARCH 1983

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

## GEOLOGIC SECTION

| REV. | DATE | BY | SUB. | APP. | DESCRIPTION |
|---|---|---|---|---|---|
| | | | | | |

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

| REV. | DATE | BY | SUB. | APP. | DESCRIPTION |
|---|---|---|---|---|---|
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DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

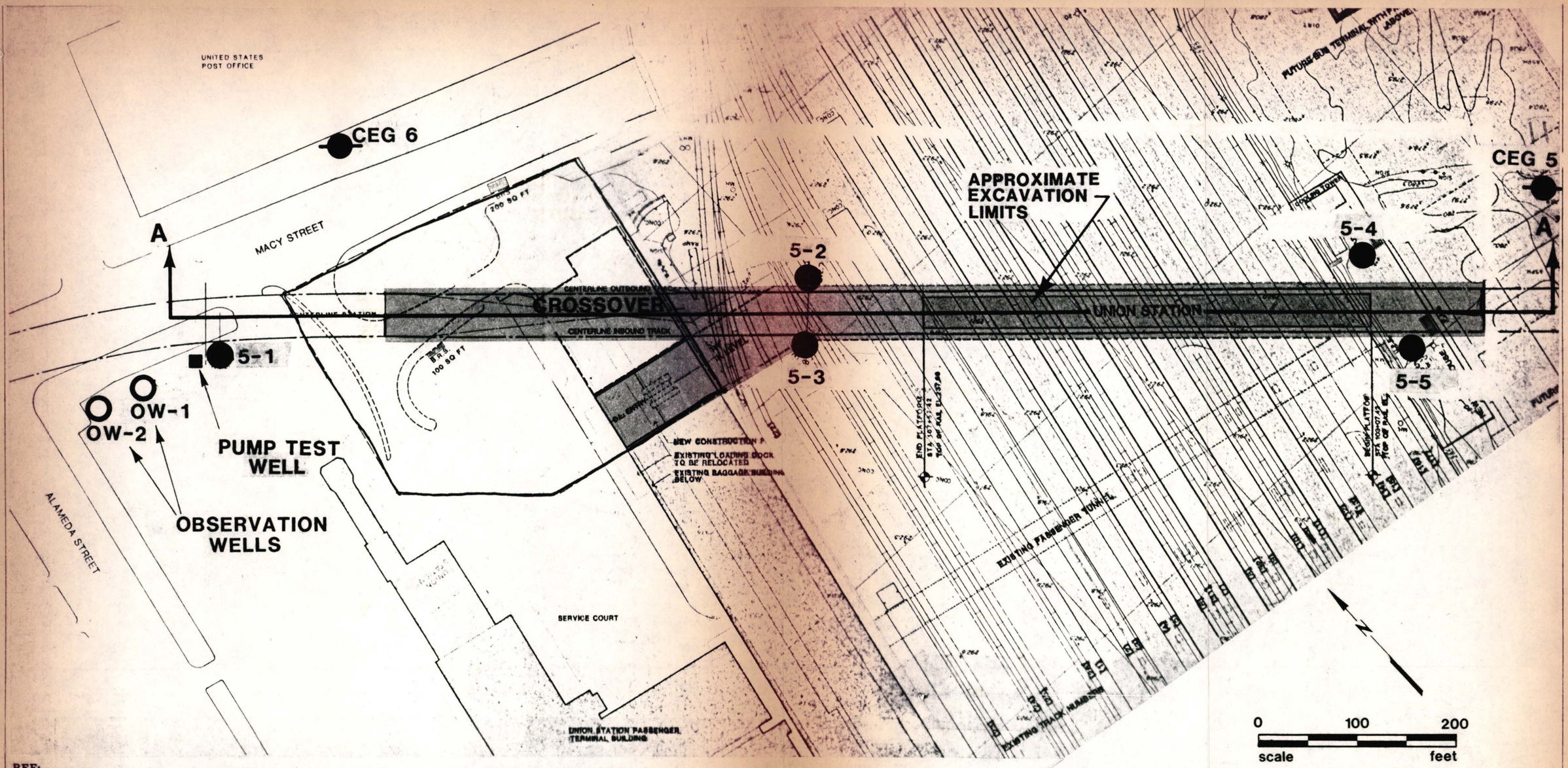
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted R.N.Pride Date 9-23-83

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

# DESIGN UNIT A135
# LOCATION OF BORINGS AND GEOLOGIC SECTION

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




REF: "PRELIMINARY UNION STATION SITE PLANS" SHEET A-1 AND A-2, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"=100', DATED 12/6/82.

**NOTES:**

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

2.) FOR EXPLANATION SEE DRAWING NO. 5

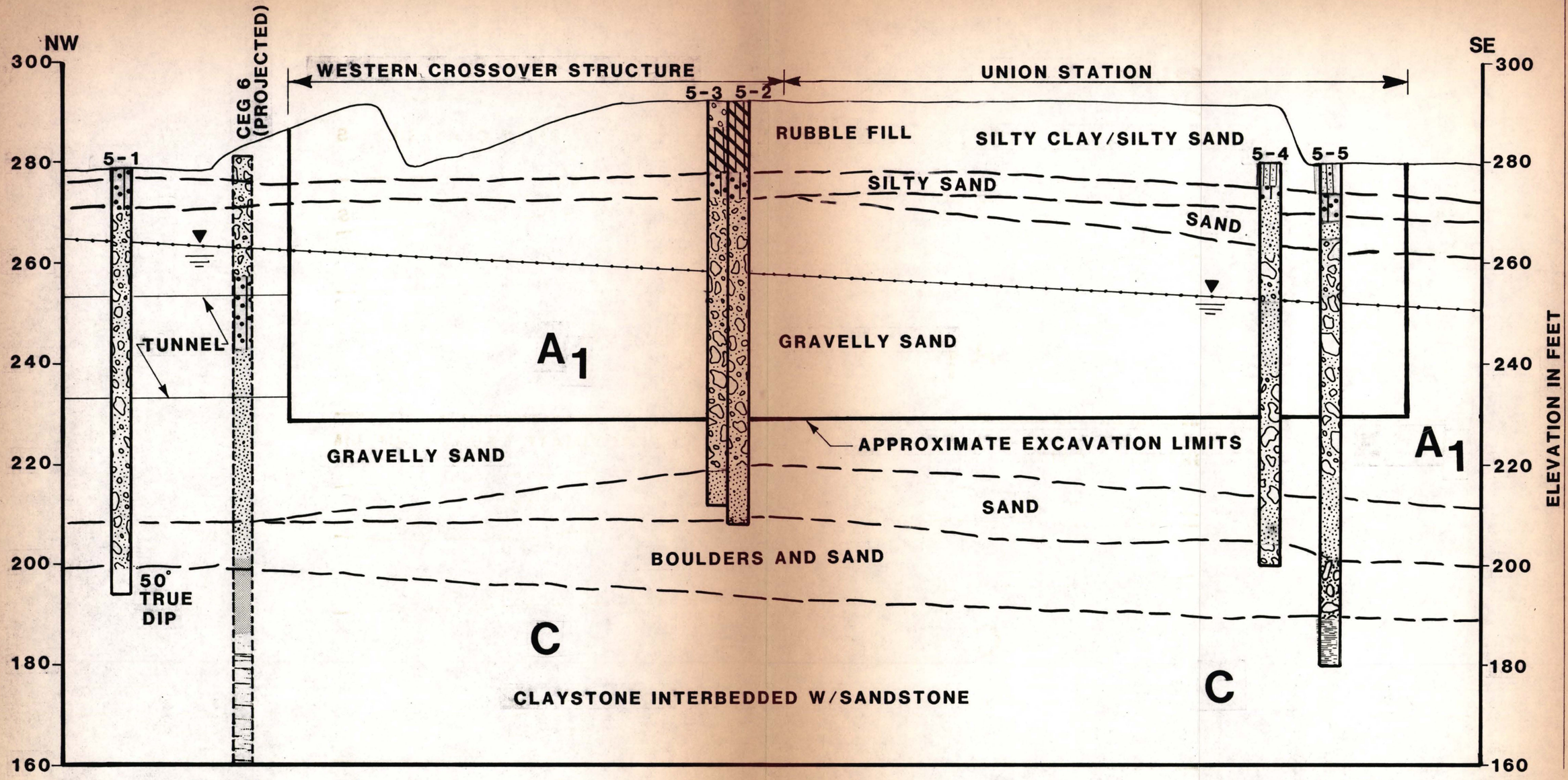
# LOCATION OF BORINGS

**DESIGN UNIT A135 - UNION STATION**
**Southern California Rapid Transit District**
**METRO RAIL PROJECT**

CCI/ESA/GRC — Geotechnical Engineering and Applied Sciences

| | |
|---|---|
| Scale | As Shown |
| Date | 9-23-83 |
| Prepared by | RG |
| Checked by | JAD |
| Approved By | RMP |
| Project No | 83-1101 |
| Drawing No | 3 |

64




SCALE
HOR. 0 50 100 200 feet
VER. 0 5 10 20 40 feet

NOTES:
1.) FOR SECTION LOCATION SEE DRAWING NO. 3
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

# GEOLOGIC SECTION A-A'

DESIGN UNIT A135 - UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

CCI/ESA/GRC
Geotechnical Engineering and Applied Sciences

| | |
|---|---|
| Scale | As Shown |
| Date | 9-23-83 |
| Prepared by | RG |
| Checked by | HAS |
| Approved By | RMP |
| Project No | 83-1101 |
| Drawing No | 4 |

65

BR[illegible]G 40.107

# GEOLOGIC UNITS

**SOFT GROUND TUNNELLING**

QUATERNARY — HOLOCENE

**$A_1$** YOUNG ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, sandy gravels, and locally contains cobbles and boulders. Primarily dense, but ranges from loose to very dense.

**$A_2$** YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.

QUATERNARY — PLEISTOCENE

**$A_3$** OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.

**$A_4$** OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from firm to hard.

**SP** SAN PEDRO FORMATION: Predominantly clean, cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium dense to very dense. Locally impregnated with oil or tar.

TERTIARY — PLIOCENE

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

**ROCK TUNNELLING**
(Terzaghi Rock Condition Numbers apply)*

**3** — Terzaghi Rock Condition Number

— Approximate boundary between Terzaghi numbers

TERTIARY — MIOCENE

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

**1-5** TOPANGA FORMATION: Basalt; intrusive, primarily hard and strong (Geologic symbol Tb).

**TERZACHI ROCK CONDITION NUMBERS:***

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

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

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

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

# SYMBOLS

Geologic contact: approximately located; queried where inferred

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)

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.

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

# GEOLOGIC EXPLANATION

**DESIGN UNIT A135 - UNION STATION**
**Southern California Rapid Transit District**
**METRO RAIL PROJECT**

Converse Consultants — Geotechnical Engineering and Applied Sciences

| | |
|---|---|
| Scale | N/A |
| Date | 9-23-83 |
| Prepared by | RG |
| Checked by | JAD |
| Approved By | HAS |
| Project No | 83-1101 |
| Drawing No | 5 |

66

BRUNING 40 107

PROJECT 80-1280-22 DATE DRILLED 2/9/81 HOLE NO. 6

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68 | SP | 38.0 – 79.0 SAND w/ GRAVELLY SAND lenses: (cont.) | | | RD | | | |
| 70 | | | | 50 | SS RD | | | 0.0/0.2 recovery Gravel shutt |
| 72 | | | | | | | | 73' – intense rig chatter |
| 74 | | | | | | | | |
| 76 | | | | | | | | |
| 78 | | PUENTE FORMATION | | | | | | NO RECOVERY on 1st DR; go down w/ bit & clean hole; take another drive sample; only DR 0.4' because hole started caving in. |
| | | 79.0 – 94.0 SILTY SANDSTONE: w/ SILT lenses: ~80% SILTY SAND and ~20% SILT lenses 0.2' to 0.5' thick; SILTY SAND is medium gray (N5); ~20% low plasticity fines; ~80% poorly graded fine sand; loose; moist; SILT is dark yellowish brown (10 YR 4/2); 100% low plasticity fines; stiff; damp. | C-1 | | DR | | | DR ⇒ 39 Blows |
| 80 | | | | 50 | SS | | | 0.4/0.4 recovery |
| | | | | | RD | | | 0.4/0.4 recovery |
| 82 | | | BOX 1 | | | 1 | | Go down & clean out hole; begin continuous Pitcher Barreling. |
| 84 | | PHYSICAL CONDITION: massive, friable, fresh. SILTY SAND: P.P. >5 SILT : P.P. >5 | | | | | | 1.7/2.8 recovery bent tube |
| 86 | | | | | | 2 | | 1.7/2.8 recovery bent tube |
| 88 | | | | | | 3 | | 1.7/2.6 recovery bent tube |
| 90 | | P.P. = 2.0 (SAND) | | | | 4 | | 2.7/2.8 recovery bent tube |
| 92 | | | | | | | | SHEET 4 OF 8 |

PROJECT 80-1280-22 DATE DRILLED 2/9/81 HOLE NO. 6

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 92 | | 79.0 - 94.0 SILTY SANDSTONE w/ SILT layers: (cont.) | BOX 1 | | PB | 4 | | |
| | | | | | | 5 | | 2.8/2.8 recovery bent tube can't be extruded |
| 94 | | 94.0 - 121.0 SILTSTONE: Dk. yellowish brown (10 YR 5/4) and light brownish gray (5 YR 5/1); 120% low plastic silt fines; massive to thinly bedded; stiff; moist; easily carved with a knife; no fractures. | S-3 | | | | | |
| 96 | | | S-4 | | | 6 | | 2.8/2.8 recovery bent tube can't be extruded |
| 98 | | | | | RD | | | Rotary Drill to tip to clean out hole. |
| 100 | | | BOX 1 (cont.) | | PB | 7 | | 2.3/2.8 recovery bent tube |
| | | 101.0'-101.5' CLAYEY [p.p. > 5] | BOX 2 | | | | | "undisturbed sample" |
| 102 | | | | | | 8 | | 2.8/2.8 recovery bent tube; hole is clean (no slutt or rop) - gravel must be within the material |
| 104 | | | S-5 | | | | | |
| 106 | | 106'-~120' Thinly bedded (0.1'-0.2' thick) w/ claystone and SAND layers. | BOX 2 (cont.) | | | 9 | | 2.8/2.8 recovery |
| 108 | 50 | | | | | 10 | | 108.4' STOP DRILLING 2/9/81 BEGIN DRILLING 2/10/81 |
| 110 | 50 | 109.4' bedding inversion, could be that core got rotated, but that doesn't seem to be the case. [p.p. = 4.5] | | | | | | 2.8/2.8 recovery |
| 112 | | dep. slumping features locally | | | | 11 | | 2.8/2.8 recovery bent tube |
| 114 | 40 | 0.2' [p.p. = 2.5] | BOX 3 | | | 12 | | 2.8/2.8 recovery bent tube |
| 116 | | 115.4 bedding disruption | | | | | | |

SHEET 5 OF 8

PROJECT 80-1280-22 DATE DRILLED 2/10/81 HOLE NO. 6

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 116 | | 94.0 – 121.0 SILTSTONE: (cont.) | BOX 3 | | PB | 12 | | |
| | | | | | | 13 | | 2.8/2.8 recovery bent tube |
| 118 | | 118' – 0.2' thick laminated sandstone. | | | | | | |
| 120 | 50/ | | | | | 14 | | 2.7/2.8 recovery bent tube |
| | | 121.0 – 134.0 SILTSTONE & SANDSTONE: ~60% SILT layers and ~40% SAND layers, layers are ~1' thick; SILT is dark yellowish brown (10YR 4/2); 100% low plasticity fines; massive (no bedding); stiff; damp; easily carved with a knife; SAND is medium gray (N5); 100% poorly graded fine sand; loose; friable; wet to saturated; no bedding. | | | | | | |
| 122 | | | S-6 | | | 15 | | 2.8/2.8 recovery ★ undisturbed sample |
| 124 | | [P.P. = 2.75 (SAND) P.P. = 5.0 (SILT)] | | | | | | |
| | | | BOX 3 (cont.) | | | 16 | | 2.5/2.8 recovery bent tube |
| 126 | | | BOX 4 | | | | | |
| 128 | | | | | | 17 | | 1.7/2.8 recovery bent tube |
| 130 | | | | | | | | |
| | | | | | | 18 | | 2.7/2.8 recovery bent tube |
| 132 | | | | | | | | |
| 134 | | 134.0 – 150.4 SILTY SANDSTONE: medium gray (N5); ~20% low plasticity fines; ~80% very poorly sorted fine grained sand; dense; wet; friable; faint bedding – rare. [P.P. = 4.25] | | | | 19 | | 2.7/2.8 recovery bent tube |
| 136 | 50/ | 136.4 – SILT layer ~0.2' thick | | | | 20 | | 2.7/2.8 recovery |
| 138 | | 138.6 – SILT layer ~0.5' thick | BOX 5 | | | | | |
| 140 | | | S-7 | | | 21 | | |

SHEET 6 OF 8

PROJECT 80-1280-22 DATE DRILLED 2/10/81 HOLE NO. 6

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 140 | | 134.0 - 150.2 SILTY SANDSTONE (cont.) | S-7 | | PB | 21 | | 1.4/2.4 recovery ★ undisturbed sample |
| 142 | | 143' - SILT layer ~0.2' thick | BOX 5 (cont.) | | | 22 | | 2.7/2.8 recovery bent tube |
| 144 | | | | | | | | |
| 146 | | | S-8 | | | 23 | | 1.5/2.4 recovery mangled tube - can't be extruded |
| 148 | | [P.P. = 5] | BOX 5 (cont.) | | | 24 | | 1.3/2.6 recovery |
| 150 | B.H. | 150.2' Bottom of Hole | | | | | | 150.2 Terminate Hole stop circulation at 11:30 2/10/81. Run e-logs (ESI). Install 150' of 2" PVC piezometer w/ cloth covered perforations from 110' to 145'; gravel pack to surface; clean-up site. Move-off site 7:00 2/11/81 water sampled 2/20/81 |

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Earth Sciences Associates
Geo/Resource Consultants

**Summary Data**
**Boring No. 6**

## SUMMARY BORING NO. 6

PROJECT 80-1280-22 STATION HOLE NO DATE DRILLED 2/3/81 - 2/10/81

OVERBURDEN DEPTH (FT.) 0 TO 79'.

BEDROCK DEPTH (FT.) 79' TO 150.2' (T.D.).

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

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

GAS TRACE; DEPTH FIRST NOTICED ____, DATE ____.

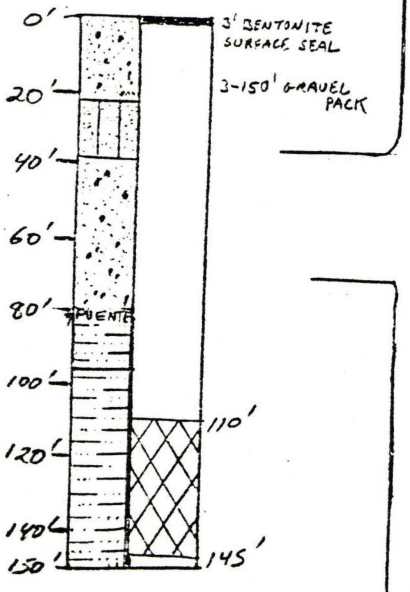
E-LOG FIES.

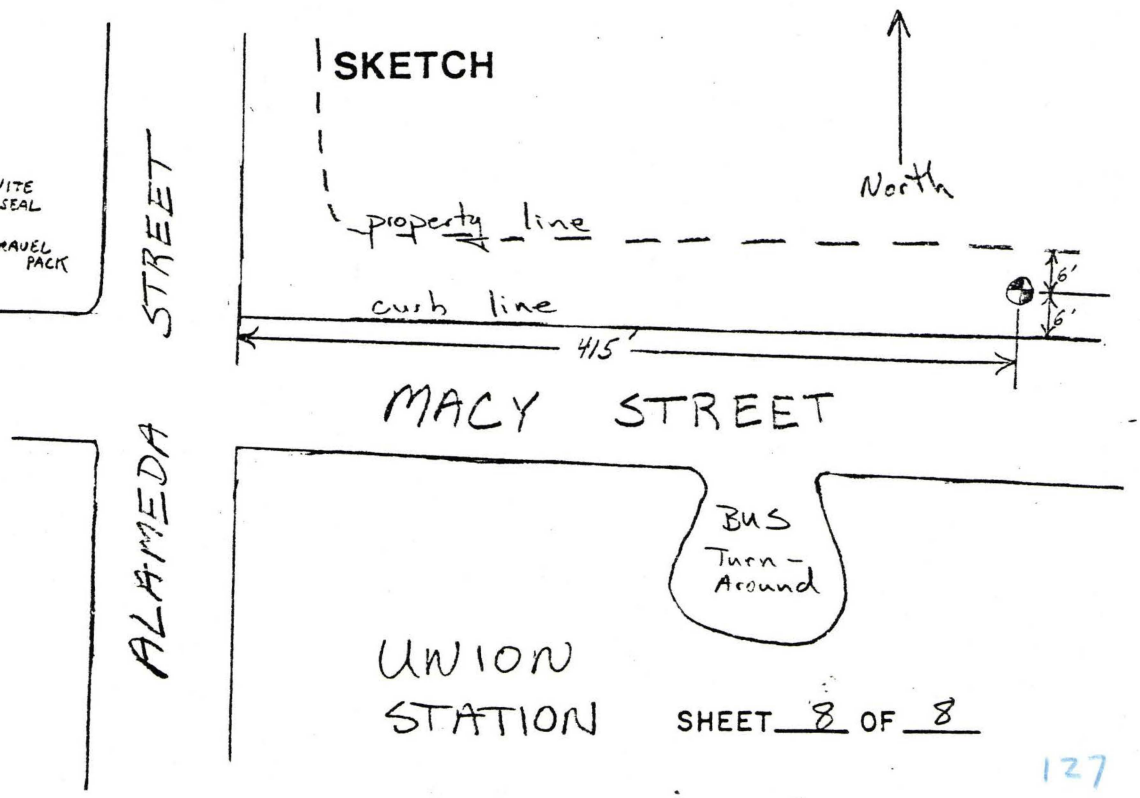
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

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

GROUND ELEVATION REF. 281'




SHEET 8 OF 8

# Appendix B
# Pump Test Results

# APPENDIX B PUMP TEST RESULTS

## B.1 INTRODUCTION AND SUMMARY

A pump test was performed near Union Station to provide data for construction dewatering. Two pump tests were run at the same well to determine aquifer properties and boundary conditions. Two pump tests were performed because gas, entrained in the water, caused the first test to be of terminated prematurely and additional testing was needed to confirm test results. The location of the pumping well and three observation holes (5-1, OW-1, OW-2) are shown on Drawing 3.

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

An effective hydraulic conductivity of 500 gpd/ft² is believed appropriate due to methane and hydrogen sulfide gas in the water. Gas bubbles in the pores reduce the ability of the aquifer to transmit water. Also, gas flow through the orifice plate used to measure well discharges result in a lower than measured water discharge. Both factors combine to reduce computed transmissitivities. However, if gas production dissipates during dewatering, hydraulic conductivity will increase to an estimated value on the order of 1,000 gpd/ft².

## B.2 SITE CONDITIONS

The pump test was located at the north end of the public parking lot near Macy Street (see Drawing 3). Bedrock of the Puente Formation underlies the test area at a depth of 80 feet. This formation consists of siltstone/claystone and clayey sandstone. The bedrock acts as an aquiclude and for practical dewatering purposes is impermeable.

Overlying the Puente Formation is 78 feet of Young (coarse-grained) Alluvium and about 2 feet of artificial fill and asphalt pavement. The alluvium thickness is very uniform over the test area. The alluvium consists of (from top to bottom) silty sand/sandy silt to a depth of about 8 feet, gravelly sand

from 8 to 70 feet, and a boulder-gravel zone from a depth of 70 to 80 feet (see Drawing 4). Based on boring samples, the aquifer generally contains high percentages of fine to medium sand (see grain-size distribution curves, Figures F-3, F-4 and F-5 in Appendix F).

Static water level was 20 feet below ground surface. The saturated thickness of the unconfined aquifer was 60 feet.

## B.3 WELL CONSTRUCTION AND DEVELOPMENT

The test well is located about 11 feet southwest of test Boring 5-1. The well was drilled by the cable tool method, driving 12-inch double walled casing. The well was drilled to a depth of 82.5 feet and the casing was perforated inplace from 21 feet to 77 feet. Perforations consist of 12 horizontal punched slots per foot that are 1-1/4 by 5/32 inches, in staggered rows.

The existing test boring, 5-1, has 2-inch PVC casing that is slotted from 45 feet to 85 feet. The boring was 4-3/4 inches in diameter drilled by the mud rotary method. The annulus between the PVC casing and the well bore was filled with pea gravel and sealed with bentonite above the perforations. This well was used as a supplemental observation well during the pump tests.

Two new observations wells were drilled by mud rotary methods, in line to the west of the test well. OW-1 is located 51 feet west of the well and was drilled to a depth of 84 feet. PVC casing, 4 inches in diameter, was installed with perforations from 5 to 70 feet. The annulus of the 6-inch bore was backfilled with pea gravel and sealed with 4 feet of cement and gravel.

Observations well OW-2 is located 110.6 feet west of the well and was drilled to a depth of 85 feet. PVC casing, 4 inches in diameter, was installed to a depth of 83 feet with perforations from 5 to 75 feet. The annulus of the 6-inch bore was backfilled with pea gravel and sealed with 4 feet of cement and gravel. The layout of the observation wells and the test well are shown on Drawing 3.

The pumping well was developed to flush mud and cuttings to provide hydraulic communication with the aquifer. The 12-inch test well was surged by bailing and then developed with the test pump for about 20 hours. Gas was noticed in the pump discharge during development pumping at maximum drawdowns. Near the end of the pump tests, distant drawdown measurements indicate that the pumped well was operating at a hydraulic efficiency of about 30%. That is, 70% of well drawdown was due to well losses in the pumped well.

## B.4 PUMP TESTING PROCEDURE

A constant discharge test was planned with a test duration of 24 to 48 hours. However, because of gas problems that developed with time, two relatively short duration tests were performed.

The gas problems are discussed at the end of the Section under comments on test results.

The first test was run on March 24, 1983 for approximately 500 minutes at a well discharge rate of 175 gpm. The test was terminated because gas was causing the discharge rate to fluctuate and observation wells began to recover indicating a reduced well discharge. The second test was run at a reduced discharge rate of 150 gpm for 1380 minutes during March 25 and 26, 1983. Again, the test was terminated because of gas interference resulting in recovery of observation wells.

The test well was pumped with an Aurora lineshaft turbine pump (capacity 500 gpm) powered by a Cummings diesel engine. Discharge was measured with an inline orifice plate and a mercury manometer. The base of the 10-inch diameter bowls were set at a depth of 78 feet below the ground surface. Water from the well was discharged through a pipe to a storm drain near the southeast corner of the intersection of Alameda and Macy Streets.

Drawdowns in the pumped well (maximum 30 feet) were measured occasionally by air line during the tests, but were not used for test interpretations because of the high well losses. Drawdowns were measured in each observation well at times selected to provide suitable logarithmic distributions of time. Drawdown measurements in observation wells OW-1 and OW-2 were made with Stevens Recorders. Times were recorded manually on the chart paper because the recorder clocks are not sensitive enough particularly during the early more frequent measurements. Drawdowns in observation well 5-1 were measured with a hand-held electric sounder. Generally, measurements of drawdown were accurate within $\pm$0.01 foot.

Recovery tests were planned after the pumping ceased. However, during both tests, recoveries had already started prior to stopping the pump due to gas problems. Regardless, recovery measurements were made in OW-1 and OW-2 after the first test. These measurements, however, produced unreliable results because of the premature recovery combined with delayed drainage effects. As a result, it was decided to test the gas responses at various pumping rates after the second test (in place of recovery measurements) using a gas detector.

Rubber tubing with a metal tip, attached to the methane reading gas detector (made by Gastech Inc.), was inserted to a depth of 5 feet in the pumping well's water level measuring hole, immediately after the pump was turned off. Instantly the methane gas detector needle surged to a reading of 100% lower explosive limit (LEL) and for some unexplainable reason the gas ignited in the instrument causing a small explosion that blew the rubber tubing out of the hole. Prior to this, the gas detector indicated around 30% LEL methane gas each time gas was measured during the second pump test and did not explode.

## B.5 TEST INTERPRETATIONS

Time-drawdown data were plotted on log-log graphs as shown on interpretation charts. Figures B-1 and B-2 show data for the first and second tests, respectively, for observation wells OW-1 and OW-2 with the log of drawdown(s) in feet plotted versus the log of time (days) divided by the distance (feet) from the pump well squared ($t/r^2$). These data plots were matched to the type curves indicated and appropriate match points were selected to determine values of s and $t/r^2$ for corresponding values of $W(\upsilon)$ and $1/\upsilon$. The calculations for transmissivity (T), Storativity (S) or Specific yield (Sy) are shown.

Figure B-3 shows data plots, match points and calculations for observation well 5-1 for both the first and second tests.

The water level responses of OW-1 indicate the typical S-shaped curve of an unconfined aquifer with delayed drainage for both tests. The plots of the first test are somewhat distorted possibly due to gas and a recharge event (intense rainfall) just prior to the test. The second test at the reduced pumping rate provided better matches for both the A region and B region of the type curves.

Data from well 5-1 indicate on both tests that most of the drawdown occurred in the first minute of pumping and the plot is in the region of delayed drainage. However, using OW-1 as a guide, reasonable matches were obtained.

Data from OW-2 indicate the possibility of delayed well response. OW-2 may have been damaged by siltation when runoff water flowed into the well immediately prior to the first test. However, good matches were obtained in the B region of the curves for both tests. Results from OW-2 for both tests must remain somewhat suspect, however.

Distance-drawdown plots shown on Figure B-4, were used as a check where log of the distance is plotted against drawdown on a semi-log chart. Wells OW-1 and OW-2 were used in this plot primarily. Well 5-1 is very close to the pumped well in a region where potential lines are relatively distorted. Since the bottom 40 feet of this observation well are perforated, drawdowns at a given time would tend to be greater than should be the case in the distance-drawdown relationship. The results of the analyses from the best fit type of curves along with distance drawdown analyses are summarized in Table B-1 below.

TABLE B-1
SUMMARY OF PUMP TEST RESULTS

| Test | Observation Well | Curve Match | Transmissivity (gpd/ft) | Average Hydraulic Conductivity (gpd/ft$^2$) | Storativity (S) or Specific Yield (Sy) |
|---|---|---|---|---|---|
| 1st | OW-1 | B Type, β=0.06 | 20,055 | 334 | 0.064 (Sy) |
| 1st | OW-2 | B Type, β=6.0 | 30,386 | 506 | 0.021 (Sy) |
| 1st | 5-1 | B Type, β=0.01 | 20,055 | 334 | 0.23 (Sy) |
| 1st | OW-1, OW-2, 5-1 | Dist. d.d. | 24,973 | 416 | 0.028 (Sy) |
| 2nd | OW-1 | A Type, β=0.06 | 20,464 | 341 | 0.0044 (S) |
| 2nd | OW-1 | B Type, β=0.06 | 21.488 | 358 | 0.069 (Sy) |
| 2nd | OW-2 | B Type. β=6.0 | 41.927 | 699 | 0.022 (Sy) |
| 2nd | 5-1 | B Type β=0.01 | 15,627 | 260 | 0.27 (Sy) |
| 2nd | OW-1, OW-2, 5-1 | Dist. d.d. | 21,405 | 357 | 0.061 (Sy) |

The transmissivities range from 16,000 to 42,000 gpd/ft with a mean of about 24,000 gpd/ft. The average hydraulic conductivity is the transmissivity divided by the saturated thickness of 60 feet. Average hydraulic conductivities range from 250 to 700 gpd/ft² (~1.3 x $10^{-2}$ to 3.2 x $10^{-2}$ cm/sec). These values are in a low range for clean stream channel deposits, but these low values may be explained by free gas, which would tend to reduce hydraulic conductivities, influencing results significantly. A hydraulic conductivity of 1,000 gpd/ft² or more is judged reasonable for these deposits near Union Station, if gas were not present.

Most of the computed specific yields appear to be very low for this type of aquifer. The values computed range from 0.021 to 0.27. A specific yield in the 0.15 to 0.25 range would be reasonable, and a value of 0.2 to 0.25 is more probable. For dewatering purposes the use of a specific yield of 0.25 is recommended because this would be more conservative.

## B.6 COMMENTS ON TEST RESULTS

Boundary conditions were not detected from the pump tests at Union Station. This was primarily due to the relatively short duration of both tests. Longer term dewatering operations should encounter barrier boundary effects as the pumping cone intersects the boundary of the aquifer about 1000 feet west of the pumping well as shown on Milestone 10 Drawings 2 and 3. This will enhance dewatering to some degree. The effect can be estimated by determining distance to a barrier(s) from geologic maps and factoring the barrier(s) into dewatering computations.

The gases encountered were probably methane and hydrogen sulfide, based on the gas detector methane readings as well as the "rotten egg" odor. These gases are potentially hazardous and will affect the hydraulic efficiencies of pumps and wells. The gases are assumed to originate from the underlying Puente Formation and confined by the aquifer. When wells are drawn down, the reduction in hydrostatic pressure releases the gas. This implies that gas volume will increase with well drawdowns which appeared to be the case during pump tests. There seemed to be a threshold where relatively large amounts of gases were released at about 9 to 10 feet of true drawdown (discounting well losses). It is unknown how long it would take to vent the gases present and what effect this would have on aquifer transmissivity.

The following information regarding gas and oil in this area helps explain the source, if not the pressure or quantity of gas in the pump test well:

- The Union Station Oil Field is located some 2000 feet south of the proposed station site. Little is known about the oil field, but it does produce from the Puente Formation at very shallow depths.

- Boring CEG-2 (see Drawing 2) located some 2000 feet east of the Union Station site penetrated the Young Alluvium/Puente Formation contact at a depth of 38 feet. Oil was encountered at this contact as well as in sandstone layers within the Puente Formation from 38 to 100 feet (bottom

of hole). Gas (hydrogen sulfide) was first detected by odor at a depth of 37 feet in Boring CEG-2. Gas chromatograms run in Boring CEG-1 indicate 100 ppm methane and 500 ppm ethane (see Appendix D). This would thus be classified in the lower explosive limit (5% LEL) as described in Appendix D, Tables D-1 through D-4.

It is believed that at least a portion of the ground water underlying the site may be either saturated with gas which originated from the underlying Puente Formation and/or contain free gas in the aquifer or underlying Puente Formation that is released as hydrostatic pressures are reduced during pumping. During the pump test, there was a considerable drop in pressure head near the well as water flowed into the pump. This pressure drop would have resulted in release of the gas and into the well head. Additional data would be required to confirm these concepts and delineate the problem. The engineering implications of these observations are discussed in Chapter 6.

The pump tests resulted in reasonably consistent data in terms of transmissivity and hydraulic conductivity. A mean transmissivity of 24,000 gpd/ft is considered a good effective value for gassy conditions, however permeability along the length of the Union Station excavation may vary significantly because of the variable conditions.

It is beyond the scope of this report to recommend specific dewatering systems. However, the aquifer is amenable to well dewatering, providing the gases are controlled safely and effectively. Also, more efficient wells would be needed to be cost effective. Mainly well screens with larger open areas are needed such as wire wound screens or Rosco Moss' "Full Flo" louvered screen, along with careful well development. Soil grain size distribution curves are presented in Figures F-3, F-4 and F-5 of Appendix F. The presence of hydrogen sulfide gas suggests some corrosion potential which should be considered with the time that dewatering is required for construction.

Union Station 1st Test 3/24/83

$T = \frac{114.6\,Q}{s}$

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

$Q = 175$ gpm

OW-1, A Match Pt
$W(u_A) = 1$
$1/u_A = 1$

Artesian Type Curve

OW 2, B Match Pt
$\beta = 6.0$
$W(u_B) = 1$
$1/u_B = 10$

OW-1, A Match Pt
$\beta = 0.06$
$W(u_A) = 1$
$1/u_A = 10$

OW-1, A Match Pt

$T = \frac{114.6\,(175)}{1.6} \cdot 1 = 12,534$ gpd/ft

$S = \frac{12,534\,(2.8 \times 10^{-7})}{1.87\,(1)} = 0.0019$

OW-1 B Match Pt, $\beta = 0.06$

$T = \frac{114.6\,(175)}{1.0} \cdot 1 = 20,055$

$S_y = \frac{20055\,(6 \times 10^{-5})}{1.87\,(10)} = 0.064$

OW-2, B Match, $\beta = 6.0$

$T = \frac{114.6\,(175)}{0.66} \cdot 1 = 30,356$

$S_y = \frac{30356\,(1.3 \times 10^{-5})}{1.87\,(10)} = 0.021$

$t/r^2$

KE LOGARITHMIC 3 X 5 CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
47 7520

135

FIGURE B-1

# Appendix A
# Field Exploration

# APPENDIX A FIELD EXPLORATION

## A.1 GENERAL

Field exploration data presented in this report for Design Unit A-135 includes information from borings drilled for the 1981 Geotechnical Investigation Report and additional borings drilled for this investigation. Three borings (CEG-4, 5 and 6) were drilled near or within Design Unit A135 during the 1981 investigation and the logs are reproduced in this appendix. Five additional borings (5-1 to 5-5) were drilled in 1983 for this investigation. Man size auger boring 6A drilled in 1983 is also included in this appendix. Locations of the borings are shown on Drawings 2 and 3. Ground water observation wells (piezometers) were installed in borings CEG-4, CEG-6, 5-1 and 5-5. A groundwater pump test was performed adjacent to boring 5-1 (see Appendix B). Geophysical downhole and crosshole surveys were made at boring CEG-5 (see Appendix C).

The borings were drilled to depths generally ranging from 80 to 150 feet, and five of the borings penetrated through the granular alluvium (Unit $A_1$) into the underlying bedrock (Unit C). All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample and core recovery was good in the siltstone and claystone bedrock (Unit C) but generally poor in the coarse granular alluvium (Unit $A_1$). Overall sample recovery average about 84%.

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

## A.2 FIELD STAFF AND EQUIPMENT

### A.2.1 Technical Staff

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

### A.2.2 Drilling Contractor and Equipment

Drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 1500 rotary wash rigs, each operated by a two man crew.

## A.3 SAMPLING AND LOGGING PROCEDURES

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

### A.3.1 Sampling

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

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

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

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

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

## A.3.2 Field Classification of Soils

All soil types were classified in the field by the 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-1 shows the correlation of standard penetration information and the physical description of the consistency of clays (hand-specimen) and the compactness of sands used by the field geologists for describing the materials encountered.

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

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

## A.3.3 Field Description of the Formations

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

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

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

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

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

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

TABLE A-2 Bedrock Description Terms

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

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

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

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

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

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

## A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings CEG-4 and CEG-6 in 1981 and two additional piezometers were installed in borings 5-1 and 5-5 in 1983. Procedures for piezometer installtion were the following:

A two-inch diameter plastic ABS pipe was installed in the boring and 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 around the 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 using air lift provided by a trailer-mounted air compressor. The piezometer was covered with a standard 7-inch diameter steel water meter cap held at surface grade by a grouted in-place 3- to 4-foot long, 5-inch diameter plastic sleeve. Ground water data obtained from the piezometer is presented in Section 5.4 of the text.

# Converse Consultants

**Boring Log** 5-1

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

PROJECT SCRTD - Union Station DATE DRILLED 2/4/83 HOLE NO. 5-1
LOCATION Northeast Corner of Public Parking Lot GROUND ELEV. 279.1'
DRILLING CONTRACTOR Converse-Las Vegas LOGGED BY B. Inghram DEPTH TO GROUND WATER 20.3 2-8-83
TYPE OF RIG _____ HOLE DIAMETER 4¾" HAMMER WEIGHT AND FALL 320#, 36"
SURFACE CONDITIONS A.C. Parking Area TOTAL DEPTH 85.0 NO. CORE BOXES _____

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | 0.0 - 0.4' Asphalt Pavement | | | RD | | | Set up 7:00 AM |
| | SM | 0.4 - 2.0 Fill - Silty Sand | | | | | | Begin Drilling 7:30 AM |
| | | Mottled Brn, Moist, Dense | 1-1 | | CCI | | | Weather: Clear, Warm |
| 2.0 | | Variable Composition | 12K | | DR | | | |
| | SM/ML | Young Alluvium | | | RD | | | |
| | | 2.0 - 4.0 Silty Sand / Sandy Silt | | | | | | Slightly Porous |
| 4.0 | | Red-Brn., Moist, M. Dense | | | | | | |
| | SM | V. Fine Sand - 50-60% | | | | | | |
| | | Silt - 40-50% | | | | | | |
| | | 4.0 - 8.0 Silty Sand | 1-2 | | CCI | | | |
| 6.0 | | Gray-Brn., Moist, M. Dense | 6K | | DR | | | |
| | | Fine Sand with 20% Silt | | | RD | | | |
| | | and trace Gravel to 1" | | | | | | |
| 8.0 | | | | | | | | |
| | SW | 8.0 - 70.0 Gravelly Sand | | | | | | |
| 10.0 | | | 5" | 100 | SPT | | | |
| | | | Refusal | | RD | | | |
| 12.0 | | | | | | | | |
| 14.0 | | | | | | | | |
| 16.0 | | | | | | | | |
| 18.0 | | | | | | | | |
| 20.0 | | | | | | | | |

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SHEET 1 OF 5

PROJECT SCRTD - UNION STATION DATE DRILLED 2/4/83 HOLE NO. 5-1

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SW | 8.0-70.0 GRAVELLY SAND (CONTINUED) | 50K | | CCI DR | | | DELICATE SAMPLE, DIFFICULTY EXTRUDING - SAMPLE DESTROYED |
| | | | | | RD | | | |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | | 1-3 45K | | CCI DR | | | POOR RECOVERY, DELICATE SAMPLE 3 RINGS ONLY |
| | | | | | RD | | | |
| 32.0 | | INTERMITTENT LENSES OF INCREASED GRAVEL CONTENT | | | | | | |
| 34.0 | | | | | | | | |
| 36.0 | | | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | 4-4 55K | | CCI DR | | | SULPHUR ODOR IN SAMPLE |
| | | | | | RD | | | POOR RECOVERY, 2 RINGS ONLY |
| 42.0 | | | | | | | | |
| 44.0 | | | | | | | | |

PROJECT SCRTD - UNION STATION DATE DRILLED 2/4/83 HOLE NO. 5-1

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | SW | 8.0 - 70.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| | | | 1-5 57K | | CCI DR | | | SULPHUR ODOR POOR SAMPLE RECOVERY - 2 RINGS ONLY |
| 46.0 | | | | | RD | | | |
| 48.0 | | | | | | | | |
| 50.0 | | | | | CCI DR | | | GRAVELLY SAMPLE - FELL OUT OF SAMPLER |
| | | | 80K | | RD | | | |
| 52.0 | | | | | | | | |
| 54.0 | | | | | | | | SLIGHT SULPHUR ODOR |
| | | | 1-6 100K | -10" | CCI DR | | | POOR SAMPLE RECOVERY 5 RINGS ONLY |
| 56.0 | | | | | RD | | | |
| 58.0 | | | | | | | | |
| 60.0 | | | | | | | | |
| 62.0 | | | | | | | | |
| 64.0 | | | | | | | | SULPHUR ODOR POOR RECOVERY 4 RINGS ONLY |
| | | | 1-7 100K | -10" | CCI DR | | | |
| 66.0 | | | | | RD | | | |
| 68.0 | | | | | | | | |



SHEET 3 OF 5

PROJECT SCRTD - UNION STA. DATE DRILLED 2/4/83 HOLE NO. 5-1

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | SW | 8.0 - 70.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| 70.0 | B | 70.0 - 79.5 BOULDERS<br>GRANITIC-TYPE CUTTINGS | | | | | | |
| 72.0 | | | | | | | | |
| 74.0 | | | | | | | | |
| | | | 100K REFUSAL | -0" | | | | ATTEMPTED TO SAMPLE - COULD NOT DRIVE SAMPLER<br>TOO HARD TO SAMPLE |
| 76.0 | | | | | | | | |
| 78.0 | | | | | | | | |
| 80.0 | CL/SC | PUENTE FORMATION<br>79.5 - 85.0 SANDY CLAYSTONE AND CLAYEY SANDSTONE<br>OLIVE-GRAY COLOR, MOIST, FRESH, THINLY LAMINATED, BEDDING PLANES DIP ≈ 50° (SAMPLES NOT ORIENTED), FRIABLE STRENGTH, FRIABLE TO LOW HARDNESS. TENDS TO FRACTURE ALONG LAMINATIONS | 1-8 50K | | CCI DR | | | |
| 82.0 | | | | | RD | | | |
| 84.0 | | | 1-9 40K | | CCI DR | | | |
| | | END BORING 85.0 FT<br>PIEZOMETER SET TO 85.0'<br>PERFORATED INTERVAL: 45'-85' | | | | | | |

# SUMMARY BORING NO. 5-1

PROJECT SCRTD STATION HOLE UNION STA DATE DRILLED 2/4/83.

OVERBURDEN DEPTH (FT.) 0.0 TO 79.5'.

BEDROCK DEPTH (FT.) 79.5' TO 85.0 (T.D.).

WATER PRESS. TEST N/A; INTERVAL(S) — TO —, — TO —.

GROUND WATER DEPTH (FT.) 20.3 DATE 2-8-83; 22.7' DATE 3/10/83.

GAS YES; DEPTH FIRST NOTICED 40', DATE 2/4/83. - SULPHUR ODOR

E-LOG N/A.

DOWN-HOLE SURVEY N/A.

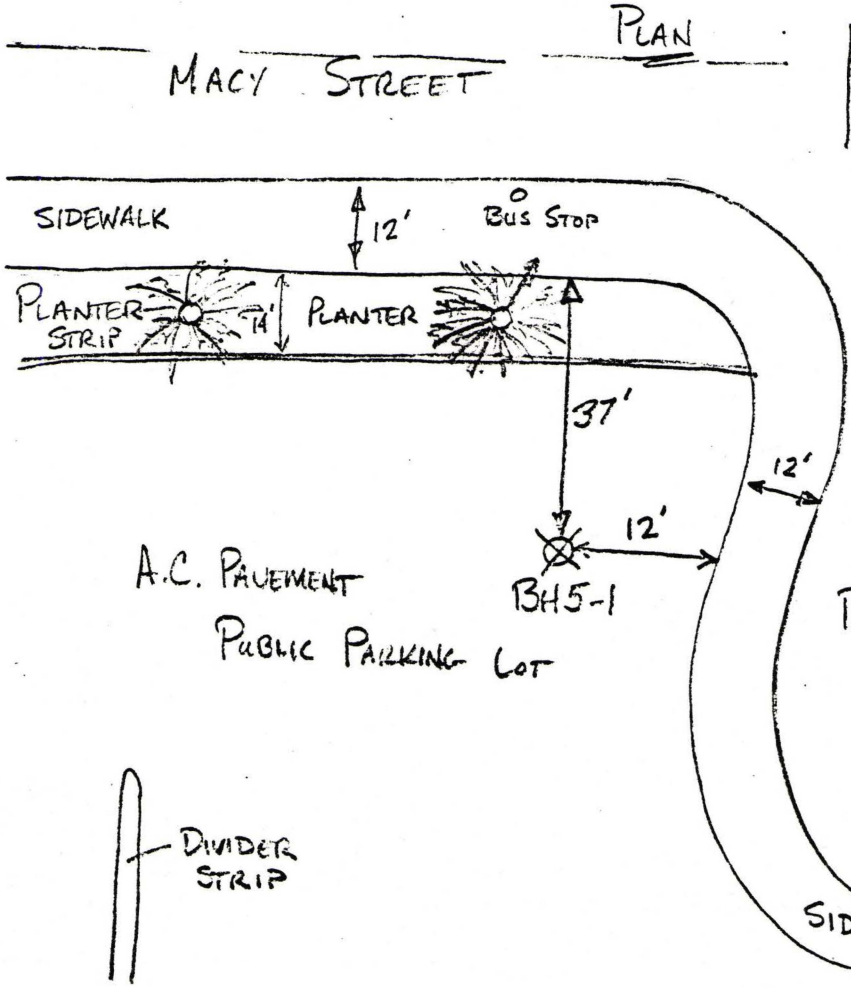
CROSS-HOLE SURVEY N/A.

PVC CASING (I.D.): 4" — TO —; 3" — TO —; 2" 0.0 TO 85.0'.

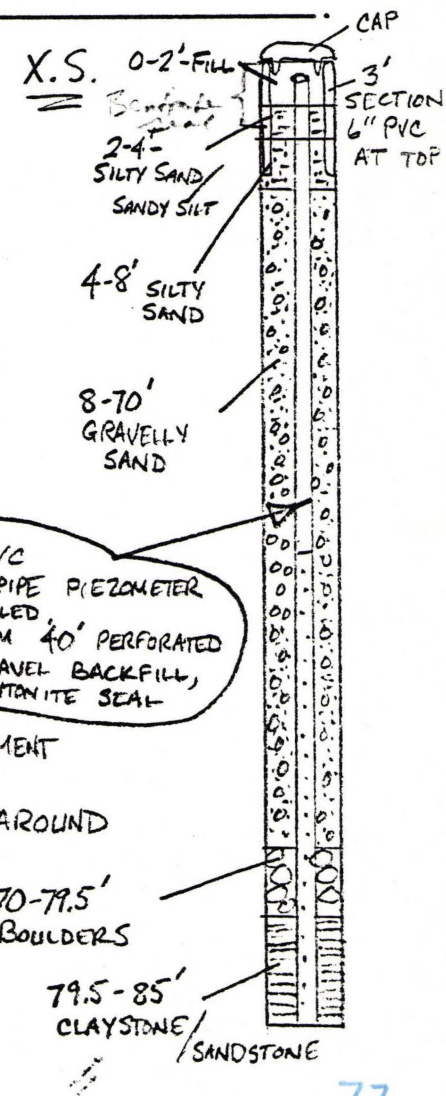
GROUND ELEVATION REF. XXX POST OFFICE BLDG XXXXXXXXXXXXXXXX

## SKETCH

PLAN




X.S.

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SHEET 5 OF 5

# Converse Consultants

**Boring Log** 5-2

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

PROJECT SCRTD - UNION STATION DATE DRILLED 2/3/83 HOLE NO. 5-2
LOCATION BETWEEN TRACKS & BAGGAGE BUILDING GROUND ELEV. 292.7'
DRILLING CONTRACTOR CONVERSE - LAS VEGAS LOGGED BY B. INGHRAM DEPTH TO GROUND WATER ___
TYPE OF RIG ___ HOLE DIAMETER 4¾" HAMMER WEIGHT AND FALL 320#, 36"
SURFACE CONDITIONS A.C. / CONCRETE PAVED AREA TOTAL DEPTH 85.0' NO. CORE BOXES ___

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | 0.0-0.4' CONCRETE SLAB | | | RD | | | SET UP 7:00 AM |
| | | 0.4-0.8' BASE COURSE | | | | | | BEGIN DRILLING 7:30 AM |
| | ML & CL | 0.8-14.0' FILL - CLAYEY SILT AND SILTY CLAY | 2-1 8K | | CCI DR | | | WEATHER: OVERCAST, COOL |
| 2.0 | | MOTTLED BRN & GREEN-GRAY, MOIST, STIFF. W/ TRACE GRAVEL AND 10% FINE SAND | | | RD | | | |
| 4.0 | | | | | | | | |
| 6.0 | | | | | | | | |
| 8.0 | | | | | | | | |
| 10.0 | | | | 8 11 24 | SPT | | | |
| 12.0 | | | | | RD | | | |
| | | 13.0' - ROCK OR CONCRETE ENCOUNTERED | | 100-0" REFUSAL | | | | TRIED TO SAMPLE WITH SPT - COULD NOT DRIVE. |
| 14.0 | SM | YOUNG ALLUVIUM 14.0-19.0 SILTY SAND | | | | | | |
| 16.0 | | GRAY - BRN, MOIST, MED. DENSE FINE TO V. FINE SAND W/ 20% SILT | | 6 7 14 | SPT | | | |
| 18.0 | | | | | RD | | | |
| 20.0 | SW | 19.0-72.0 GRAVELLY SAND (SEE NEXT PAGE) | | | | | | |

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SHEET 1 OF 5

PROJECT SCRTD - Union Station DATE DRILLED 2/3/83 HOLE NO. 5-2

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SW | 19.0 - 72.0 GRAVELLY SAND (CONTINUED) GRAY, DENSE WELL GRADED - MED. TO COARSE SAND, GRAVEL TO 2" ϕ, PERCENTAGES VARYING WITH DEPTH GRANITIC COMPOSITION | 2-2 | | CCI DR | | | DISTURBED SAMPLE - 4 RINGS ONLY |
| | | | 25K | | RD | | | |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | | | 70 | SPT | | | RUSTY STAINED GRAINS IN SAMPLE - OXYDIZING ENVIRONMENT |
| | | | 3" REFUSAL | 100 | RD | | | |
| 32.0 | | | | | | | | |
| 34.0 | | | | | | | | |
| 36.0 | | | | | | | | |
| | | INCREASING CONTENT OF FINE & V. FINE GRAINED SAND | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | | | CCI DR | | | NO SAMPLE RECOVERY |
| | | | 50K | | RD | | | |
| 42.0 | | | | | | | | |
| 44.0 | | | | | | | | |

PROJECT SCRTD - Union Station DATE DRILLED 2/3/83 HOLE NO. 5-2

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | SW | 19.0 - 72.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| | | COLOR CHANGE OBSERVED - TO DK. GRAY | 2-3 | | CCI DR | | | SLIGHT SULPHUR ODOR, GRAY SAMPLE - REDUCING ENVIRONMENT |
| 46.0 | | | 75K | | RD | | | POOR SAMPLE RECOVERY - - 4 DISTURBED RINGS ONLY |
| 48.0 | | | | | | | | |
| 50.0 | | | | | | | | |
| 52.0 | | | | | | | | |
| 54.0 | | | | | | | | |
| | | | | | CCI DR | | | NO SAMPLE RECOVERY |
| 56.0 | | | 80K | | RD | | | |
| 58.0 | | | | | | | | |
| 60.0 | | | | | CCI DR | | | NO SAMPLE RECOVERY |
| | | | 75K | | RD | | | |
| 62.0 | | GRAVEL CONTENT DECREASES W/ DEPTH | | | | | | |
| 64.0 | | | | | | | | |
| | | | J-1 5' | 100 | SPT | | | |
| 66.0 | | | REFUSAL | | RD | | | |
| 68.0 | | | | | | | | |

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SHEET 3 OF 5

PROJECT SCRTD - UNION STATION DATE DRILLED 2/3/83 HOLE NO. 5-2

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | SW | 19.0-72.0 GRAVELLY SAND | | | RD | | | |
| | | (CONTINUED) | | | | | | |
| 70.0 | | DECREASING GRAVEL CONTENT | 2-4 60K | | CCI DR | | | SLIGHT SULPHUR ODOR |
| | | | | | RD | | | |
| 72.0 | SP | 72.0-83.0 SAND | | | | | | EASIER DRILLING |
| | | DARK GRAY, DENSE, POORLY GRADED PREDOM. MED. GRAINED TRACE GRAVEL TO 2" | | | | | | |
| 74.0 | | | 5" | 100 | SPT | | | SULPHUR ODOR |
| | | | | | RD | | | |
| 76.0 | | | | | | | | |
| | | SAND BECOMES MORE FINE-GRAINED | | | | | | |
| 78.0 | | | | | | | | |
| | | FINE TO V. FINE GRAINED | | | | | | |
| 80.0 | | | 2-5 50K | | CCI DR | | | SULPHUR ODOR DISTURBED SAMPLE ONLY 5 RINGS |
| | | THIN LENSES OF SILTY SAND | | | RD | | | |
| 82.0 | | | | | | | | |
| | | 83.0-84.0 BOULDERS | | | | | | HARD DRILLING |
| | | GRANITIC-TYPE CUTTINGS | 50K-0" REFUSAL | | CCI | | | TRIED TO SAMPLE - COULD NOT DRIVE SAMPLER |
| 84.0 | | END BORING 84.0 FT | | | | | | |

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SHEET 4 OF 5

# SUMMARY BORING NO. 5-2

PROJECT SCRTD STATION HOLE UNION STATION DATE DRILLED 2/3/83.

OVERBURDEN DEPTH (FT.) 0.0 TO 84.0 (T.D.)

BEDROCK DEPTH (FT.) — TO — ~~(T.D.)~~.

WATER PRESS. TEST No; INTERVAL(S) — TO —, — TO —.

GROUND WATER DEPTH (FT.) — DATE —; — DATE —. ROTARY WASH BORING - COULD NOT DETERMINE

GAS YES; DEPTH FIRST NOTICED 45', DATE 2/3/83. - SULPHUR ODOR

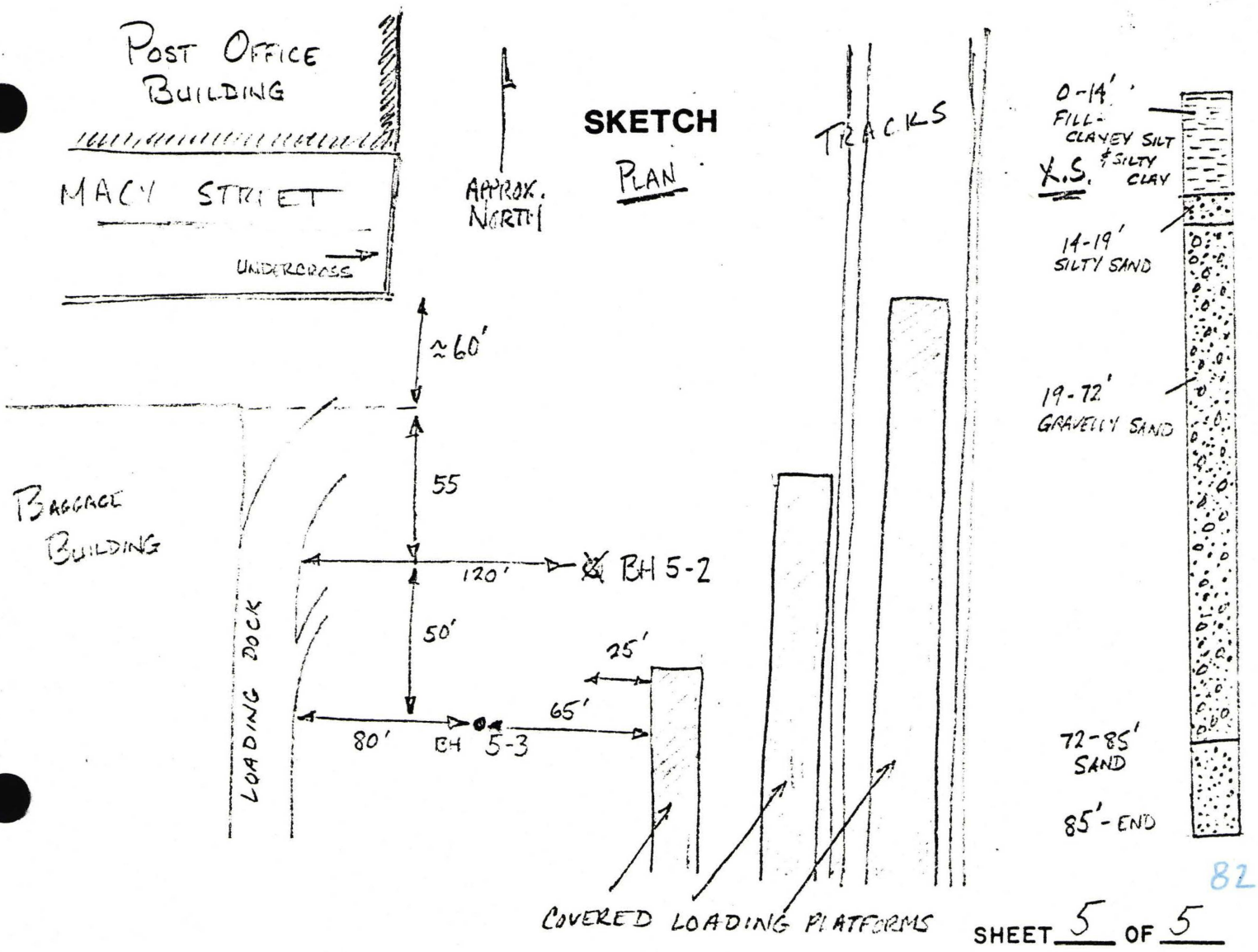
E-LOG No.

DOWN-HOLE SURVEY No.

CROSS-HOLE SURVEY No.

PVC CASING (I.D.): 4" — TO —; 3" — TO —; 2" — TO —. NONE

GROUND ELEVATION REF. _____.

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SHEET 5 OF 5

# Converse Consultants

**Boring Log** 5-3

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

PROJECT SCRTD - UNION STATION DATE DRILLED 2/2/83 HOLE NO. 5-3
LOCATION BETWEEN TRACKS & BAGGAGE BLDG. GROUND ELEV. 292.8'
DRILLING CONTRACTOR CONVERSE - LAS VEGAS LOGGED BY B. INGHRAM DEPTH TO GROUND WATER
TYPE OF RIG HOLE DIAMETER 4¾" HAMMER WEIGHT AND FALL 320#, 36"
SURFACE CONDITIONS A.C. PAVED AREA TOTAL DEPTH 80.0' NO. CORE BOXES

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | 0.0 - 0.8' ASPHALT PAVEMENT | | | RD | | | SET UP 7:00 AM |
| | GW | 0.8 - 4.6' FILL - GRAVEL | | | CCI DR | | | BEGIN DRILLING 7:30 AM |
| 2.0 | | WELL GRADED GRAVEL TO 2½", COMPOSITION VARIABLE | 20K | | RD | | | WEATHER: COOL, RAINING |
| | | SUBANGULAR TO SUBROUNDED | | | | | | NO SAMPLE RECOVERY |
| 4.0 | | | | | | | | LOST CIRCULATION OF DRILLING FLUID |
| | ML | 4.6 - 14.0 CLAYEY SILT | | | | | | |
| | | GREEN-GRAY, MOIST; STIFF CONSISTENCY | 3-1 | | CCI DR | | | |
| 6.0 | | VARIABLE SAND CONTENT (LOCALLY 3 - 20%) | 15K | | RD | | | |
| 8.0 | | | | | | | | |
| | | TO VERY STIFF CONSISTENCY | | | | | | |
| 10.0 | | | | 15 | S | | | |
| | | | | 25 | P | | | |
| | | | | 45 | T | | | |
| 12.0 | | | | | RD | | | |
| 14.0 | SM | YOUNG ALLUVIUM | | | | | | |
| | | 14.0 - 19.0 SILTY SAND | | 20 | S | | | |
| 16.0 | | GRAY-BROWN, MOIST, MED. DENSE. | | 13 | P | | | |
| | | FINE TO V. FINE SAND WITH 20% SILT | | 20 | T | | | |
| 18.0 | | | | | RD | | | |
| | SW | 19.0 - 74.0 GRAVELLY SAND | | | | | | |
| 20.0 | | (SEE NEXT PAGE) | | | | | | |

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SHEET 1 OF 5

PROJECT SCRTD - UNION STA. DATE DRILLED 2/2/83 HOLE NO. 5-3

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SW | 19.0 - 74.0 GRAVELLY SAND (CONTINUED) | 37K | | CCI DR | | | NO SAMPLE RECOVERY |
| | | BROWN, VERY DENSE, WELL GRADED - MED. TO COARSE SAND (70%), GRAVEL TO 3"+ (30%) SUBANGULAR TO SUBROUNDED GRAINS, GRANITIC COMP. | | | RD | | | |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | 24-28' INTERMITTENT RIG CHATTER |
| | | ALTERNATING SAND & GRAVEL STRATA | | | | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | | | | CCI DR | | | POOR SAMPLE RECOVERY LARGE GRAVEL - SAMPLE DESTROYED BY HANDLING |
| | | | 80K | 9" | RD | | | |
| 32.0 | | | | | | | | |
| 34.0 | | | | | | | | |
| 36.0 | | | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | CHANGE OF SAMPLE COLOR TO DARK GRAY | | | CCI DR | | | SULPHUR ODOR LARGE GRAVEL IN SAMPLE - SAMPLE DESTROYED BY HANDLING |
| | | | 57K | | RD | | | |
| 42.0 | | | | | | | | RIG CHATTER AT 43' |
| 44.0 | | | | | | | | |

PROJECT SCRTD - UNION STA. DATE DRILLED 2/2/83 HOLE NO. 5-3

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | SW | 19.0 - 74.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| 46.0 | | | | | | | | |
| 48.0 | | | | | | | | |
| 50.0 | | | 3-2 41K | | CCI DR | | | SULPHUR ODOR IN SAMPLE |
| | | | | | RD | | | |
| 52.0 | | ALTERNATING SAND AND GRAVEL STRATA | | | | | | |
| | | | | | | | | 53' INTERMITTENT RIG CHATTER TO 58' |
| 54.0 | | | | | | | | |
| 56.0 | | | | | | | | |
| 58.0 | | | | | | | | |
| 60.0 | | | | | CCI DR | | | SLIGHT SULPHUR ODOR IN SAMPLE |
| | | | 82K | | RD | | | POOR RECOVERY - SAMPLE DESTROYED BY HANDLING |
| 62.0 | | | | | | | | |
| 64.0 | | LESS GRAVEL, LENSES FURTHER APART | | | | | | |
| 66.0 | | | | | | | | |
| 68.0 | | | | | | | | |

PROJECT SCRTD - Union Sta DATE DRILLED 2/2/83 HOLE NO. 5-3

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | SW | 19.0 - 74.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| 70.0 | | (SAND STRATA SAMPLED) | 3-3 85K | | CCI DR | | | SLIGHT SULPHUR ODOR IN SAMPLE |
| | | | | | RD | | | |
| 72.0 | | | | | | | | |
| 74.0 | SP | 74.0 - 80.0 SAND | | | | | | 74' EASIER DRILLING |
| | | DARK GRAY, MED. DENSE TO DENSE; POORLY GRADED, UNIFORM FINE TO V. FINE GRAINED, MICACEOUS | | | | | | |
| 76.0 | | SILTY SAND LENSES | | | | | | |
| 78.0 | | | | | | | | |
| | | | 3-4 35K | | CCI DR | | | SULPHUR ODOR IN SAMPLE |
| 80.0 | | END BORING 80.0 FT | | | | | | |

# SUMMARY BORING NO. 5-3

PROJECT SCRTD STATION HOLE UNION STATION DATE DRILLED 2/2/83.

OVERBURDEN DEPTH (FT.) 0.0 TO 80.0 (T.D.)

BEDROCK DEPTH (FT.) — TO — ~~(T.D.)~~.

WATER PRESS. TEST No; INTERVAL(S) — TO —, — TO —.

GROUND WATER DEPTH (FT.) — DATE —; — DATE —. ROTARY WASH BORING - COULD NOT DETERMINE

GAS Yes; DEPTH FIRST NOTICED 40', DATE 2/2/83. - SULPHUR ODOR

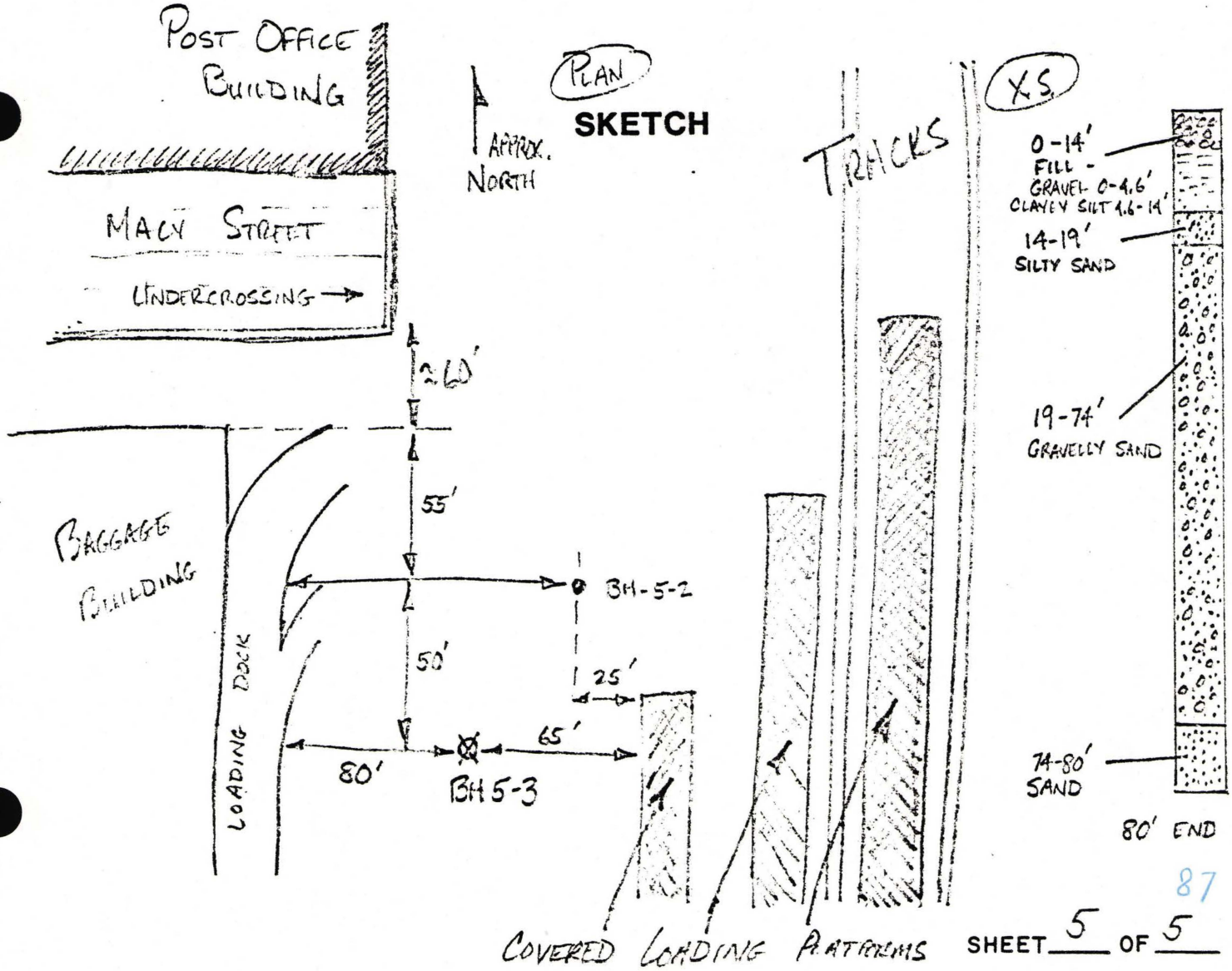
E-LOG No.

DOWN-HOLE SURVEY No.

CROSS-HOLE SURVEY No.

PVC CASING (I.D.): 4" — TO NONE; 3" — TO —; 2" — TO —.

GROUND ELEVATION REF. —

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SHEET 5 OF 5

# Converse Consultants

**Boring Log** 5-4

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

PROJECT SCRTD DATE DRILLED 1/31/83 HOLE NO. 5-4
LOCATION LOADING DOCK, INT. VIGNES & RAMIREZ STREETS GROUND ELEV. 280.6'
DRILLING CONTRACTOR CONVERSE - LAS VEGAS LOGGED BY B. INGHRAM DEPTH TO GROUND WATER
TYPE OF RIG HOLE DIAMETER 4¾" HAMMER WEIGHT AND FALL 320#, 36"
SURFACE CONDITIONS A.C. PARKING AREA TOTAL DEPTH 80.0' NO. CORE BOXES

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | 0.0 - 0.3 ASPHALT PAVEMENT | | | RD | | | BEGIN DRILLING 8:15AM |
| | ML | 0.3 - 3.5 FILL - SANDY SILT | | | | | | WEATHER: CLEAR, WARM |
| | | DARK BRN, MOIST, STIFF | 4-1 | | CCI DR | | | |
| 2.0 | | SILT - 50%, FINE TO MED. SAND 40%, 10% GRAVEL & BRICK CHUNKS | 16K | | RD | | | |
| | | 3.5 - 4.0 CONCRETE BLOCK | | | | | | |
| 4.0 | SM | - YOUNG ALLUVIUM - | | | | | | |
| | | 4.0 - 7.0 SILTY SAND | | | | | | |
| | | GRAY - BRN, MOIST, DENSE | | 30 | S | | | |
| 6.0 | | FINE TO VERY FINE - 70%, | J-1 | 18 | P | | | |
| | | SILT - 30% | | 24 | T | | | |
| | | | | | RD | | | |
| | SP | 7.0 - 18.0 SAND | | | | | | |
| 8.0 | | GRAY, MOIST, DENSE. | | | | | | |
| | | CLEAN, UNIFORM FINE SAND | | | | | | |
| 10.0 | | | 4-2 | | CCI DR | | | |
| | | | 17K | | RD | | | |
| 12.0 | | BECOMES COARSER TO MED. GRAINED | | | | | | |
| 14.0 | | | | | | | | |
| | | | | 5 | S | | | |
| 16.0 | | | J-2 | 4 | P | | | |
| | | | | 11 | T | | | |
| | | | | | RD | | | |
| 18.0 | | 18.0 - 28.0 GRAVELLY SAND | | | | | | |
| | SW | BRN / GRAY / WHT, MOIST, DENSE | | | | | | |
| | | MED. TO COARSE CLEAN SAND - 60%, | | | | | | |
| | | GRAVEL TO 2" - 40%, SUBROUNDED TO | | | | | | |
| 20.0 | | SUBANGULAR GRAINS. | | | | | | |
| | | GRANITIC / CRYSTALLINE COMPOSITION | | | | | | |

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PROJECT SCRTD - UNION STA. DATE DRILLED 1/31/83 HOLE NO. 5-4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SW | 18.0-28.0 GRAVELLY SAND (CONTINUED) | A-3 75K | | CCI DR | | | HIGHLY DISTURBED SAMPLE DUE TO LG. GRAVEL PIECES IN SAMPLE BARREL. |
| | | | | | RD | | | |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | |
| | | | 5" REFUSAL | 100 | SPT | | | |
| 26.0 | | | | | RD | | | |
| 28.0 | SP/CL | 25.0-41.0' SAND/SILTY CLAY GRAY, MOIST, DENSE CLEAN SAND; INTERBEDS OF DARK GRAY, STIFF, MOIST SILTY CLAY TO 31.5' | | | | | | |
| 30.0 | | | A-4 55K | | CCI DR | | | |
| | | | | | RD | | | |
| 32.0 | SP | 31.5' END CLAY INTERBEDS | | | | | | |
| 34.0 | | | | | | | | |
| | | | | 67 | S | | | |
| 36.0 | | | J-3 | 55 | P | | | |
| | | | | 80 | T | | | |
| | | | | | RD | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | 30K 0" REFUSAL | | | | | No Sample Recovery |
| | SW | 41.0-68.0 GRAVELLY SAND DARK GRAY, VERY DENSE SUBROUNDED GRAINS, WELLGRADED V. FINE TO COARSE SAND - 75% GRAVEL TO 1" - 25% HIGHLY MICACEOUS - DIORITIC COMPOSITION | | | | | | |
| 42.0 | | | | | | | | |
| 44.0 | | | | | | | | |

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SHEET 2 OF 5

PROJECT SCRTD - UNION STA. DATE DRILLED 1/31/83 HOLE NO. 5-4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | SW | 41.0-68.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| | | | | 75 | SPT | | | |
| 46.0 | | | | 78 | | | | |
| | | | 5"- | 100 | | | | |
| | | | | | RD | | | |
| 48.0 | | | | | | | | |
| 50.0 | | | | | | | | |
| | | 50.5' CLAY INTERBEDS DECREASING GRAVEL (TO ≈ 15%) | 4-5 70K | | CCI DR | | | 4 RINGS ONLY |
| | | | | | RD | | | SULPHUREOUS ODOR IN SAMPLE |
| 52.0 | | | | | | | | |
| 54.0 | | | | | | | | |
| | | | 3"- REFUSAL | 100 | SPT RD | | | SULPHUREOUS ODOR |
| 56.0 | | | | | | | | |
| 58.0 | | | | | | | | |
| 60.0 | | | 4-6 75K | -9" | CCI DR RD | | | SULPHUR, HYDROTHERMAL ODOR |
| 62.0 | | | | | | | | |
| 64.0 | | | | | | | | |
| | | | 1"- | 105 50 | SPT RD | | | SULPHUR ODOR |
| 66.0 | | | | | | | | |
| 68.0 | | | | | | | | |

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SHEET 3 OF 5

PROJECT SCRTD - UNION ST. DATE DRILLED 1/31/83 HOLE NO. 5-4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | SP | 68.0 - 75.0 SAND<br>GRAY, DENSE<br>UNIFORM MICACEOUS<br>FINE TO VERY FINE GRAINED | | | RD | | | |
| 70.0 | | | 4-7<br>25K | | CCI DR | | | SULPHUR/HYDROTHERMAL ODOR IN SAMPLE |
| | | | | | RD | | | |
| 72.0 | | | | | | | | |
| 74.0 | | | | | | | | |
| | SW | 75.0 - 80.0 GRAVELLY SAND<br>20% GRAVEL TO 1½"<br>GRAY, DENSE TO V. DENSE | | | | | | |
| 76.0 | | | | | | | | |
| 78.0 | | | | | | | | |
| | | | 4-8<br>75K | | CCI DR | | | STOP DRILLING 2:30 PM |
| 80.0 | | END BORING 80.0 FT | | | | | | |

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SHEET 4 OF 5

# SUMMARY BORING NO. 5-4

PROJECT 83-1101-41 STATION HOLE UNION STA. DATE DRILLED 1/31/83.

OVERBURDEN DEPTH (FT.) 0.0 TO 80.0 (T.D.)

BEDROCK DEPTH (FT.) NOT ENCOUNTED TO ~~(T.D.)~~.

WATER PRESS. TEST No; INTERVAL(S) — TO —, — TO —.

GROUND WATER DEPTH (FT.) ROTARY WASH - COULD NOT DETERMINE DATE —; — DATE —.

GAS Yes; DEPTH FIRST NOTICED 50', DATE 1/31. - SULFUR ODOR

E-LOG No.

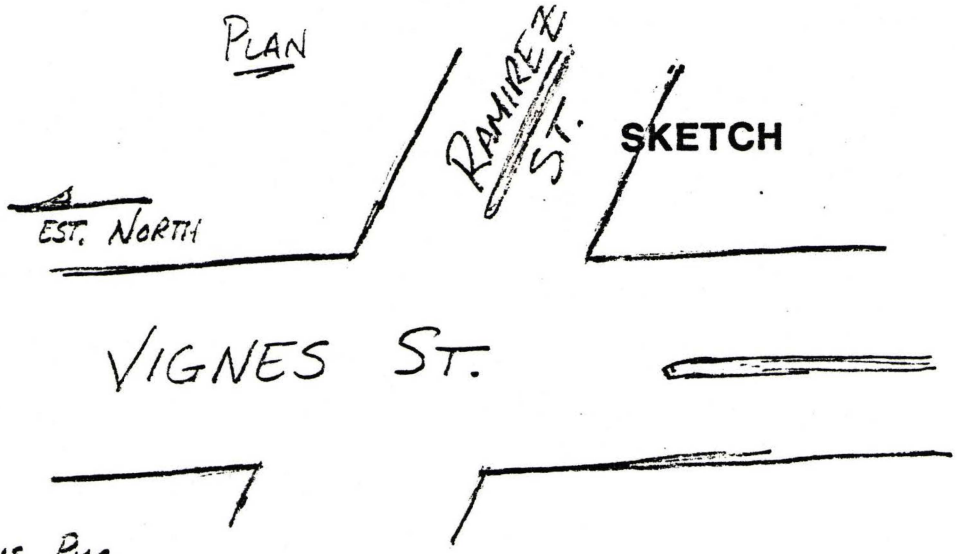
DOWN-HOLE SURVEY No.

CROSS-HOLE SURVEY No.

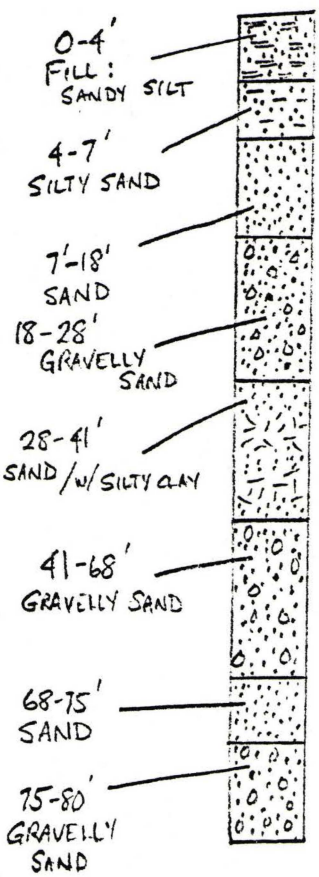
PVC CASING (I.D.): 4" — (NONE) TO —; 3" — TO —; 2" — TO —.

GROUND ELEVATION REF. —.

PLAN SKETCH




X.S.

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SHEET 5 OF 5

# Converse Consultants

**Boring Log** 5-5

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

PROJECT SCRTD - UNION STA. DATE DRILLED 2/1/83 HOLE NO. 5-5
LOCATION LOADING DOCK AT INT. VIGNES & RAMIREZ STS GROUND ELEV. 280.8'
DRILLING CONTRACTOR CONVERSE - LAS VEGAS LOGGED BY T. INGHRAM DEPTH TO GROUND WATER 27.9
TYPE OF RIG _____ HOLE DIAMETER 4¾" HAMMER WEIGHT AND FALL 320#, 36" (2-8-83)
SURFACE CONDITIONS A.C. PARKING AREA 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 - 0.3 · ASPHALT PAVEMENT | | | RD | | | SET UP 2:45 PM 1/31 |
| | ML & SM | 0.3 - 5.5 - FILL - SANDY SILT & SILTY SAND | | 16 | SPT | | | BEGIN DRILLING 7:15 AM 2/1/83 |
| 2.0 | | | | 25 | | | | WEATHER: CLEAR, WARM |
| | | MOTTLED & INTERMIXED | | 57 | | | | |
| | | MOIST, STIFF/M. DENSE | | | RD | | | |
| 4.0 | | W/ BRICK DEBRIS | | | | | | DRILLED TO 5' WITH 7" BIT FOR PIEZO INSTALLATION. 4¾" BIT BELOW |
| | | | 5-1 | | CCI | | | |
| 6.0 | SM | YOUNG ALLUVIUM | 3K | | DR | | | CONTACT CONTAINED WITHIN SAMPLE |
| | | 5.5 - 11.0 SILTY SAND | | | RD | | | |
| | | GRAY-BRN., MOIST, | | | | | | |
| 8.0 | | MED. DENSE | | | | | | |
| | | FINE TO V. FINE SAND - 70% | | | | | | |
| | | 30% SILT | | | | | | |
| 10.0 | | | | 10 | SPT | | | |
| | | | | 12 | | | | |
| | SP | 11.0 - 14.0 SAND | | 25 | | | | |
| 12.0 | | GRAY-BRN, MOIST, | | | RD | | | |
| | | M. DENSE TO DENSE | | | | | | |
| | | POORLY GRADED FINE SAND | | | | | | |
| | | W/ TRACE SILT | | | | | | |
| 14.0 | SW | 14.0 - 62.0 GRAVELLY SAND | | | | | | |
| | | BROWN, DENSE | | | | | | |
| | | WELL GRADED - | 5-2 | | CCI | | | GRAVELLY - |
| 16.0 | | MED. TO COARSE CLEAN SAND | 50K | | DR | | | DISTURBED SAMPLE |
| | | - 70%, GRAVEL TO 2" - 30% | | | RD | | | |
| | | SUBANGULAR TO SUBROUNDED | | | | | | |
| 18.0 | | GRAINS, GRANITIC COMP. | | | | | | |
| | | | | | | | | |
| 20.0 | | | | | | | | |

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SHEET 1 OF 6

PROJECT SCRTD - UNION STA. DATE DRILLED 2/1/83 HOLE NO. 5-5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SW | 14.0-62.0 GRAVELLY SAND (CONTINUED) | 5" RECOVAL | 100 | SPT / RD | | | |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | |
| | | 25.5-26.0 - LENSE OF FINE SAND IN SAMPLE | 5-3 36K | | CCI DR / RD | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | 30.0 COLOR CHANGE TO DK. GRAY INCREASED MAFIC CONTENT: GRANITIC/DIORITIC COMP. | 3" RECOVAL | 75/100 | SPT / RD | | | SLIGHT GAS ODOR |
| 32.0 | | | | | | | | |
| 34.0 | | | | | | | | |
| | | | 5-4 40K | | CCI DR / RD | | | SLIGHT GAS ODOR. GRAVELLY SAMPLE - ONLY 5 GOOD RINGS |
| 36.0 | | | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | J-1 9" | 25/75/100 | SPT / RD | | | |
| 42.0 | | | RECOVAL | | | | | |
| 44.0 | | | | | | | | |

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SHEET 1 OF 6

PROJECT SCRTD - UNION STA. DATE DRILLED 2/1/83 HOLE NO. 5-5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | | 14.0 - 62.0 GRAVELLY SAND (CONTINUED) | | | RD | | | |
| 46.0 | | | | | | | | |
| 48.0 | | | | | | | | |
| 50.0 | | | | | CCI DR | | | SULPHUR ODOR |
| | | | 75K | | RD | | | POOR SAMPLE RECOVERY - REMAINING SAMPLE DESTROYED IN HANDLING |
| 52.0 | | | | | | | | |
| 54.0 | | GRAD. DECREASING GRAVEL CONTENT | | | | | | |
| 56.0 | | | | | | | | |
| 58.0 | | | | | | | | |
| 60.0 | | | 5-5 | | CCI DR | | | SLIGHT SULPHUR ODOR |
| | | | 50K | | RD | | | |
| 62.0 | SP | 62.0 - 78.0 SAND | | | | | | |
| | | DARK GRAY, DENSE POORLY GRADED UNIFORM FINE TO VERY FINE GRAINED MICACEOUS | | | | | | |
| 64.0 | | | | | | | | |
| 66.0 | | | | | | | | |
| 68.0 | | | | | | | | |

PROJECT SCRTD - UNION STA. DATE DRILLED 2/1/83 HOLE NO. 5-5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | SP | 62.0-78.0 SAND (CONTINUED) | | | RD | | | |
| 70.0 | | | 5-6 50K | | CCI DR RD | | | STRONGER SULPHUR ODOR. POOR SAMPLE RECOVERY: 4½ RINGS ONLY |
| 72.0 | | | | | | | | |
| 74.0 | | | | | | | | |
| 76.0 | | | | | | | | |
| 78.0 | | 78.0-90.0 BOULDERS. CUTTINGS: MED. TO COARSE SUBANGULAR GRAINS GRANITIC COMPOSITION | | | | | | 78.0 - CHANGE IN DRILLING CONDITIONS - V. HARD DRILLING. FULL WT. OF RIG (10 TON) ON BIT. TOO HARD TO SAMPLE TO 90.0' |
| 80.0 | | | 60K-1" REFUSAL | | CCI DR RD | | | NO SAMPLE RECOVERY |
| 82.0 | | | | | | | | |
| 84.0 | | | | | | | | |
| 86.0 | | | 100K-0" REFUSAL | | RD | | | ATTEMPTED TO SAMPLE - UNSUCCESSFUL |
| 88.0 | | | J-2 CUTTINGS | | | | | JAR SAMPLE OF CUTTINGS TAKEN |
| 90.0 | CL | PUENTE FORMATION. 90.0-100.0 CLAYSTONE (SEE NEXT PAGE) | | | | | | EASY DRILLING FROM 90.0' CLAYSTONE CUTTINGS OBTAINED |
| 92.0 | | | | | | | | |

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SHEET 4 OF 6

PROJECT SCRTD - UNION STA DATE DRILLED 2/1/83 HOLE NO. 5-5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 92.0 | CL | 90.0-100.0 CLAYSTONE (CONTINUED) | | | RD | | | |
| 94.0 | | OLIVE-GRAY COLOR, MOIST. PLASTIC TO FRIABLE STRENGTH; SOFT FRIABLE HARDNESS, THINLY LAMINATED W/ SILTY CLAYSTONE - 20%. SANDSTONE BLEBS. | | | | | | |
| 96.0 | | TENDS TO FRACTURE ALONG LAMINATIONS | | | | | | |
| 98.0 | | | | | | | | |
| | | | 5-7 50K | | CCI DR | | | |
| 100.0 | | END BORING 100.0 FT PIEZOMETER SET TO 100' PERFORATED IN LOWEST 40' | | | | | | |

# SUMMARY BORING NO. 5-5

PROJECT SCRTD STATION HOLE UNION STA. DATE DRILLED 2/1/83.

OVERBURDEN DEPTH (FT.) 0.0 TO 90.0.

BEDROCK DEPTH (FT.) 90.0 TO 100.0 (T.D.).

WATER PRESS. TEST No; INTERVAL(S) — TO —, — TO —.

GROUND WATER DEPTH (FT.) 28.0' DATE 2/3/83; 27.9 DATE 2/8/83.

GAS YES; DEPTH FIRST NOTICED 30', DATE —. (SULPHUR ODOR)

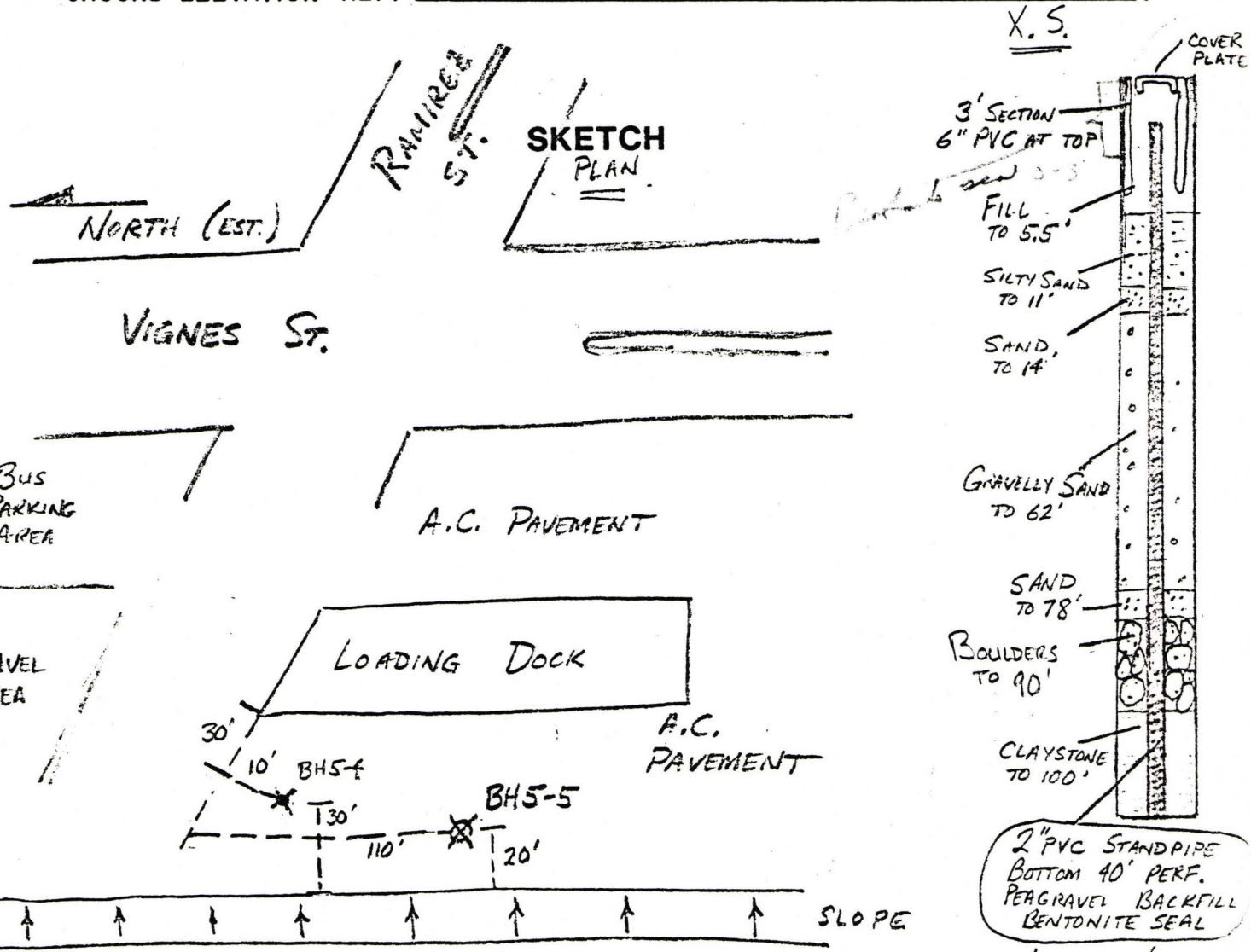
E-LOG No.

DOWN-HOLE SURVEY No.

CROSS-HOLE SURVEY No.

PVC CASING (I.D.): 4" — TO —; 3" — TO —; 2" 0.4 TO 100.0.

GROUND ELEVATION REF. —.



# Converse Consultants

**Boring Log** 6A

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

PROJECT SC VTD 82-1101-21 DATE DRILLED Feb 10 1983 HOLE NO. 6A
LOCATION 60' W. NORTH BOUND ON S-SIDE OF MACY Street GROUND ELEV. 290
DRILLING CONTRACTOR ANN Drilling LOGGED BY Dan Gillette DEPTH TO GROUND WATER 19.8' 2-14-83
TYPE OF RIG Mayhew HOLE DIAMETER 24" HAMMER WEIGHT AND FALL N/A
SURFACE CONDITIONS Concrete Pavement - Macy St TOTAL DEPTH 55.0' NO. CORE BOXES N/A

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | YOUNG ALLUVIUM | | | | | | |
| 0 | SP | 0.0-22.0 SAND AND GRAVEL | | | | | | MACY STREET CONCRETE PAVEMENT |
| 1 | | LIGHT BROWN (5YR 5/6) | | NO | | | | |
| | | SAND-75-85% MEDIUM AND | | SAMPLES | | | | |
| 2 | | COARSE; SILT 0-5%; | | | | | | 0.0-15.0 EXCESSIVE Boiling + CAVING |
| | | GRAVEL-10-15%, ½" SUB-ROUNDED | OBS | Groundwater | | | | |
| 3 | | TRACE (0-5%) COBBLES | | Hole | | | | |
| | | AND BOULDERS (to 24") | | | | | | |
| 4 | | MOIST, MEDIUM DENSE | | | | | | |
| 5.0 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |
| 8 | | | | | | | | |
| 9 | | | | | | | | |
| 10.0 | | | | | | | | |
| 1 | | | | | | | | |
| 2 | | 12.0-13.0 CLAY | | | | | | |
| | | 6" to 8" STREAK | | | | | | |
| 3 | | W/ TRACE GRAVEL | | | | | | |
| 4 | | | | | | | | 15.0-22.0 |
| 15.0 | | | | | | | | Hole Ravels |
| 6 | | 15.5-16.0 CLAY | | | | | | Slightly |
| | | 6" to 8" STREAK | | | | | | |
| 7 | | 17.0-18.0 Very Moist | | | | | | Water level at |
| 8 | | | | | | | | 19.8' AFTER 80 hours |
| 9 | | | | | | | | 18.0 - FIRST GROUND |
| | | 19.0-19.5 Gravel | | | | | | WATER SEEP ENCOUNTERED |
| 20.0 | W.L. 19.8 ▼ | 2" lens; WET | | ▼ | | | | SHEET 1 OF 4 |

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PROJECT SC ETD 82-1101-21 DATE DRILLED FEB 10 1983 HOLE NO. 6A

| DEPTH | CLASS. | FIELD DESCRIPTION YOUNG ALLUVIUM | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SP | 0.0-22.0 SAND AND GRAVEL | | | | | | |
| 1 | | | | | | | | |
| 2 | SM | 22.0-55.0 SILTY SAND OLIVE GRAY (5G 4/1) SAND 60-80% fine Slightly micaceous Moist to Very Moist; Medium Dense | | | | | | $H_2O$ Serp @ 22.0' 1gpm < NE side of hole |
| 3 | | | | | | | | |
| 4 | | | | | | | | 22.0-55.0 Hole Stands Well |
| 25.0 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |
| 8 | | 28.0 Dark Greenish Gray (5G 4/1) | | | | | | WATER level at 29.8' 14 hours AFTER completing to T.D. 55' |
| 9 | | | | | | | | |
| 30.0 | | | | | | | | |
| 1 | | | | | | | | |
| 2 | | | | | | | | |
| 3 | | | | | | | | |
| 4 | | | | | | | | |
| 35.0 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |
| 8 | | | | | | | | |
| 9 | | | | | | | | |
| 40.0 | | | | | | | | |
| 1 | | | | | | | | |
| 2 | | | | | | | | |
| 3 | | | | | | | | |
| 44.0 | | | | | | | | |

SHEET 2 OF 4

PROJECT SC RTD 83-1101-21 DATE DRILLED FEB 10 1983 HOLE NO. 6A

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 14.0 | | YOUNG ALLUVIUM | | | | | | |
| 45.0 | SM | 22.0-55.0 SILTY SAND | | | | | | 22.0-55.0 Hole Stands Well |
| 6 | | | | | | | | |
| 7 | | 46.0 CLAY 46" Shrnk | | | | | | |
| 8 | | | | | | | | |
| 9 | | | | | | | | |
| 50.0 | | | | | | | | |
| 1 | | 51.0 SILT 4.6" Shrnk | | | | | | |
| 2 | | | | | | | | |
| 3 | | | | | | | | |
| 4 | | | | | | | | |
| 55.0 | | END BORING 55.0' | | | | | | SPECIAL HOLE CLOSURE 2-14-83 Hole filled to 25' with pea gravel 2-15-83 hole pumped to 25' (Top of gravel) & slurry placed to -1' 2-16-83 placed concrete cap to surface |

SHEET 3 OF 4

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# SUMMARY BORING NO. 6A

82-1161-21

PROJECT SC DTT STATION HOLE NO DATE DRILLED 6A.

OVERBURDEN DEPTH (FT.) 0.0 TO 55.0 T.D.

BEDROCK DEPTH (FT.) NONE TO _____ (T.D.).

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

GROUND WATER DEPTH (FT.) 19.8 DATE 2/14/83; _____ DATE _____.

GAS NO; DEPTH FIRST NOTICED _____, DATE _____.

E-LOG NO.

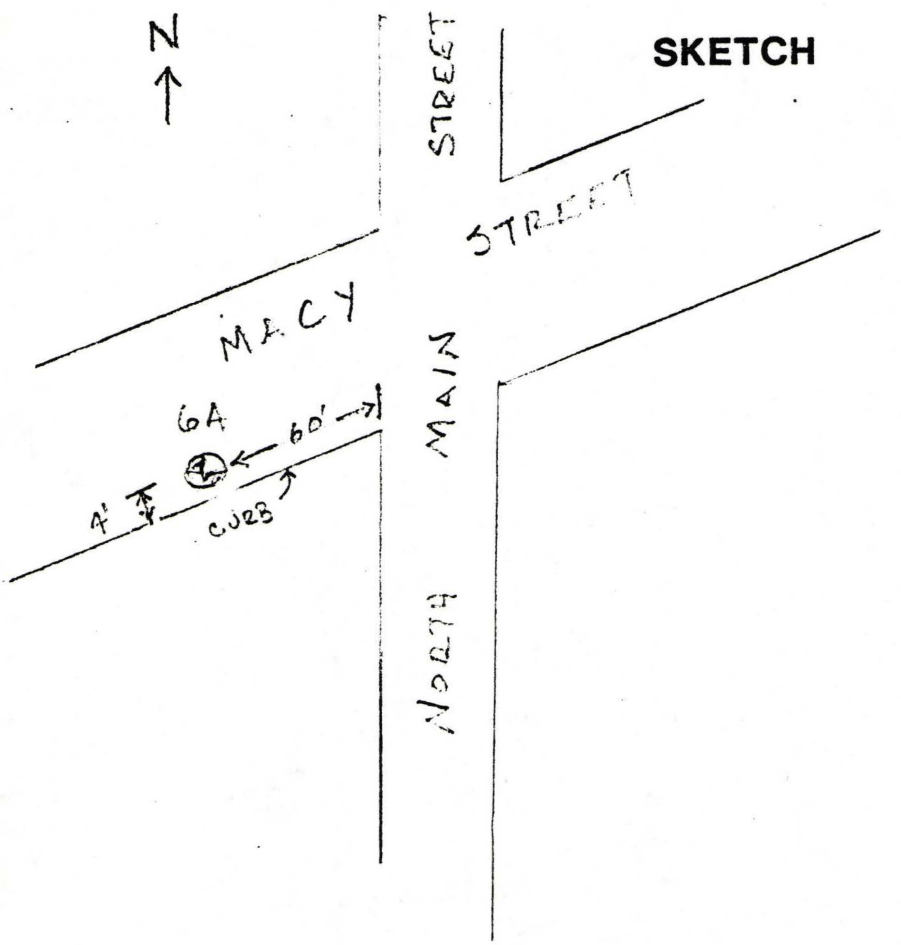
DOWN-HOLE SURVEY NO.

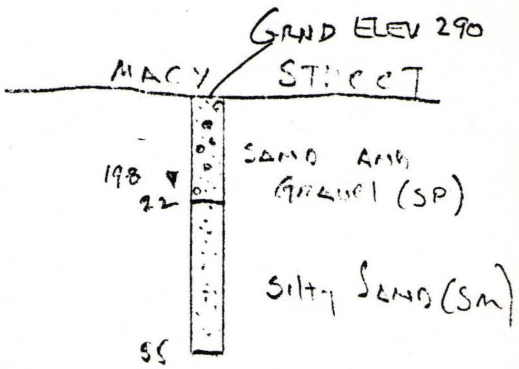
CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" NO TO _____; 3" NO TO _____; 2" NO TO _____.

GROUND ELEVATION REF. L.A City Drainage Map #516 (Rev 10/24/77) Scale 1"=400'

## SKETCH

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SHEET 4 OF 4

1981 BORING LOGS

**ConverseWardDavisDixon**
**Earth Sciences Associates**
**Geo/Resource Consultants**

# Boring Log 4

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

PROJECT SCRTD DATE DRILLED 2-9-81 HOLE NO. 4
LOCATION 210' E. OF RAMIREZ ST. & CENTER ST. INTERSEC. ON S. SIDE OF RAMIREZ GROUND ELEV. 279'
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Stephen M. Testa DEPTH TO GROUND WATER ___
TYPE OF RIG Failing 1500 HOLE DIAMETER ___ HAMMER WEIGHT AND FALL 140lb. - 30in.
SURFACE CONDITIONS concrete sidewalk TOTAL DEPTH 150.0' NO. CORE BOXES 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | 0.0-0.2 CONCRETE:<br>0.2-14.0 FILL:<br>Primarily crushed asphalt and brick.. | | | AD | | | Started drilling at 8:45; cloudy day, light drizzle; augered down to 10.0'. |
| 2.0 | | | | | | | | |
| 4.0 | | | | | | | | |
| 6.0 | | | | | | | | |
| 8.0 | | | | | | | | |
| 10.0 | | | | | RD | | | |
| 12.0 | | | | | | | | |
| 14.0 | SP | 14.0-21.0 GRAVELLY SAND:<br>olive black (5Y2/1); non-plastic fines (<5%); fine to medium sand (80%); gravel (15%) up to 2.0mm. in max. dia.; wet; very dense; poorly graded; odorous (oil); dilatancy quick; toughness none. | J-1 | 19<br>30<br>50 | SS<br>RD | | | SPT at 15.0'; 1.0/1.5 recovery. |
| 16.0 | | | | | | | | |
| 18.0 | | | | | | | | |
| 20.0 | | | | | | | | |

SHEET 1 OF 8

PROJECT SCRTD DATE DRILLED 2-9-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | SP | 14.0-21.0 GRAVELLY SAND: (CONTINUED) | | 50 | DR | | | (CONTINUED) 10:13 converse sample at 20.0', no recovery; SPT at 20.5', no recovery. moderate to heavy rod chatter (episodic) from 21.0 to 30.0. |
| | | | | 50/5" | SS | | | |
| | GP | 21.0-101.5 SANDY GRAVEL: | | | RD | | | |
| 22.0 | | numerous cobbles and boulders. | | | | | | |
| 24.0 | | | | | | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | | | 50/4" | CS | | | SPT at 30.0', no recovery. 10:45 |
| | | | | | RD | | | |
| 32.0 | | gravels range up to 6.0mm in max. dia., consist primarily of granitics and metamorphics; subangular to subrounded; poorly graded. | J-2 | | | | | caving is preventing advancement of hole; added two sacks of bentonite; cuttings sampled at 32.0. |
| 34.0 | | | | | | | | |
| 36.0 | | | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | | 50/3" | SS | | | SPT at 40.0'; no recovery; much difficulty getting back into hole. 12:10 |
| | | | | | RD | | | |
| 42.0 | | | | | | | | |
| 44.0 | | | | | | | | |

SHEET 2 OF 8

PROJECT SCRTD DATE DRILLED 2-9-81/2-10-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | GP | 21.0-101.5 SANDY GRAVEL: (CONTINUED) numerous cobbles and boulders. | | | RD | | | (CONTINUED) |
| 46.0 | | | | | | | | moderate to heavy rig shaking continues. |
| 48.0 | | | | | | | | |
| 50.0 | | | | | | | | 2:10 due to heavy rig shaking, no SPT was taken at 50.0! |
| 52.0 | | | | | | | | |
| 54.0 | | | | | | | | |
| 56.0 | | | | | | | | |
| 58.0 | | | | | | | | |
| 60.0 | | continued, numerous cobbles and boulders. | | SPT | SC RD | | | SPT at 60.0', no recovery. |
| 62.0 | | | | | | | | Stopped drilling at a depth of 70.0'. 5:00 2-10-81 7:00 Resumed drilling at 7:00AM, clear day. |
| 64.0 | | | | | | | | |
| 66.0 | | | | | | | | |
| 68.0 | | | | | | | | |

SHEET 3 OF 8

PROJECT SCATD DATE DRILLED 2-10-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | GP | 21.0-101.5 SANDY GRAVEL: (CONTINUED) primarily cobbles and boulders. | | | RD | | | (CONTINUED) moderate to heavy rig shaking continues |
| 70.0 | | | | 50/2" | SS | | | SPT at 70.0', no recovery; much difficulty getting back into hole. |
| 72.0 | | | | | | | | |
| 74.0 | | | | | | | | |
| 76.0 | | | | | | | | |
| 78.0 | | | | | | | | |
| 80.0 | | continued; primarily cobbles and boulders. | | 50/4" | SS RD | | | SPT at 80.0, no recovery. |
| 82.0 | | | | | | | | |
| 84.0 | | | | | | | | |
| 86.0 | | | | | | | | |
| 88.0 | | | | | | | | |
| 90.0 | | | | | | | | |
| 92.0 | | | | | | | | |

SHEET 4 OF 8

PROJECT SCRTD DATE DRILLED 2-10-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 92.0 | GP | 21.0-101.5 SANDY GRAVEL: (CONTINUED) fine to coarse sand with numerous cobbles and boulders. | | | RD | | | (CONTINUED) moderate to heavy rig shaking to 101.5' |
| 94.0 | | | | | | | | |
| 96.0 | | | | | | | | |
| 98.0 | | | | | | | | |
| 100.0 | | | | | | | | |
| 102.0 | | PUENTE FORMATION 101.5-150.0 CLAYSTONE: primary olive gray (5Y3/2) micaceous claystone (90%), very fine sand (10%); compositional banding apparent. PHYSICAL CONDITION: massive; soft to friable hardness; plastic to weak strength; fresh; tends to fracture along bedding planes. continued. | | | PB | 1 | 2.8/2.8 | gas check 0.0% LEL; no gas encountered; continuous pitcher barrel sampling from 102.5; had to ream hole out to 7.6" with tri-cone bit from 12:45 to 1:20 PM in order to pitcher barrel; pitcher barrel samples are at 2.8' intervals. |
| 104.0 | | | | | | 2 | 2.8/2.8 | |
| 106.0 | | | Box #1 | | | | | |
| 108.0 | 80 FD | from 107.8, very thin to medium compositional banding, contacts sharp and parallel; hairline fractures at 108.4 dipping 80° to core axis with an oriented strike 70° to bedding. | | | | 3 | 2.8/2.8 | 200 psi'. pocket penetrometer >4.5 kg/cm2 |
| 110.0 | | | | | | | | |
| 112.0 | 70 | primarily claystone exhibiting compositional banding; at 112.1 clay filled fracture with undeterminable offset | | | | 4 | 2.8/2.8 | 200 psi' |
| 114.0 | 80 70 | continued; very thin to medium compositional banding evident in claystone (depositional features); from 114.7 alternating claystone and very thin grayish brown (5YR 3/2) fine sandstone lamina | Box #2 | | | 5 | 2.8/2.8 | 200 psi' |
| 116.0 | | | | | | | | |

SHEET 5 OF 8

PROJECT SCRTD DATE DRILLED 2-10-81/2-11-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 116.0 | | 101.5 - 150.0 CLAYSTONE: (CONTINUED) | Box # 2 | | PB | 5 | 2.8/2.8 | (CONTINUED) |
| | | PHYSICAL CONDITION: (CONT'D) massive; soft to friable hardness; plastic to weak strength fresh; tends to fracture along bedding planes, notably sandstone lamina. | | | | | | 200 psi. |
| 118.0 | | primarily claystone with wavy parallel alternating very thin to thin fine sandstone (5%) lamina. | S-1 | | | 6 | 2.8/2.8 | |
| 120.0 | | 119.3 to 119.9 intensely fractured, (orientation undeterminable). | Box # 2 | | | 7 | 2.4/2.4 | 200 psi. |
| 122.0 | 70 | continued; compositional banding apparent in claystone. | | | | 8 | 2.6/2.6 | 200 psi. |
| 124.0 | | | | | | | | Stopped drilling at a depth of 124.9' at 5:00 PM. |
| 126.0 | 70 | continued; primarily claystone (80-85%), fine sandstone (10-15%). | Box # 3 | | | | | 2-11-81 5:00 7:00 Resumed drilling at 7:00 AM; clear day. |
| | | | | | | 9 | 2.8/2.8 | |
| 128.0 | 70 | continued. | | | | | | 7:20 |
| | | | | | | 10 | 2.8/2.8 | 200 psi. |
| 130.0 | | | | | | | | |
| | | continued; compositional banding apparent in claystone (bluish green clay). | Box # 4 | | | 11 | 2.8/2.8 | 200 psi. pocket penetrometer 4.5 kg/cm2 |
| 132.0 | | | | | | | | |
| 134.0 | | | S-2 | | | 12 | 2.5/2.8 | 200 psi. 8:00 8:15 |
| 136.0 | | continued. | | | | | | |
| | | | Box # 4 | | | 13 | 2.8/2.8 | 200 psi. 8:45 9:05 |
| 138.0 | | continued. | | | | | | |
| | | | | | | 14 | 2.8/2.8 | 200 psi. |
| 140.0 | | | | | | | | SHEET 6 OF 8 |

PROJECT SCRTD DATE DRILLED 2-11-81 HOLE NO. 4

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 140.0 | | 101.5 - 150.0 CLAYSTONE: (CONTINUED) 140.7 - 141.7 well cemented very fine greenish gray (5G6/1) sandstone. PHYSICAL CONDITION: continues as previously described. | Box #4 | | PB | 14 | 2.8/2.8 | (CONTINUED) |
| 142.0 | | | | | | | | 200 psi. 9:33 |
| | | | | | | 15 | 1.6/2.8 | pocket penetrometer >4.5 kg/cm2 |
| 144.0 | 10-20 / 1 | 144.5 to 145.3 grayish brown (5YR3/2) fine to medium sand, partially saturated with oil. from 145.3 primarily mottled claystone (70%); oil saturated fine to medium sand (30%). | Box #5 | | | | | 200 psi. petroleum sample 145 - 146 |
| 146.0 | | | | | | 16 | 2.5/2.8 | |
| 148.0 | | | S-3 | | | 17 | 2.7/2.7 | Terminated hole at a depth of 150.0' at 11:20 AM; E-log conducted from 11:30 to 2:00 P.M. 11:2 |
| 150.0 | B.H. | TERMINATE HOLE 150' | | | | | | Installed 150.0' of 2" PVC piezometer (PVC slotted from 110.0' to 145') and backfilled hole with pea gravel. 5' BENTONITE PLUG AT SURFACE. Water sampled 2/19/81 |

SHEET 7 OF 8

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Summary Data
Boring No. 4

# SUMMARY BORING NO. 4

PROJECT SCRTD STATION HOLE NB DATE DRILLED 2-9-81/2-11-81.

OVERBURDEN DEPTH (FT.) 0.0 TO 101.5.

BEDROCK DEPTH (FT.) 101.5 TO 150.0 (T.D.).

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

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

GAS TRACE; DEPTH FIRST NOTICED —, DATE —.

E-LOG YES.

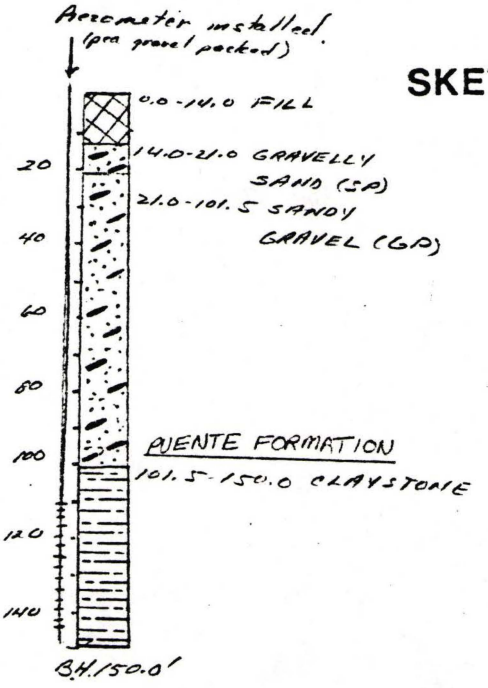
DOWN-HOLE SURVEY NO.

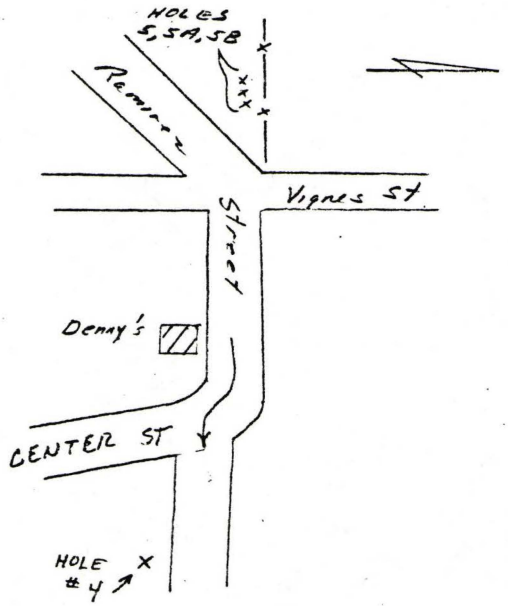
CROSS-HOLE SURVEY NB.

PVC CASING (I.D.): 4" — TO —; 3" — TO —; 2" 0.0' TO 150.0'.

GROUND ELEVATION REF. 279'.

## SKETCH




SHEET 8 OF 8

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**ConverseWardDavisDixon**
**Earth Sciences Associates**
**Geo/Resource Consultants**

# Boring Log 5

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

PROJECT SCRTD DATE DRILLED 12/80 - 1/4/80 HOLE NO. 5
LOCATION at intersection of Vignes & Ramirez St., Los Angeles GROUND ELEV. 280
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Stephen M. Testa DEPTH TO GROUND WATER
TYPE OF RIG Failing 1500 HOLE DIAMETER 6 7/8, 4 7/8 HAMMER WEIGHT AND FALL 140lb - 30in
SURFACE CONDITIONS asphalt roadway TOTAL DEPTH 150.0' NO. CORE BOXES 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 2.0 | GP | 0.0 - 0.3 ASPHALT;<br>0.3 - 52.0 SANDY GRAVEL; non-plastic fines (<5%); medium- to coarse sand (40%); gravel (55%); gravel primarily composed of granitics ranging up to 6" in max dia.; poorly graded. | | | RD | | | Started drilling at 900 AM., cloudy day.<br>moderate to heavy rig chatter from 3.0'; averaging 300 psi; did not begin sampling schedule until 52.0' due to the cobbly and gravelly nature of the material. |
| 4.0 | | | | | | | | |
| 6.0 | | | | | | | | |
| 8.0 | | | | | | | | |
| 10.0 | | | | | | | | |
| 12.0 | | | | | | | | |
| 14.0 | | | | | | | | |
| 16.0 | | | | | | | | |
| 18.0 | | | | | | | | |
| 20.0 | | | | | | | | |

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SHEET 1 OF 8

PROJECT SCATD DATE DRILLED 1-2-80 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20.0 | GP | 0.3 - 52.0 SANDY GRAVEL: (CONTINUED) | | | RD | | | (CONTINUED) moderate to heavy rig chatter from 2.0' averaging 300 psi |
| 22.0 | | | | | | | | |
| 24.0 | | | | | | | | |
| 26.0 | | | | | | | | |
| 28.0 | | | | | | | | |
| 30.0 | | | | | | | | |
| 32.0 | | | | | | | | |
| 34.0 | | | | | | | | |
| 36.0 | | | | | | | | |
| 38.0 | | | | | | | | |
| 40.0 | | | | | | | | |
| 42.0 | | | | | | | | |
| 44.0 | | | | | | | | |



SHEET 2 OF 8

PROJECT SCRTA DATE DRILLED 1-2-80 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44.0 | GP | 0.3 - 52.0 SANDY GRAVEL: (CONTINUED) | | | | RD | | (CONTINUED) heavy rig shaking to 52.0'; averaging 300 psi |
| 46.0 | | | | | | | | |
| 48.0 | | | | | | | | |
| 50.0 | | | | | | | | |
| 52.0 | SP | 52.0 - 55.0 GRAVELLY SAND: medium dark gray (N4), plastic fines (<5%); medium-to coarse-sand (75%); gravel up to ½ inch in max. dia. (20%), primarily granitics; toughness none; dilatancy none; odor apparent (similar to thatopa cesspool). | J-1 | 47<br>50/4" | SS<br>RD | | | SPT at 52.0'; .5/1.5 recovery<br>easy drilling from 53.0 to 55.0. heavy rig shaking from 55.0 to 60.0' |
| 54.0 | | | | | | | | |
| | GP | 55.0 - 84.2 SANDY GRAVEL: | | | | | | |
| 56.0 | | | | | | | | |
| 58.0 | | | | | | | | |
| 60.0 | | cuttings primarily gravel of granitic composition. | | 40<br>50/2" | SS<br>RD | | | SPT at 60.0'; no recovery.<br>heavy rig shaking from 60.6 to 66.0' |
| 62.0 | | | | | | | | |
| 64.0 | | | | | | | | |
| 66.0 | | medium dark gray (N4) very fine-grained sand; moist | J-2 | 8<br>20<br>50/1" | SS<br>RD | | | SPT at 66.0'<br>.3/1.5 recovery |
| 68.0 | | | | | | | | |

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SHEET 3 OF 8

PROJECT SCRTD DATE DRILLED 12-81/1-3-81 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 68.0 | | 55.0-84.2 SANDY GRAVEL: (CONTINUED) | | | RD | | | (CONTINUED) |
| 70.0 | | | | 50 50/1" | DR | | | Converse sampler at 70.0'; no recovery; clutch dropped. |
| | | | | | RD | | | moderate to heavy rig shaking from 70.6 to 84.2. |
| 72.0 | | | | | | | | |
| 74.0 | | | | | | | | |
| 76.0 | | cuttings primarily gravel of granitic composition | | | | | | heavy rig shaking at 75.0'; cuttings obtained. |
| 78.0 | | | | | | | | |
| 80.0 | | | | | | | | heavy rig shaking at 80.0' |
| 82.0 | | | | | | | | |
| 84.0 | | PIENTE FORMATION<br>84.2-150.0 CLAYSTONE: | | | | | | rosy drilling from 84.2; claystone cuttings and in top of bit |
| | 40°/43 | Wavy, parallel bedding; primarily olive gray (5Y 3/2) claystone (50%) with alternating very thin to thin lamina of dusky yellowish brown (10YR 2/2) siltier claystone (30%) and light gray (N7) friable sandstone (20%); micaceous; claystone fossiliferous (<1%); opposite dip orientation apparent in PB-1: | Box #1 | | PB | 1 | 1.0/1.0 | 2-3-81<br>Resumed drilling at 85.0'; clear clay; about 10.0' of caving, cleaned out hole with pitcher barrel. |
| 86.0 | 47°/ | | | | PB | 2 | 1.5/2.5 | pitcher barrel from 85.0-86.0; 86.0 to 88.5. |
| 88.0 | | PB-1 40° 43° | | | PB | 3 | 1.6/2.5 | pitcher barrel from 88.5 to 91.0; 91.0 to 93.5 |
| 90.0 | 50°/ | PHYSICAL CONDITION:<br>soft to friable hardness; plastic to friable strength; fresh; tends to fracture along bedding planes, notably friable sandstone beds and lamina. | | | PB | 4 | 2.2/2.5 | |
| 92.0 | | sandstone blebs apparent, | | | | | | |

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SHEET 4 OF 8

PROJECT SCRTD DATE DRILLED 1-3-80 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 92.0 | 52° Y | 842-150.0 CLAYSTONE: CONTINUED<br>PHYSICAL CONDITIONS: (continues to be the same as previous description)<br>folding of lamina apparent at 92ft; primarily claystone from 93.5 to 100.4, some faintly discernable very thin discontinuous sandstone lamina. | | | PB | 4 | 2.2/2.5 | (CONTINUED)<br>pocket penetrometer >4.5 2/9/81 |
| 94.0 | | | Box #1 | | PB | 5 | 1.9/2.5 | pitcher barrel from 93.5 to 96.0; 100 psi. |
| 96.0 | 40° Y | thin sandstone lamina at 96.6, 97.1, | | | PB | 6 | 2.0/2.5 | pitcher barrel from 96 to 98.5; 100 psi |
| 98.0 | 20-30° Y | | | | PB | 7 | 1.9/2.5 | pitcher barrel from 98.5 to 101; 100 psi.<br>pocket penetrometer >4.5 2/9/81 |
| 100.0 | | | Box #2 | | | | | |
| 102.0 | 51° Y | alternating very thin to medium lamina of claystone, sandstone and silty claystone; some lamina truncated, x-bedding (upright). | | | PB | 8 | 1.9/2.5 | pitcher barrel from 101 to 103.5; 100 psi. |
| 104.0 | 40° Y | repeating sequence of very thin to thin sandstone, claystone and silty claystone; organics (trace <1%). | | | PB | 9 | 1.5/2.5 | pitcher barrel from 103.5 to 106; 100 psi. |
| 106.0 | 43° Y | primarily claystone; wavy discontinuous sandstone, dark green wavy stringers (up to 2cm) from 108.0; preferred orientation dipping about 43° | | | PB | 10 | 2.5/2.5 | pitcher barrel from 106 - 108.5; 100 psi.<br>pocket penetrometer >4.5 2/9/81 |
| 108.0 | | | | | | | | |
| 110.0 | 43° Y | primarily claystone, silty claystone rounded concretions apparent, randomly distributed from 108.5 - 112.5 | Box #3 | | PB | 11 | 2.5/2.5 | pitcher barrel from 108.5 to 111.0; 100 psi |
| 112.0 | | primarily claystone with occassional discontinuous wavy sandstone up to 4cm in max. thickness from 111.0 - 116.0; variable bedding 5-40° | | | PB | 12 | 1.9/2.5 | pitcher barrel from 111.0 to 113.5; 100 psi<br>gas check; no gas encountered 0.0% LEL. |
| 114.0 | | sandstone lamina occassionally discontinuous | | | PB | 13 | 2.5/2.5 | pitcher barrel from 113.5 to 116.0; 100 psi. |
| 116.0 | | | | | | | | 116 |

SHEET 5 OF 8

PROJECT SCRTD DATE DRILLED 1-3-80 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 116.0 | | 114.2-150.0 CLAYSTONE: (CONTINUED) primarily claystone with alternating wavy, parallel very thin to medium lamina of sandstone, siltstone claystone with minor siltstone; micaceous; fossiliferous (<1%) claystone. | | | PB | 14 | 2.5/2.5 | (CONTINUED) pitcher barrel from 116 to 118.5; 100psi. |
| 118.0 | 44° Y | PHYSICAL CONDITION: (continues as previously described) occassional discontinuous, non-parallel lamina; folding apparent, depth 119.0; | Box #3 | | PB | 15 | 1.7/2.5 | pitcher barrel from 118.5 to 121; 100psi. |
| 120.0 | 48° Y | 48° silty claystone / claystone / sandstone; 10-20° | | | PB | 16 | 1.3/2.5 | pitcher barrel from 121 to 123.5; 100psi. |
| 122.0 | | bedding planes sharp, dip of bedding planes variable (10-46°), folding apparent. 46° 10mm. sandstone contact 10-20° | | | PB | 17 | 1.9/2.5 | pitcher barrel from 123.5 to 126; 100psi. |
| 124.0 | 46° Y | folding apparent from 125-126.0'. | | | PB | 18 | 1.7/2.5 | pitcher barrel from 126 to 128.5; 100psi. |
| 126.0 | 46° Y | sharp contacts between alternating lamina; cross-bedding evident (sequence is upright) | Box #4 | | PB | 19 | 1.8/2.5 | pitcher barrel from 128.5 to 131; 100psi. pocket penetrometer > 4.5 2/9/81 |
| 128.0 | | wavy parallel alternating lamina of sandstone, claystone & silty claystone; dark yellowish brown (10YR 4/2) siltstone (1.0mm) at 129.6; graded bedding apparent in very fine- to fine-grained sandstone lamina at 129.7 - 130.0 | | | PB | 20 | 1.3/2.5 | pitcher barrel from 131 to 133.5; 200psi. |
| 130.0 | 46° Y | | | | PB | 21 | 1.4/2.8 | pitcher barrel from 133.5 to 136.3; 200psi. |
| 132.0 | | alternating lamina of sandstone, claystone, silty claystone and siltstone; clay filled fracture & hairline fractures with offsets apparent at 132.0; prominent fracture plane dips 47° 46° | | | PB | 22 | 2.0/2.7 | pitcher barrel from 136.3 to 139; 200 psi; 139.0 to 141.8; 200psi. |
| 134.0 | 46° Y | some folding evident in sandstone lamina at 134.5 sandstone | | | PB | 23 | 1.9/2.8 | |
| 136.0 | | | | | | | | |
| 138.0 | 42° Y | cross-bedding apparent (sequence is upright) PHYSICAL CONDITION: soft to friable hardness; plastic to friable strength; fresh; tends to fracture along bedding planes, notably friable sandstone lamina. | Box #5 | | | | | |
| 140.0 | | | | | | | | |

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SHEET 6 OF 8

PROJECT SCRTD DATE DRILLED 12-3-81 HOLE NO. 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 140 | 44 Y | 84.2-150.0 CLAYSTONE: (CONTINUED) wavy, parallel very thin to medium alternating lamina of claystone, subordinate sandstone and siltstone and minor siltstone; variable thickness of lamina; cross-bedding apparent (sequence is upright). primarily claystone to 142.3', very fine grained sandstone from 142.3-142.8, 143.1-145.3. | | | PB | 23 | 1.9/2.8 | (CONTINUED) pocket penetrometer in sand lenses 3.5 12/9/81 |
| 142 | | | | | PB | 24 | 1.5/2.7 | pitcher barrel from 141.8 to 144.5; 200 psi. |
| 144 | | alternating wavy parallel lamina; sandstone from 144.5-144.7, 145.1-145.6, 145.9-146.2. PHYSICAL CONDITION: soft to friable hardness; plastic to friable strength; fresh; tends to fracture along bedding planes, notably sandstone laminae. | Box #5 | | PB | 25 | 1.8/2.8 | pitcher barrel from 144.5 to 147.3; 200 psi. |
| 146 | | | | | PB | 26 | 2.0/2.7 | pitcher barrel from 147.3 to 150.0; 200 psi. |
| 148 | 44 Y | primarily claystone from 147.3 with fine-to coarse-grained sandstone at 147.3-147.6, 148.0-148.4, 148.6-148.9 and 149.1-149.3. | | | | | | Terminated hole at 150.0', 12-3-81 |
| 150 | B.H. | | | | | | | 5:30 7:00 |
| | | | | | | | | 12-4-81 Installed 100.0' of 4" PVC and grouted. Pushed 3.0' length of 6" I.D. PVC ½ below asphalt surface; steel H₂O cover was then set flush with concrete surface. |

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SHEET 7 OF 8

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Summary Data
Boring No. 5

## SUMMARY BORING NO. 5

PROJECT 80-1280-22 STATION HOLE YES DATE DRILLED 1-2-81/1-4-81.

OVERBURDEN DEPTH (FT.) 0.0' TO 84.2'.

BEDROCK DEPTH (FT.) 84.2' TO 150.0' (T.D.).

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

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

GAS NO; DEPTH FIRST NOTICED ____, DATE ____.

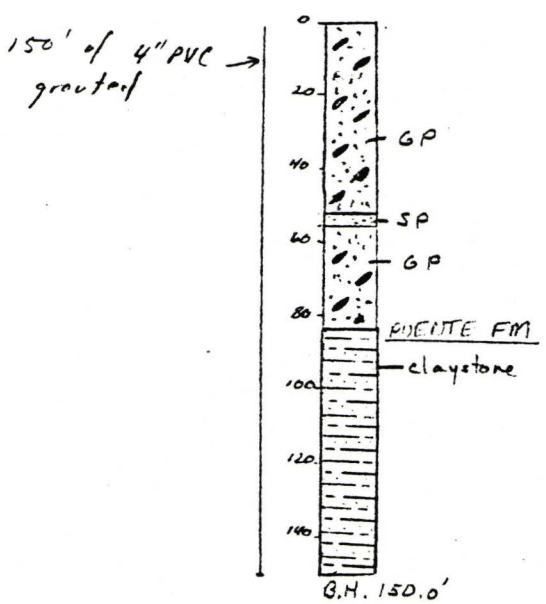
E-LOG YES.

DOWN-HOLE SURVEY YES.

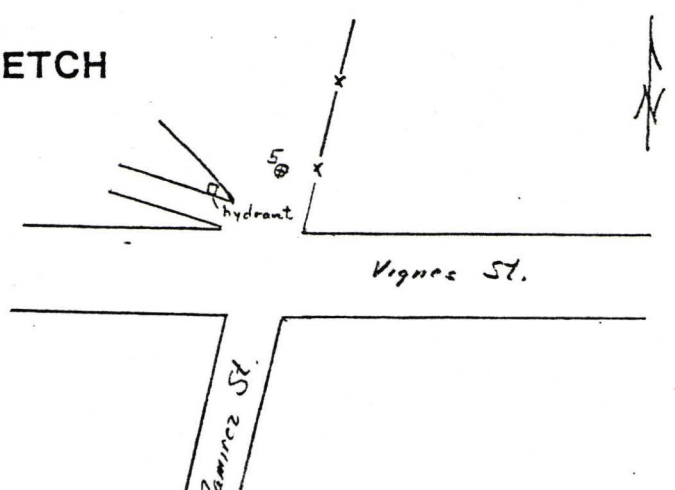
CROSS-HOLE SURVEY YES.

PVC CASING (I.D.): 4" 0.0' TO 100.0'; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. 280'.




SKETCH




SHEET 8 OF 8

**ConverseWardDavisDixon**
**Earth Sciences Associates**
**Geo/Resource Consultants**

# Boring Log 6

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

PROJECT 80-1230-2.2 DATE DRILLED 2/3/81-2/10/81 HOLE NO. 6
LOCATION Macy St. east of Alameda ST. GROUND ELEV. 281'
DRILLING CONTRACTOR Pitcher LOGGED BY Gallinotti 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 TOTAL DEPTH 150.2' NO. CORE BOXES 5

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 0 | | 0.0 - 0.5 CEMENT | | | AD | | | Begin drilling at 3:30 2/3/81. Auger to 10'; set 13' of 5" surface casing; set mud tub, mix mud; begin rotary drilling. |
| 1 | SP | 0.5 - 23.5 GRAVELLY SAND: light olive gray (5Y5/2); ~85-90% fine grained sand; 10-15% rounded to sub-rounded gravels, mostly 1/2" but up to 4"-5" cobbles; loose; damp to dry. | | | | | | |
| 2 | | | | | | | | |
| 4 | | | | | | | | [P.P.] stands for pocket penetrometer: data obtained from field-extruded PB samples |
| 6 | | | | | | | | |
| 8 | | increase in gravel content | | | | | | |
| 10 | | (~70% fine sand; ~10% medium to coarse sand. ~20% subrounded to sub-angular gravel). | J-1 | 29<br>24<br>17 | SS | | | 1.5/1.5 recovery |
| 12 | | | | | RD | | | 11.5' - stop drilling 2/3/81. Begin 2/4/81. |
| 14 | | | | | | | | |
| 16 | | 16.0 color change to Medium bluish gray (5B5/1) | | | | | | ★ Top of water table between 10' and 20'. |
| 18 | | | | | | | | |
| 20 | | | | | | | | SHEET 1 OF 8 |

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PROJECT 80-1280-22 DATE DRILLED 2/3/81 HOLE NO. 6

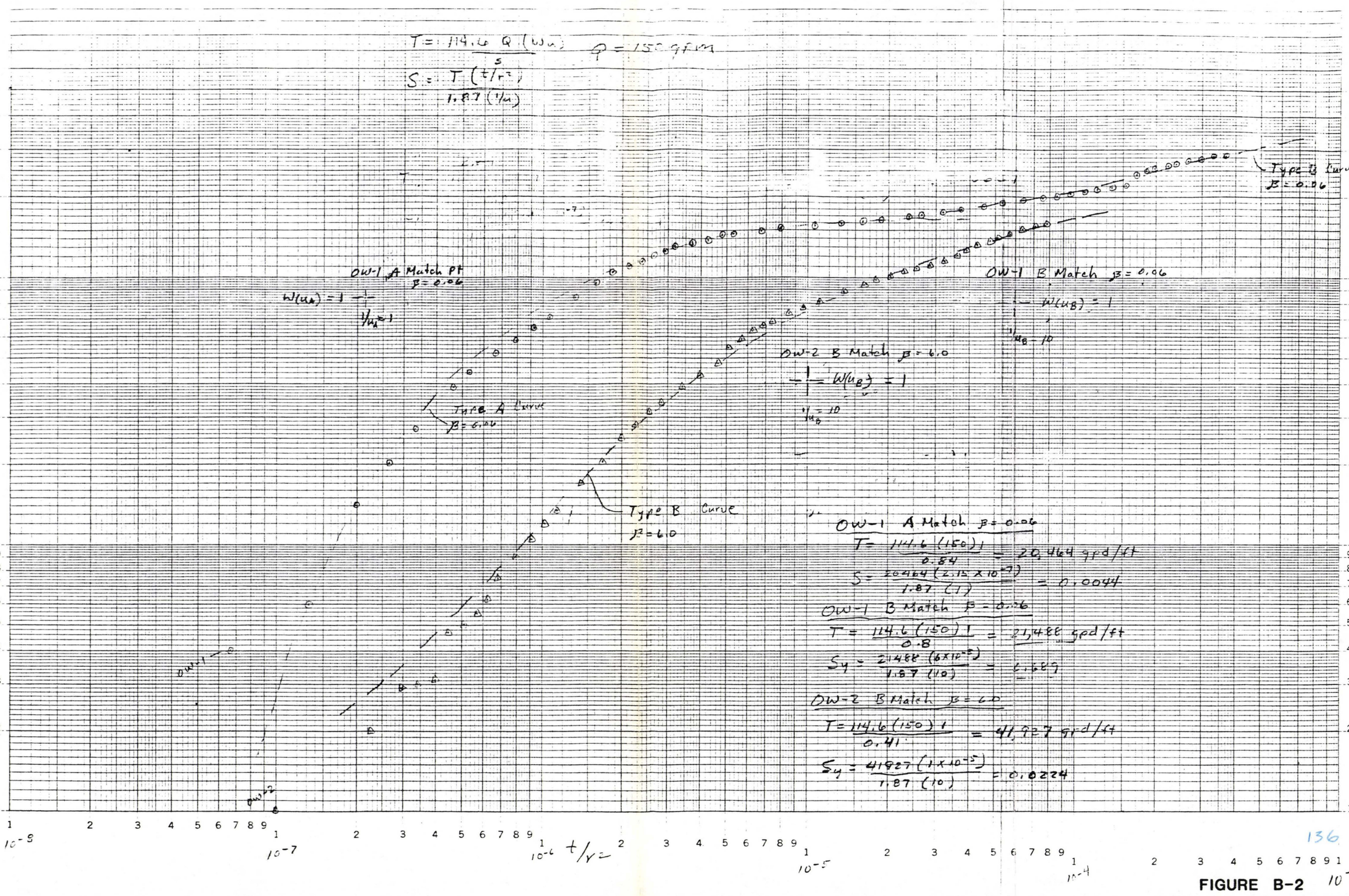
| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 20 | SP | 0.5 – 23.5 GRAVELLY SAND: (cont.) sample: loose sand w/ subangular to sub-rounded gravel; mostly ¼" to ½"; water saturated, | S-1 | | PB | | | take undisturbed PB rather than DR sample; because stuff barrel for DR sampler has been lost. 1.7/2.5 recovery |
| 22 | | | J-2 | 10<br>14<br>26 | SS | | | 1.5/1.5 recovery |
| 24 | SM | 23.5 – 38.0 SILTY SAND with GRAVELLY layers: medium bluish gray (5B5/1); ~20% low plasticity fines; ~80% very fine to fine sand; occasional gravelly layers contain ~20% sub-angular to sub-rounded ¼" to 1" gravels; clasts are predominately granitic or high-grade metamorphic quartzofeldspathic; medium dense; moist to wet; not saturated. | | | RD | | | water saturated |
| 26 | | | | | | | | |
| 28 | | | | | | | | |
| 30 | | | J-3 | 16<br>17<br>11 | SS | | | 1.0/1.5 recovery material is not saturated. |
| 32 | | | | | RD | | | |
| 34 | | | | | | | | 35' – begin rig chatter. |
| 36 | | granitic gravels (medium grained, Biotite – Hornblende Granodiorite) | | | | | | |
| 38 | SP | 38.0 – 79.0 SAND w/ GRAVELLY layers: Medium bluish gray (5B5/1); ~80% fine grained sand; ~20% medium to coarse sand; some gravelly layers w/ 10-20% subangular to subrounded granitic gravels; medium dense to loose; wet. | | | | | | |
| 40 | | | S-2 | | PB | | | 1.4/2.5 recovery tube bent on bottom; sample might be disturbed. |
| 42 | | | J-4 | 24<br>50 | SS | | | 0.8/0.8 recovery |
| 44 | | | | | RD | | | SHEET 2 OF 8 |

PROJECT 80-1240-22 DATE DRILLED 2/3/81 HOLE NO. 6

| DEPTH | CLASS. | FIELD DESCRIPTION | SAMPLE | SPT (6") | DRILL MODE | RUN NO. | CORE REC. % | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 44 | SP | 38.0 – 79.0 SAND with GRAVELLY layers: (cont.) | | | RD | | | |
| 46 | | | | | | | | |
| 48 | | | | | | | | |
| 50 | | | I-5 | 50 | 53 | | | 0.4/0.4 recovery |
| | | | | | RD | | | |
| 52 | | | | | | | | 50.4' – stop drilling 9:30 2/4/81. Drillers need to take drum off rig and are out of mud. |
| 54 | | | | | | | | 50.4' – Gas Test 2/9/81 – No combustibles; 20% $O_2$. |
| | | | | | | | | Begin drilling 10:20 2/9/81 – hole filled in to 30' with slush. |
| 56 | | | | | | | | |
| 58 | | | | | | | | |
| 60 | | | | | DR | | | No Recovery – loose gravelly sand – no binder to hold in sampler. DR ⇒ 37 blows 0.0/1.0 recovery |
| | | | | 11 | 55 | | | |
| 62 | | | I-6 | 45 | | | | 0.8/1.3 recovery |
| | | | | 50 | | | | |
| | | | | | AD | | | |
| 64 | | | | | | | | |
| 66 | | | | | | | | |
| 68 | | | | | | | | |

SHEET 3 OF 8

Union Station 2nd Test 3/25/83




OW-1 A Match β = 0.06

$T = \frac{114.6\,(150)\,1}{0.84} = 20{,}464$ gpd/ft

$S = \frac{20464\,(2.15 \times 10^{-7})}{1.87\,(1)} = 0.0044$

OW-1 B Match β = 0.06

$T = \frac{114.6\,(150)\,1}{0.8} = 21{,}488$ gpd/ft

$S_y = \frac{21488\,(6 \times 10^{-5})}{1.87\,(10)} = 0.689$

OW-2 B Match β = 6.0

$T = \frac{114.6\,(150)\,1}{0.41} = 41{,}927$ gpd/ft

$S_y = \frac{41927\,(1 \times 10^{-5})}{1.87\,(10)} = 0.0224$

FIGURE B-2

# Union Station Pump Tests
## Observation Well 5-1

△ 1st Test 3/24-25 Q = 175 gpm

○ 2nd Test 3/25-3/26/83 Q = 150 gpm

$\beta = 0.01$

2nd Test A Match Pt ⊙ $\beta = 0.01$

$W(u_A) = 1$

$1/u_A = 10$

2nd Test B Match Pt. $\beta = 0.01$

$W(u_B) = 1$

$1/u_B = 1$

Match Pt △ 1st Test $W(u_B) = 1$

$\beta = 0.01$

$1/u_B = 10$

$$T = \frac{114.6(Q)\,W(u)}{s}$$

$$\text{So } S_y = \frac{T\,(t/r^2)}{1.87\,(1/u)}$$

1st Test △ B Match Pt, $\beta = 0.01$

$$T = \frac{114.6(175)}{1.0}(1) = 20,055 \text{ gpd/ft}$$

$$S_y = \frac{20,055\,(2.2\times10^{-4})}{1.87\,(10)} = 0.23$$

2nd Test ⊙ A Match Pt $\beta = 0.01$

$$T = \frac{114.6(150)}{1.2}(1) = 14,325 \text{ gpd/ft}$$

$$S = \frac{14,325\,(4.6\times10^{-6})}{1.87\,(1)} = 0.035$$

2nd Test ⊙ B Match Pt $\beta = 0.01$

$$T = \frac{114.6(150)}{1.1}(1) = 15,627 \text{ gpd/ft}$$

$$S_y = \frac{15,627\,(3.25\times10^{-5})}{1.87\,(1)} = 0.27$$

t/r²

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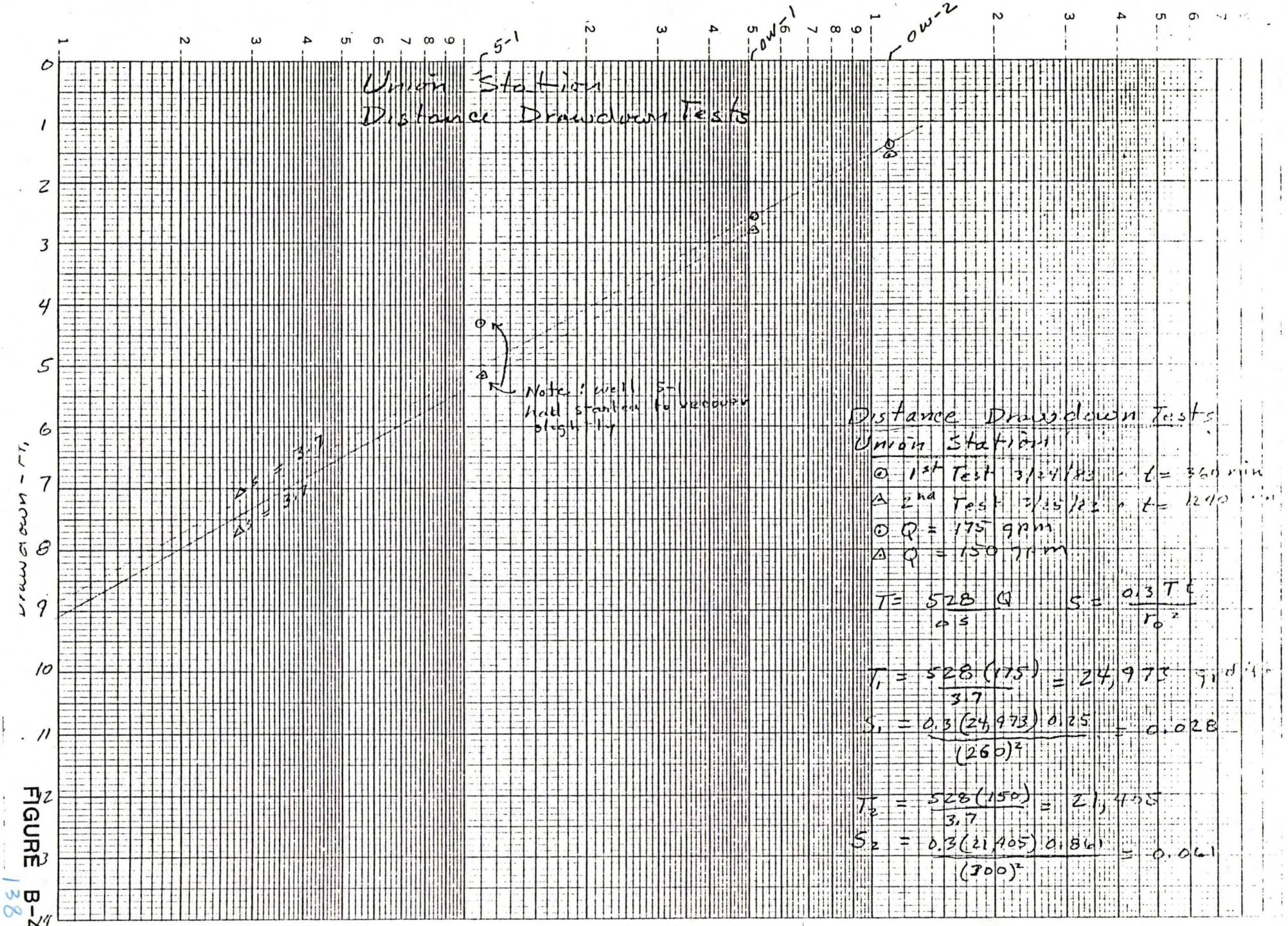
**FIGURE B-3**

47 7520
K&E LOGARITHMIC 3 X 5 CYCLES
KEUFFEL & ESSER CO.

K&E SEMI-LOGARITHMIC 3 CYCLES x 140 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 5810




Distance Drawdown Tests
Union Station

⊙ 1st Test 3/24/43 , t = 360 min
Δ 2nd Test 3/25/43 , t = 1210 "
⊙ Q = 175 gpm
Δ Q = 150 gpm

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

$T_1 = \frac{528(175)}{3.7} = 24,973$ gpd/ft

$S_1 = \frac{0.3(24,973)\,0.25}{(260)^2} = 0.028$

$T_2 = \frac{528(150)}{3.7} = 21,405$

$S_2 = \frac{0.3(21,405)\,0.86}{(300)^2} = 0.061$

FIGURE B-14

# AQUIFER TEST DATA SHEET

Observation Well No. OW-1 | Project No. E167

Test Well No. Union Station | Date of Test 03/24/83

Static Water Level 20.49 | Observed By WRH, TDH

Radius from Pumped Well 51.1 | Average Discharge 175 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:45 | 0 | --- | --- | 20.49 | 0 | |
| | .25 | $1.74 \times 10^{-4}$ | $6.66 \times 10^{-8}$ | 20.51 | 0.02 | |
| | .50 | $3.47 \times 10^{-4}$ | $1.33 \times 10^{-7}$ | 20.56 | 0.07 | |
| | .75 | $5.21 \times 10^{-4}$ | $2.00 \times 10^{-7}$ | 20.66 | 0.17 | |
| | 1 | $6.94 \times 10^{-4}$ | $2.66 \times 10^{-7}$ | 20.83 | 0.34 | |
| | 1.5 | $1.04 \times 10^{-3}$ | $3.98 \times 10^{-7}$ | 21.09 | 0.60 | |
| | 2 | $1.39 \times 10^{-3}$ | $5.32 \times 10^{-7}$ | 21.34 | 0.85 | |
| | 2.5 | $1.74 \times 10^{-3}$ | $6.66 \times 10^{-7}$ | 21.52 | 1.03 | |
| | 3 | $2.08 \times 10^{-3}$ | $7.97 \times 10^{-7}$ | 21.65 | 1.16 | |
| | 3.5 | $2.43 \times 10^{-3}$ | $9.31 \times 10^{-7}$ | 21.74 | 1.25 | |
| | 4 | $2.77 \times 10^{-3}$ | $1.06 \times 10^{-6}$ | 21.79 | 1.30 | |
| | 5 | $3.47 \times 10^{-3}$ | $1.33 \times 10^{-6}$ | 21.89 | 1.40 | |
| | 6 | $4.17 \times 10^{-3}$ | $1.60 \times 10^{-6}$ | 21.94 | 1.45 | |
| | 7 | $4.86 \times 10^{-3}$ | $1.86 \times 10^{-6}$ | 21.99 | 1.50 | |
| | 8 | $5.56 \times 10^{-3}$ | $2.13 \times 10^{-6}$ | 22.03 | 1.54 | |
| | 9 | $6.25 \times 10^{-3}$ | $2.39 \times 10^{-6}$ | 22.07 | 1.58 | |
| | 10 | $6.94 \times 10^{-3}$ | $2.66 \times 10^{-6}$ | 22.09 | 1.60 | |
| | 12 | $8.33 \times 10^{-3}$ | $3.19 \times 10^{-6}$ | 22.13 | 1.64 | |
| | 14 | $9.72 \times 10^{-3}$ | $3.72 \times 10^{-6}$ | 22.16 | 1.67 | |
| | 16 | $1.11 \times 10^{-2}$ | $4.25 \times 10^{-6}$ | 22.18 | 1.69 | |
| | 20 | $1.39 \times 10^{-2}$ | $5.32 \times 10^{-6}$ | 22.21 | 1.72 | |
| | 25 | $1.74 \times 10^{-2}$ | $6.66 \times 10^{-6}$ | 22.26 | 1.77 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 30 | $2.08 \times 10^{-2}$ | $7.97 \times 10^{-6}$ | 22.30 | 1.81 | |
| | 35 | $2.43 \times 10^{-2}$ | $9.31 \times 10^{-6}$ | 22.32 | 1.83 | |
| | 40 | $2.77 \times 10^{-2}$ | $1.06 \times 10^{-5}$ | 22.37 | 1.88 | |
| | 50 | $3.47 \times 10^{-2}$ | $1.33 \times 10^{-5}$ | 22.44 | 1.95 | |
| | 60 | $4.17 \times 10^{-2}$ | $1.60 \times 10^{-5}$ | 22.47 | 1.98 | |
| | 70 | $4.86 \times 10^{-2}$ | $1.86 \times 10^{-5}$ | 22.51 | 2.02 | |
| | 80 | $5.56 \times 10^{-2}$ | $2.13 \times 10^{-5}$ | 22.55 | 2.06 | |
| | 92 | $6.39 \times 10^{-2}$ | $2.45 \times 10^{-5}$ | 22.57 | 2.08 | |
| | 100 | $6.94 \times 10^{-2}$ | $2.66 \times 10^{-5}$ | 22.57 | 2.08 | |
| 1:45p | 120 | $8.33 \times 10^{-2}$ | $3.19 \times 10^{-5}$ | 22.67 | 2.18 | |
| | 140 | $9.72 \times 10^{-2}$ | $3.72 \times 10^{-5}$ | 22.71 | 2.22 | |
| 2:25p | 160 | $1.11 \times 10^{-1}$ | $4.25 \times 10^{-5}$ | 22.77 | 2.28 | |
| 2:45p | 180 | $1.25 \times 10^{-1}$ | $4.79 \times 10^{-5}$ | 22.82 | 2.33 | |
| | 200 | $1.39 \times 10^{-1}$ | $5.32 \times 10^{-5}$ | 22.86 | 2.37 | |
| 3:35 | 230 | $1.60 \times 10^{-1}$ | $6.13 \times 10^{-5}$ | 22.91 | 2.42 | |
| 5:01 | 316 | $2.19 \times 10^{-1}$ | $8.39 \times 10^{-5}$ | 23.01 | 2.52 | |
| 5:45 | 360 | $2.50 \times 10^{-1}$ | $9.57 \times 10^{-5}$ | 23.08 | 2.59 | |
| 6:45 | 420 | $2.92 \times 10^{-1}$ | $1.12 \times 10^{-4}$ | 23.01 | 2.52 | |
| 7:06 | 441 | $3.06 \times 10^{-1}$ | $1.17 \times 10^{-4}$ | 22.96 | 2.47 | |
| 8:02 | 497 | $3.45 \times 10^{-1}$ | $1.32 \times 10^{-4}$ | 22.91 | 2.42 | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

# AQUIFER TEST DATA SHEET

Observation Well No. OW-1 | Project No. E167

Test Well No. Union Station | Date of Test 03/25/83

Static Water Level 20.84 | Observed By WRH, TDH

Radius from Pumped Well 51.1 | Average Discharge 150 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:30a | 0 | -- | -- | 20.84 | 0 | |
| | .25 | $1.74 \times 10^{-4}$ | $6.66 \times 10^{-8}$ | 20.88 | 0.04 | |
| | .50 | $3.47 \times 10^{-4}$ | $1.33 \times 10^{-7}$ | 20.90 | 0.06 | |
| | .75 | $5.21 \times 10^{-4}$ | $2.00 \times 10^{-7}$ | 20.98 | 0.14 | |
| | 1 | $6.94 \times 10^{-4}$ | $2.66 \times 10^{-7}$ | 21.04 | 0.20 | |
| | 1.25 | $8.68 \times 10^{-4}$ | $3.32 \times 10^{-7}$ | 21.11 | 0.27 | |
| | 1.75 | $1.21 \times 10^{-3}$ | $4.63 \times 10^{-7}$ | 21.23 | 0.39 | |
| | 2 | $1.39 \times 10^{-3}$ | $5.32 \times 10^{-7}$ | 21.28 | 0.44 | |
| | 2.5 | $1.74 \times 10^{-3}$ | $6.66 \times 10^{-7}$ | 21.36 | 0.52 | |
| | 3 | $2.08 \times 10^{-3}$ | $7.97 \times 10^{-7}$ | 21.42 | 0.58 | |
| | 3.5 | $2.43 \times 10^{-3}$ | $9.31 \times 10^{-7}$ | 21.49 | 0.65 | |
| | 4 | $2.77 \times 10^{-3}$ | $1.06 \times 10^{-6}$ | 21.55 | 0.71 | |
| | 5 | $3.47 \times 10^{-3}$ | $1.33 \times 10^{-6}$ | 21.68 | 0.84 | |
| | 6 | $4.17 \times 10^{-3}$ | $1.60 \times 10^{-6}$ | 21.79 | 0.95 | |
| | 7 | $4.86 \times 10^{-3}$ | $1.86 \times 10^{-6}$ | 21.88 | 1.04 | |
| | 8 | $5.56 \times 10^{-3}$ | $2.13 \times 10^{-6}$ | 21.95 | 1.11 | |
| | 9 | $6.25 \times 10^{-3}$ | $2.39 \times 10^{-6}$ | 21.98 | 1.14 | |
| | 10 | $6.94 \times 10^{-3}$ | $2.66 \times 10^{-6}$ | 22.05 | 1.21 | |
| | 11 | $7.64 \times 10^{-3}$ | $2.93 \times 10^{-6}$ | 22.10 | 1.26 | |
| | 12 | $8.33 \times 10^{-3}$ | $3.19 \times 10^{-6}$ | 22.14 | 1.30 | |
| | 14 | $9.72 \times 10^{-3}$ | $3.72 \times 10^{-6}$ | 22.18 | 1.34 | |
| | 16 | $1.11 \times 10^{-3}$ | $4.25 \times 10^{-6}$ | 22.22 | 1.38 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 18 | $1.25\times10^{-2}$ | $4.79\times10^{-6}$ | 22.28 | 1.44 | |
| | 20 | $1.39\times10^{-2}$ | $5.32\times10^{-6}$ | 22.30 | 1.46 | |
| | 25 | $1.74\times10^{-2}$ | $6.66\times10^{-6}$ | 22.33 | 1.49 | |
| | 30 | $2.08\times10^{-2}$ | $7.97\times10^{-6}$ | 22.36 | 1.52 | |
| | 40 | $2.77\times10^{-2}$ | $1.06\times10^{-5}$ | 22.40 | 1.56 | |
| | 50 | $3.47\times10^{-2}$ | $1.33\times10^{-5}$ | 22.43 | 1.59 | |
| | 60 | $4.17\times10^{-2}$ | $1.60\times10^{-5}$ | 22.46 | 1.62 | |
| | 70 | $4.86\times10^{-2}$ | $1.86\times10^{-5}$ | 22.48 | 1.64 | |
| | 90 | $6.25\times10^{-2}$ | $2.39\times10^{-5}$ | 22.53 | 1.69 | |
| | 100 | $6.94\times10^{-2}$ | $2.66\times10^{-5}$ | 22.55 | 1.71 | |
| 1:30p | 120 | $8.33\times10^{-2}$ | $3.19\times10^{-5}$ | 22.60 | 1.76 | |
| | 140 | $9.72\times10^{-2}$ | $3.72\times10^{-5}$ | 22.63 | 1.79 | |
| | 170 | $1.18\times10^{-1}$ | $4.52\times10^{-5}$ | 22.67 | 1.83 | |
| | 200 | $1.39\times10^{-1}$ | $5.32\times10^{-5}$ | 22.73 | 1.89 | |
| 3:30 | 240 | $1.67\times10^{-1}$ | $6.40\times10^{-5}$ | 22.78 | 1.94 | |
| 4:15 | 285 | $1.98\times10^{-1}$ | $7.58\times10^{-5}$ | 22.82 | 1.98 | |
| 4:49 | 319 | $2.22\times10^{-1}$ | $8.50\times10^{-5}$ | 22.85 | 2.01 | |
| 5:30 | 360 | $2.50\times10^{-1}$ | $9.57\times10^{-5}$ | 22.89 | 2.05 | |
| 6:15 | 405 | $2.81\times10^{-1}$ | $1.08\times10^{-4}$ | 22.93 | 2.09 | |
| 7:00 | 450 | $3.13\times10^{-1}$ | $1.20\times10^{-4}$ | 22.97 | 2.13 | |
| 8:00 | 510 | $3.54\times10^{-1}$ | $1.36\times10^{-4}$ | 23.01 | 2.17 | |
| 9:04 | 574 | $3.99\times10^{-1}$ | $1.53\times10^{-4}$ | 23.04 | 2.20 | |
| 10:02 | 632 | $4.39\times10^{-1}$ | $1.68\times10^{-4}$ | 23.26 | 2.42 | |
| 10:58 | 688 | $4.78\times10^{-1}$ | $1.83\times10^{-4}$ | 23.34 | 2.50 | |
| 11:58 | 748 | $5.19\times10^{-1}$ | $1.99\times10^{-4}$ | 23.38 | 2.54 | |
| 1:30a | 840 | $5.83\times10^{-1}$ | $2.23\times10^{-4}$ | 23.46 | 2.62 | |
| | 900 | $6.25\times10^{-1}$ | $2.39\times10^{-4}$ | 23.51 | 2.67 | |
| 4:00a | 1000 | $6.94\times10^{-1}$ | $2.66\times10^{-4}$ | 23.56 | 2.72 | |

Well No. OW-1



| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 6:00a | 1120 | $7.77 \times 10^{-1}$ | $2.98 \times 10^{-4}$ | 23.61 | 2.77 | |
| 8:00a | 1240 | $8.61 \times 10^{-1}$ | $3.30 \times 10^{-4}$ | 23.66 | 2.82 | |
| 10:00a | 1380 | $9.58 \times 10^{-1}$ | $3.67 \times 10^{-4}$ | 23.65 | 2.81 | |

# AQUIFER TEST DATA SHEET

Observation Well No. OW-2 | Project No. E167

Test Well No. Union Station | Date of Test 03/24/83

Static Water Level 19.51 | Observed By WRH, TDH

Radius from Pumped Well 110.6 | Average Discharge 175 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:45 | 0 | -- | -- | 19.510 | 0 | |
| | .25 | $1.74 \times 10^{-4}$ | $1.42 \times 10^{-8}$ | 19.510 | 0 | |
| | .50 | $3.47 \times 10^{-4}$ | $2.84 \times 10^{-8}$ | 19.510 | 0 | |
| | .75 | $5.21 \times 10^{-4}$ | $4.26 \times 10^{-8}$ | 19.510 | 0 | |
| | 1 | $6.94 \times 10^{-4}$ | $5.67 \times 10^{-8}$ | 19.510 | 0 | |
| | 1.5 | $1.04 \times 10^{-3}$ | $8.50 \times 10^{-8}$ | 19.515 | 0.005 | |
| | 2 | $1.39 \times 10^{-3}$ | $1.14 \times 10^{-7}$ | 19.515 | 0.005 | |
| | 2.5 | $1.74 \times 10^{-3}$ | $1.42 \times 10^{-7}$ | 19.515 | 0.005 | |
| | 3 | $2.08 \times 10^{-3}$ | $1.70 \times 10^{-7}$ | 19.520 | 0.010 | |
| | 3.5 | $2.43 \times 10^{-3}$ | $1.99 \times 10^{-7}$ | 19.525 | 0.015 | |
| | 4 | $2.78 \times 10^{-3}$ | $2.27 \times 10^{-7}$ | 19.530 | 0.020 | |
| | 5 | $3.47 \times 10^{-3}$ | $2.83 \times 10^{-7}$ | 19.535 | 0.025 | |
| | 6 | $4.17 \times 10^{-3}$ | $3.41 \times 10^{-7}$ | 19.540 | 0.030 | |
| | 7 | $4.86 \times 10^{-3}$ | $3.97 \times 10^{-7}$ | 19.550 | 0.040 | |
| | 8 | $5.56 \times 10^{-3}$ | $4.55 \times 10^{-7}$ | 19.560 | 0.050 | |
| | 9 | $6.25 \times 10^{-3}$ | $5.11 \times 10^{-7}$ | 19.570 | 0.060 | |
| | 10.5 | $7.29 \times 10^{-3}$ | $5.96 \times 10^{-7}$ | 19.580 | 0.070 | |
| | 12 | $8.33 \times 10^{-3}$ | $6.81 \times 10^{-7}$ | 19.595 | 0.085 | |
| | 14 | $9.72 \times 10^{-3}$ | $7.95 \times 10^{-7}$ | 19.615 | 0.105 | |
| | 16 | $1.11 \times 10^{-2}$ | $9.07 \times 10^{-7}$ | 19.630 | 0.120 | |
| | 20 | $1.39 \times 10^{-2}$ | $1.14 \times 10^{-6}$ | 19.665 | 0.155 | |
| | 25 | $1.74 \times 10^{-2}$ | $1.42 \times 10^{-6}$ | 19.705 | 0.195 | |

Well No. OW-2



| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 30 | $2.08 \times 10^{-2}$ | $1.70 \times 10^{6}$ | 19.750 | 0.240 | |
| | 35 | $2.43 \times 10^{-2}$ | $1.99 \times 10^{6}$ | 19.805 | 0.295 | |
| | 40 | $2.78 \times 10^{-2}$ | $2.27 \times 10^{6}$ | 19.850 | 0.340 | |
| | 50 | $3.47 \times 10^{-2}$ | $2.84 \times 10^{6}$ | 19.920 | 0.410 | |
| | 60 | $4.17 \times 10^{-2}$ | $3.41 \times 10^{6}$ | 20.000 | 0.490 | |
| | 70 | $4.86 \times 10^{-2}$ | $3.97 \times 10^{6}$ | 20.070 | 0.560 | |
| | 80 | $5.56 \times 10^{-2}$ | $4.55 \times 10^{6}$ | 20.125 | 0.615 | |
| | 92 | $6.39 \times 10^{-2}$ | $5.22 \times 10^{6}$ | 20.190 | 0.680 | |
| | 100 | $6.94 \times 10^{-2}$ | $5.67 \times 10^{6}$ | 20.230 | 0.720 | |
| | 120 | $8.33 \times 10^{-2}$ | $6.81 \times 10^{6}$ | 20.340 | 0.830 | |
| | 140 | $9.72 \times 10^{-2}$ | $7.95 \times 10^{6}$ | 20.420 | 0.910 | |
| | 160 | $1.11 \times 10^{-1}$ | $9.07 \times 10^{6}$ | 20.490 | 0.980 | |
| 2:45 | 180 | $1.25 \times 10^{-1}$ | $1.02 \times 10^{5}$ | 20.560 | 1.050 | |
| 3:05 | 200 | $1.39 \times 10^{-1}$ | $1.14 \times 10^{5}$ | 20.620 | 1.110 | |
| 3:35 | 230 | $1.60 \times 10^{-1}$ | $1.31 \times 10^{5}$ | 20.690 | 1.180 | |
| 4:05 | 260 | $1.81 \times 10^{-1}$ | $1.48 \times 10^{5}$ | -- | -- | |
| 4:56 | 311 | $2.16 \times 10^{-1}$ | $1.77 \times 10^{5}$ | 20.850 | 1.340 | |
| 5:47 | 362 | $2.51 \times 10^{-1}$ | $2.05 \times 10^{5}$ | 20.920 | 1.410 | |
| 6:47 | 422 | $2.93 \times 10^{-1}$ | $2.40 \times 10^{5}$ | 20.980 | 1.470 | |
| 7:07 | 442 | $3.07 \times 10^{-1}$ | $2.51 \times 10^{5}$ | 20.985 | 1.475 | |
| 8:04 | 499 | $3.47 \times 10^{-1}$ | $2.84 \times 10^{5}$ | 20.990 | 1.480 | |
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# AQUIFER TEST DATA SHEET

Observation Well No. OW-2 | Project No. E167

Test Well No. Union Station | Date of Test 03/25/83

Static Water Level 19.50 | Observed By WRH, TDH

Radius from Pumped Well 110.6 | Average Discharge 150 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:30 | 0 | -- | -- | 19.500 | 0 | |
| | .25 | $1.74 \times 10^{-4}$ | $1.42 \times 10^{-8}$ | 19.500 | 0 | |
| | .50 | $3.47 \times 10^{-4}$ | $2.84 \times 10^{-8}$ | 19.500 | 0 | |
| | .75 | $5.21 \times 10^{-4}$ | $4.26 \times 10^{-8}$ | 19.504 | 0.004 | |
| | 1 | $6.94 \times 10^{-4}$ | $5.67 \times 10^{-8}$ | 19.504 | 0.004 | |
| | 1.25 | $8.68 \times 10^{-4}$ | $7.10 \times 10^{-8}$ | 19.504 | 0.004 | |
| | 1.50 | $1.04 \times 10^{-3}$ | $8.50 \times 10^{-8}$ | 19.504 | 0.004 | |
| | 1.75 | $1.22 \times 10^{-3}$ | $9.97 \times 10^{-8}$ | 19.510 | 0.010 | |
| | 3 | $2.08 \times 10^{-3}$ | $1.70 \times 10^{-7}$ | 19.515 | 0.015 | |
| | 3.50 | $2.43 \times 10^{-3}$ | $1.99 \times 10^{-7}$ | 19.515 | 0.015 | |
| | 4 | $2.78 \times 10^{-3}$ | $2.27 \times 10^{-7}$ | 19.520 | 0.020 | |
| | 5 | $3.47 \times 10^{-3}$ | $2.83 \times 10^{-7}$ | 19.520 | 0.020 | |
| | 5.25 | $3.65 \times 10^{-3}$ | $2.98 \times 10^{-7}$ | 19.529 | 0.029 | |
| | 6 | $4.17 \times 10^{-3}$ | $3.41 \times 10^{-7}$ | 19.530 | 0.030 | |
| | 7 | $4.86 \times 10^{-3}$ | $3.97 \times 10^{-7}$ | 19.532 | 0.032 | |
| | 8 | $5.56 \times 10^{-3}$ | $4.55 \times 10^{-7}$ | 19.547 | 0.047 | |
| | 9 | $6.25 \times 10^{-3}$ | $5.11 \times 10^{-7}$ | 19.550 | 0.050 | |
| | 10 | $6.94 \times 10^{-3}$ | $5.67 \times 10^{-7}$ | 19.555 | 0.055 | |
| | 11 | $7.64 \times 10^{-3}$ | $6.25 \times 10^{-7}$ | 19.562 | 0.062 | |
| | 12 | $8.33 \times 10^{-3}$ | $6.81 \times 10^{-7}$ | 19.575 | 0.075 | |
| | 14 | $9.72 \times 10^{-3}$ | $7.95 \times 10^{-7}$ | 19.590 | 0.090 | |
| | 16 | $1.11 \times 10^{-3}$ | $9.07 \times 10^{-7}$ | 19.605 | 0.105 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 18 | $1.25\times10^{-2}$ | $1.02\times10^{-6}$ | 19.620 | 0.120 | |
| | 20 | $1.39\times10^{-2}$ | $1.14\times10^{-6}$ | 19.635 | 0.135 | |
| | 25 | $1.74\times10^{-2}$ | $1.42\times10^{-6}$ | 19.670 | 0.170 | |
| | 30 | $2.08\times10^{-2}$ | $1.70\times10^{-6}$ | 19.705 | 0.205 | |
| 12:05 | 35 | $2.43\times10^{-2}$ | $1.99\times10^{-6}$ | 19.750 | 0.250 | |
| 12:10 | 40 | $2.77\times10^{-2}$ | $2.27\times10^{-6}$ | 19.780 | 0.280 | |
| | 45 | $3.13\times10^{-2}$ | $2.56\times10^{-6}$ | 19.815 | 0.315 | |
| 12:20 | 50 | $3.47\times10^{-2}$ | $2.84\times10^{-6}$ | 19.840 | 0.340 | |
| 12:30 | 60 | $4.17\times10^{-2}$ | $3.41\times10^{-6}$ | 19.890 | 0.390 | |
| 12:40 | 70 | $4.86\times10^{-2}$ | $3.97\times10^{-6}$ | 19.930 | 0.430 | |
| 12:52 | 82 | $5.69\times10^{-2}$ | $4.65\times10^{-6}$ | 19.980 | 0.480 | |
| 1:00 | 90 | $6.25\times10^{-2}$ | $5.11\times10^{-6}$ | 20.050 | 0.550 | |
| 1:10 | 100 | $6.94\times10^{-2}$ | $5.67\times10^{-6}$ | 20.090 | 0.590 | |
| 1:20 | 110 | $7.64\times10^{-2}$ | $6.25\times10^{-6}$ | 20.130 | 0.630 | |
| 1:30 | 120 | $8.33\times10^{-2}$ | $6.81\times10^{-6}$ | 20.160 | 0.660 | |
| 1:40 | 130 | $9.03\times10^{-2}$ | $7.38\times10^{-6}$ | 20.180 | 0.680 | |
| 2:00 | 150 | $1.04\times10^{-1}$ | $8.50\times10^{-6}$ | 20.230 | 0.730 | |
| 2:20 | 170 | $1.18\times10^{-1}$ | $9.65\times10^{-6}$ | 20.270 | 0.770 | |
| 2:46 | 196 | $1.36\times10^{-1}$ | $1.11\times10^{-5}$ | 20.310 | 0.810 | |
| 3:30 | 240 | $1.67\times10^{-1}$ | $1.37\times10^{-5}$ | 20.390 | 0.890 | |
| 4:15 | 285 | $1.98\times10^{-1}$ | $1.62\times10^{-5}$ | 20.440 | 0.940 | |
| 4:47 | 317 | $2.20\times10^{-1}$ | $1.80\times10^{-5}$ | 20.480 | 0.980 | |
| 5:30 | 360 | $2.50\times10^{-1}$ | $2.04\times10^{-5}$ | 20.510 | 1.010 | |
| 6:14 | 404 | $2.81\times10^{-1}$ | $2.30\times10^{-5}$ | 20.550 | 1.050 | |
| 7:00 | 450 | $3.13\times10^{-1}$ | $2.56\times10^{-5}$ | 20.580 | 1.080 | |
| 7:56 | 506 | $3.51\times10^{-1}$ | $2.87\times10^{-5}$ | 20.620 | 1.120 | |
| 9:00 | 570 | $3.96\times10^{-1}$ | $3.24\times10^{-5}$ | 20.660 | 1.160 | |
| 10:00 | 630 | $4.38\times10^{-1}$ | $3.58\times10^{-5}$ | 20.700 | 1.200 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:00 | 690 | $4.79 \times 10^{-1}$ | $3.92 \times 10^{-5}$ | 20.770 | 1.270 | |
| 12:00 | 750 | $5.21 \times 10^{-1}$ | $4.26 \times 10^{-5}$ | 20.820 | 1.320 | |
| | 840 | $5.83 \times 10^{-1}$ | $4.77 \times 10^{-5}$ | 20.880 | 1.380 | |
| | 900 | $6.25 \times 10^{-1}$ | $5.11 \times 10^{-5}$ | 20.910 | 1.410 | |
| | 1000 | $6.94 \times 10^{-1}$ | $5.67 \times 10^{-5}$ | 20.960 | 1.460 | |
| | 1120 | $7.77 \times 10^{-1}$ | $6.35 \times 10^{-5}$ | 21.010 | 1.510 | |
| | 1240 | $8.61 \times 10^{-1}$ | $7.04 \times 10^{-5}$ | 21.060 | 1.560 | |
| | 1380 | $9.58 \times 10^{-1}$ | $7.83 \times 10^{-5}$ | 21.100 | 1.600 | |

# AQUIFER TEST DATA SHEET

Observation Well No. 5-1 | Project No. E167

Test Well No. Union Station | Date of Test 03/24/83

Static Water Level 19.92 | Observed By DG

Radius from Pumped Well 11.1 ft. | Average Discharge 175 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:45 | 0 | 0 | --- | 19.92 | 0.00 | large initial drawdown-missed |
| | 1 | $6.94 \times 10^{-4}$ | $5.33 \times 10^{-6}$ | -- | -- | 1 min. reading |
| | 2 | $1.39 \times 10^{-3}$ | $1.07 \times 10^{-5}$ | 23.58 | 3.66 | |
| | 3 | $2.08 \times 10^{-3}$ | $1.60 \times 10^{-5}$ | 23.60 | 3.68 | |
| | 4 | $2.78 \times 10^{-3}$ | $2.14 \times 10^{-5}$ | 23.31 | 3.39 | |
| | 5 | $3.47 \times 10^{-3}$ | $2.67 \times 10^{-5}$ | 23.32 | 3.40 | |
| | 6 | $4.17 \times 10^{-3}$ | $3.20 \times 10^{-5}$ | 23.40 | 3.48 | |
| | 7 | $4.86 \times 10^{-3}$ | $3.73 \times 10^{-5}$ | -- | -- | |
| | 8 | $5.56 \times 10^{-3}$ | $4.27 \times 10^{-5}$ | 23.42 | 3.50 | |
| | 9 | $6.25 \times 10^{-3}$ | $4.80 \times 10^{-5}$ | 23.42 | 3.50 | |
| | 10 | $6.94 \times 10^{-3}$ | $5.33 \times 10^{-5}$ | 23.43 | 3.51 | |
| | 11 | $7.64 \times 10^{-3}$ | $5.87 \times 10^{-5}$ | 23.43 | 3.51 | |
| | 12 | $8.33 \times 10^{-3}$ | $6.40 \times 10^{-5}$ | 23.46 | 3.54 | |
| | 13 | $9.03 \times 10^{-3}$ | $6.94 \times 10^{-5}$ | 23.47 | 3.55 | |
| | 14 | $9.72 \times 10^{-3}$ | $7.47 \times 10^{-5}$ | 23.47 | 3.55 | |
| | 15 | $1.04 \times 10^{-2}$ | $7.99 \times 10^{-5}$ | 23.50 | 3.58 | |
| | 16 | $1.11 \times 10^{-2}$ | $8.53 \times 10^{-5}$ | 23.48 | 3.56 | |
| | 17 | $1.18 \times 10^{-2}$ | $9.06 \times 10^{-5}$ | 23.47 | 3.55 | |
| | 18 | $1.25 \times 10^{-2}$ | $9.60 \times 10^{-5}$ | -- | -- | |
| | 19 | $1.32 \times 10^{-2}$ | $1.01 \times 10^{-4}$ | -- | -- | |
| | 20 | $1.39 \times 10^{-2}$ | $1.07 \times 10^{-4}$ | 23.47 | 3.55 | |
| | 22 | $1.53 \times 10^{-2}$ | $1.18 \times 10^{-4}$ | -- | -- | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 24 | $1.67 \times 10^{-2}$ | $1.28 \times 10^{-4}$ | 23.53 | 3.61 | |
| | 25 | $1.74 \times 10^{-2}$ | $1.34 \times 10^{-4}$ | 23.55 | 3.63 | |
| | 28 | $1.94 \times 10^{-2}$ | $1.49 \times 10^{-4}$ | 23.57 | 3.65 | |
| | 31 | $2.15 \times 10^{-2}$ | $1.65 \times 10^{-4}$ | 23.55 | 3.63 | |
| | 35 | $2.43 \times 10^{-2}$ | $1.87 \times 10^{-4}$ | 23.50 | 3.58 | |
| | 37 | $2.57 \times 10^{-2}$ | $1.97 \times 10^{-4}$ | 23.71 | 3.79 | |
| | 40 | $2.78 \times 10^{-2}$ | $2.14 \times 10^{-4}$ | 23.62 | 3.70 | |
| | 43 | $2.99 \times 10^{-2}$ | $2.30 \times 10^{-4}$ | 23.67 | 3.75 | |
| | 45 | $3.13 \times 10^{-2}$ | $2.40 \times 10^{-4}$ | 23.64 | 3.72 | |
| | 50 | $3.47 \times 10^{-2}$ | $2.67 \times 10^{-4}$ | 23.65 | 3.73 | |
| | 56 | $3.89 \times 10^{-2}$ | $2.99 \times 10^{-4}$ | 23.66 | 3.74 | |
| | 60 | $4.17 \times 10^{-2}$ | $3.20 \times 10^{-4}$ | 23.66 | 3.74 | |
| | 67 | $4.65 \times 10^{-2}$ | $3.57 \times 10^{-4}$ | 23.70 | 3.78 | |
| | 70 | $4.86 \times 10^{-2}$ | $3.73 \times 10^{-4}$ | 23.69 | 3.77 | |
| | 75 | $5.21 \times 10^{-2}$ | $4.00 \times 10^{-4}$ | 23.71 | 3.79 | |
| | 80 | $5.56 \times 10^{-2}$ | $4.27 \times 10^{-4}$ | 23.71 | 3.79 | |
| | 85 | $5.90 \times 10^{-2}$ | $4.53 \times 10^{-4}$ | 23.83 | 3.91 | |
| | 90 | $6.25 \times 10^{-2}$ | $4.80 \times 10^{-4}$ | 23.75 | 3.83 | |
| | 95 | $6.60 \times 10^{-2}$ | $5.07 \times 10^{-4}$ | 23.80 | 3.88 | |
| | 100 | $6.94 \times 10^{-2}$ | $5.33 \times 10^{-4}$ | -- | -- | |
| 1:30 | 105 | $7.29 \times 10^{-2}$ | $5.60 \times 10^{-4}$ | 23.77 | 3.85 | |
| | 110 | $7.64 \times 10^{-2}$ | $5.87 \times 10^{-4}$ | 23.78 | 3.86 | |
| 1:45 | 120 | $8.33 \times 10^{-2}$ | $6.40 \times 10^{-4}$ | 23.90 | 3.98 | |
| 1:50 | 125 | $8.68 \times 10^{-2}$ | $6.67 \times 10^{-4}$ | 23.92 | 4.00 | |
| 2:00 | 135 | $9.38 \times 10^{-2}$ | $7.20 \times 10^{-4}$ | 23.91 | 3.99 | |
| 2:10 | 145 | $1.01 \times 10^{-1}$ | $7.76 \times 10^{-4}$ | 23.98 | 4.06 | |
| 2:26 | 155 | $1.08 \times 10^{-1}$ | $8.29 \times 10^{-4}$ | 24.03 | 4.11 | |
| 2:30 | 165 | $1.15 \times 10^{-1}$ | $8.83 \times 10^{-4}$ | 24.03 | 4.11 | |
| 2:45 | 180 | $1.25 \times 10^{-1}$ | $9.60 \times 10^{-4}$ | 24.01 | 4.09 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 3:00 | 195 | $1.35 \times 10^{-1}$ | $1.04 \times 10^{-3}$ | 24.07 | 4.15 | |
| 3:15 | 210 | $1.46 \times 10^{-1}$ | $1.12 \times 10^{-3}$ | 24.16 | 4.24 | |
| 3:45 | 240 | $1.67 \times 10^{-1}$ | $1.28 \times 10^{-3}$ | 24.20 | 4.28 | |
| 4:15 | 270 | $1.88 \times 10^{-1}$ | $1.44 \times 10^{-3}$ | 24.23 | 4.31 | |
| 4:45 | 300 | $2.08 \times 10^{-1}$ | $1.60 \times 10^{-3}$ | 24.19 | 4.27 | |
| 5:50 | 365 | $2.53 \times 10^{-1}$ | $1.94 \times 10^{-3}$ | 24.22 | 4.30 | |
| 7:04 | 439 | $3.05 \times 10^{-1}$ | $2.34 \times 10^{-3}$ | 23.68 | 3.76 | |

# AQUIFER TEST DATA SHEET

Observation Well No. 5-1 | Project No. E167

Test Well No. Union Station | Date of Test 03/25/83

Static Water Level 19.01 | Observed By DG

Radius from Pumped Well 11.1 ft. | Average Discharge 150 gpm

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 11:30 | 0 | 0 | -- | 19.01 | 0.0 | started pump 11:15, stopped |
| | 1 | $6.94 \times 10^{-4}$ | $5.63 \times 10^{-6}$ | -- | -- | 11:20-start 11:30 static before 1st |
| | 2 | $1.39 \times 10^{-3}$ | $1.13 \times 10^{-5}$ | 21.72 | 2.71 | start-19.01 |
| | 3 | $2.08 \times 10^{-3}$ | $1.69 \times 10^{-5}$ | 21.85 | 2.84 | |
| | 4 | $2.78 \times 10^{-3}$ | $2.26 \times 10^{-5}$ | 22.30 | 3.29 | |
| | 5 | $3.47 \times 10^{-3}$ | $2.82 \times 10^{-5}$ | 22.35 | 3.34 | |
| | 6 | $4.17 \times 10^{-3}$ | $3.38 \times 10^{-5}$ | 22.55 | 3.54 | |
| | 7 | $4.86 \times 10^{-3}$ | $3.94 \times 10^{-5}$ | 22.69 | 3.68 | |
| | 8 | $5.56 \times 10^{-3}$ | $4.51 \times 10^{-5}$ | 22.76 | 3.75 | |
| | 9 | $6.25 \times 10^{-3}$ | $5.07 \times 10^{-5}$ | 22.77 | 3.76 | |
| | 10 | $6.94 \times 10^{-3}$ | $5.63 \times 10^{-5}$ | 22.82 | 3.81 | |
| | 11 | $7.64 \times 10^{-3}$ | $6.20 \times 10^{-5}$ | 22.86 | 3.85 | |
| | 12 | $8.33 \times 10^{-3}$ | $6.76 \times 10^{-5}$ | 22.90 | 3.89 | |
| | 13 | $9.03 \times 10^{-3}$ | $7.55 \times 10^{-5}$ | 22.86 | 3.85 | |
| | 14 | $9.72 \times 10^{-3}$ | $7.89 \times 10^{-5}$ | 22.90 | 3.89 | |
| | 15 | $1.04 \times 10^{-2}$ | $8.44 \times 10^{-5}$ | 22.90 | 3.89 | |
| | 17 | $1.18 \times 10^{-2}$ | $9.58 \times 10^{-5}$ | 22.94 | 3.93 | |
| | 19 | $1.32 \times 10^{-2}$ | $1.07 \times 10^{-4}$ | 22.95 | 3.94 | |
| | 21 | $1.46 \times 10^{-2}$ | $1.19 \times 10^{-4}$ | 22.97 | 3.96 | |
| | 23 | $1.60 \times 10^{-2}$ | $1.30 \times 10^{-4}$ | 23.01 | 4.00 | |
| | 25 | $1.74 \times 10^{-2}$ | $1.41 \times 10^{-4}$ | 23.00 | 3.99 | |
| | 27 | $1.88 \times 10^{-2}$ | $1.53 \times 10^{-4}$ | 23.02 | 4.01 | |

Well No. 5-1



| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| 12:00 | 30 | $2.08 \times 10^{-2}$ | $1.69 \times 10^{-4}$ | 23.01 | 4.00 | |
| | 32 | $2.22 \times 10^{-2}$ | $1.80 \times 10^{-4}$ | 23.02 | 4.01 | |
| | 34 | $2.36 \times 10^{-2}$ | $1.92 \times 10^{-4}$ | 22.99 | 3.98 | |
| | 36 | $2.50 \times 10^{-2}$ | $2.03 \times 10^{-4}$ | 23.00 | 3.99 | |
| | 40 | $2.78 \times 10^{-2}$ | $2.26 \times 10^{-4}$ | 23.00 | 3.99 | |
| 12:20 | 50 | $3.47 \times 10^{-2}$ | $2.82 \times 10^{-4}$ | 23.03 | 4.02 | |
| 12:30 | 60 | $4.17 \times 10^{-2}$ | $3.38 \times 10^{-4}$ | 23.05 | 4.04 | |
| 12:40 | 70 | $4.86 \times 10^{-2}$ | $3.94 \times 10^{-4}$ | 23.06 | 4.05 | |
| 12:50 | 80 | $5.56 \times 10^{-2}$ | $4.51 \times 10^{-4}$ | 23.09 | 4.08 | |
| 1:00 | 90 | $6.25 \times 10^{-2}$ | $5.07 \times 10^{-4}$ | 23.08 | 4.07 | |
| 1:15 | 105 | $7.29 \times 10^{-2}$ | $5.92 \times 10^{-4}$ | 23.10 | 4.09 | |
| 1:30 | 120 | $8.33 \times 20^{-2}$ | $6.76 \times 10^{-4}$ | 23.13 | 4.12 | |
| 1:45 | 135 | $9.38 \times 10^{-2}$ | $7.61 \times 10^{-4}$ | 23.14 | 4.13 | |
| 2:00 | 150 | $1.04 \times 10^{-1}$ | $8.44 \times 10^{-4}$ | 23.17 | 4.16 | |
| 2:50 | 200 | $1.39 \times 10^{-1}$ | $1.13 \times 10^{-3}$ | 23.20 | 4.19 | |
| 3:15 | 225 | $1.56 \times 10^{-1}$ | $1.27 \times 10^{-3}$ | 23.26 | 4.25 | |
| 3:30 | 240 | $1.67 \times 10^{-1}$ | $1.36 \times 10^{-3}$ | 23.30 | 4.29 | |
| 4:00 | 270 | $1.88 \times 10^{-1}$ | $1.53 \times 10^{-3}$ | 23.31 | 4.30 | |
| 4:50 | 320 | $2.22 \times 10^{-1}$ | $1.80 \times 10^{-3}$ | 23.31 | 4.30 | |
| 5:34 | 364 | $2.53 \times 10^{-1}$ | $2.05 \times 10^{-3}$ | 23.33 | 4.32 | |
| 6:20 | 410 | $2.85 \times 10^{-1}$ | $2.31 \times 10^{-3}$ | 23.36 | 4.35 | |
| 7:03 | 453 | $3.15 \times 10^{-1}$ | $2.56 \times 10^{-3}$ | 23.35 | 4.34 | |
| 8:04 | 514 | $3.57 \times 10^{-1}$ | $2.90 \times 10^{-3}$ | 23.37 | 4.36 | |
| 9:08 | 578 | $4.01 \times 10^{-1}$ | $3.25 \times 10^{-3}$ | 23.38 | 4.37 | |
| 10:06 | 636 | $4.42 \times 10^{-1}$ | $3.59 \times 10^{-3}$ | 23.82 | 4.81 | |
| 11:04 | 694 | $4.82 \times 10^{-1}$ | $3.91 \times 10^{-3}$ | 23.91 | 4.90 | |
| 6/83 12:03 | 753 | $5.23 \times 10^{-1}$ | $4.25 \times 10^{-3}$ | 23.92 | 4.91 | |
| | 843 | $5.85 \times 10^{-1}$ | $4.75 \times 10^{-3}$ | 23.95 | 4.94 | |

| Time | t min. | t days | $t/r^2$ | Water Level feet | Drawdown, s feet | Remarks |
|---|---|---|---|---|---|---|
| | 905 | $6.28 \times 10^{-1}$ | $5.10 \times 10^{-3}$ | 24.03 | 5.02 | |
| | 1000 | $6.94 \times 10^{-1}$ | $5.63 \times 10^{-3}$ | 24.09 | 5.08 | |
| | 1120 | $7.78 \times 10^{-1}$ | $6.31 \times 10^{-3}$ | 24.05 | 5.04 | |
| | 1240 | $8.61 \times 10^{-1}$ | $6.99 \times 10^{-3}$ | 24.08 | 5.07 | |
| | 1380 | $9.58 \times 10^{-1}$ | $7.78 \times 10^{-3}$ | 23.98 | 4.97 | |

# Appendix C

# Geophysical Exploration

# APPENDIX C GEOPHYSICAL EXPLORATION

## C.1 DOWNHOLE SURVEY

### C.1.1 Summary

A downhole shear wave velocity survey was performed in Boring CEG-5 Design Unit A135. Measurements were made at 5-foot intervals from the ground surface to depths of 130 feet. A description of the technique and a summary of the results are attached.

### C.1.2 Field Procedure

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

### C.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure C-1. The timing line shows a 20 millisecond (MS) break at the end of the record, indicating that each vertical line is 10 MS. The time of the first arrivals of compressional shear energy is indicated by P and S, respectively. Wave arrival records similar to Figure C-1 were analyzed to estimate wave travel times and velocities for CEG-5.

### C.1.4 Discussion of Results

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

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

In general, near-surface shear wave velocity was found to be in the range of 1000 feet per second. To depths of 130 feet, shear velocity estimates increased to 1400±100 feet per second.

## C.2 CROSSHOLE SURVEY

### C.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed in Boring CEG-5. The crosshole technique for determining shear wave velocities of in-situ materials was utilized in a three-borehole array. The array consisted of boring CEG-5 and two additional holes drilled approximately 15 feet away. All boreholes were drilled to a depth of 100 feet. Compressional wave and shear wave velocities are presented in Table C-2.

### C.2.2 Field Procedure

The shear wave hammer is placed in an end hole of the array, and vertical geophones are placed in the remaining two boreholes. The shear wave generating hammer and the two geophones are lowered to the same depth in all boreholes. The hammer is coupled to the wall of the hole by means of hydraulic jacks, and the geophones are coupled by means of expanding heavy rubber balloons which protrude from one side of the geophone housings. The hammer is then used to create vertically polarized shear waves with either an up or down first motion. A 12-channel signal enhancement seismograph with oscilloscope and electrostatic paper camera is used as a signal storage device.

### C.2.3 Data Analysis

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

### C.2.4 Discussion of Data Analysis

Seismic wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet (see Figures C-3 and C-4). The shear wave velocity ($V_s$) is equal to the difference in travel path distance from the shear source to each geophone divided by the difference in shear wave arrival times. The results of the compressional and shear wave velocity analyses are shown in Table C-2. It should be noted that compression wave velocities below the ground water table may be masked by the compression wave response of the water ($V_c$ = 5000 fps).

TABLE C-1 Downhole Velocities

| Boring No. | Depth (ft) | COMPRESSIONAL WAVE | | | | | SHEAR WAVE | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $\overline{V}p$ | $\sigma$ p | Ep | Np | Vp* | $\overline{V}s$ | $\sigma$s | Es | Ns | Vs* |
| 5 | 20-75 | 4831 | 736 | 242 | 10 | 4830±98 | 994 | 79 | 50 | 12 | 990±130 |
| | 75-130 | 5458 | 500 | 273 | 12 | 5460±770 | 1391 | 39 | 70 | 12 | 1390±110 |

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

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

$\sigma p$ = standard deviation of estimated compressional wave velocity

$\sigma s$ = 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 C-2 Crosshole Velocities

| Boring No. | Depth (ft) | COMPRESSIONAL WAVE | | | | | SHEAR WAVE | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $\bar{V}p$ | $\sigma p$ | Ep | Np | Vp* | $\bar{V}s$ | $\sigma s$ | Es | Ns | Vs* |
| 5 | 10 | 2555 | --- | 510 | 1 | 2560±510 | 948 | 3 | 47 | 6 | 950±50 |
| | 15 | 4948 | 319 | 247 | 4 | 4950±570 | 1081 | 17 | 54 | 10 | 1080±70 |
| | 20 | 5847 | 524 | 292 | 4 | 5850±820 | 1107 | 4 | 55 | 4 | 1110±60 |
| | 25 | 4886 | --- | 490 | 2 | 4890±490 | 1122 | 41 | 56 | 9 | 1120±100 |
| | 30 | 4512 | 1056 | 226 | 10 | 4510±1280 | 1103 | 4 | 55 | 8 | 1100±60 |
| | 40 | 5307 | 300 | 265 | 8 | 5310±570 | 1193 | 63 | 60 | 8 | 1190±120 |
| | 45 | 5981 | 305 | 300 | 4 | 5980±600 | 1158 | 7 | 58 | 12 | 1160±60 |
| | 50 | 6256 | 590 | 313 | 4 | 6260±900 | 1025 | 6 | 51 | 13 | 1025±60 |
| | 55 | 5810 | 369 | 290 | 4 | 5180±660 | 1168 | 28 | 58 | 12 | 1170±90 |
| | 60 | 4651 | 607 | 233 | 4 | 4650±840 | 1167 | 4 | 58 | 13 | 1170±60 |
| | 65 | 5147 | 1193 | 257 | 9 | 5150±1450 | 1173 | 37 | 59 | 12 | 1170±100 |
| | 70 | 5041 | 422 | 252 | 6 | 5040±670 | 1080 | 12 | 54 | 6 | 1080±70 |
| | 75 | 5803 | 947 | 290 | 4 | 5800±1240 | 1224 | 42 | 61 | 12 | 1220±100 |
| | 80 | 7852 | --- | 1570 | 2 | 7850±1570 | 1400 | 22 | 70 | 16 | 1400±90 |
| | 85 | 5200 | 426 | 260 | 8 | 5200±690 | 1186 | 42 | 59 | 10 | 1190±100 |
| | 90 | 5740 | 409 | 287 | 4 | 5740±700 | 1364 | 10 | 68 | 12 | 1360±80 |
| | 95 | 5873 | 1180 | 294 | 4 | 5870±1470 | 1304 | 8 | 65 | 10 | 1300±70 |
| | 99 | 5592 | 291 | 280 | 8 | 5590±570 | 1417 | 14 | 71 | 16 | 1410±90 |

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

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

$\sigma p$ = standard deviation of estimated compressional wave velocity

$\sigma s$ = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

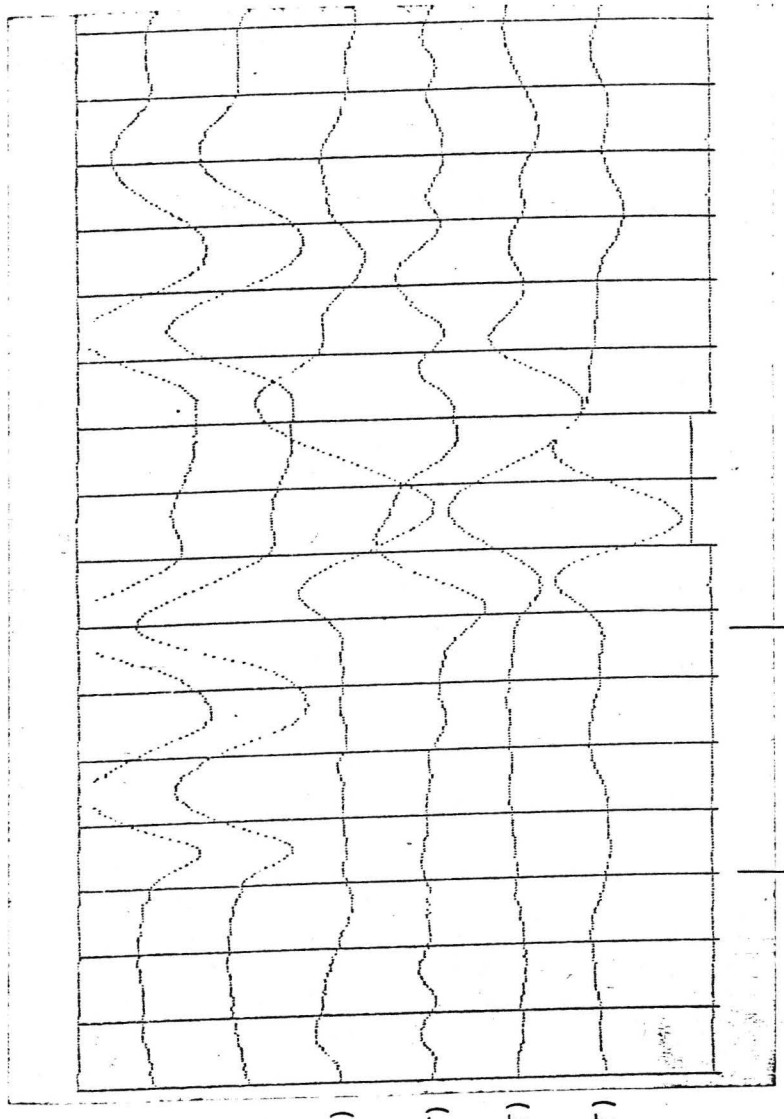
Es = estimated accuracy of shear survey

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

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

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




DOWNHOLE SAMPLE RECORD

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83-1101

Figure No.
C-1

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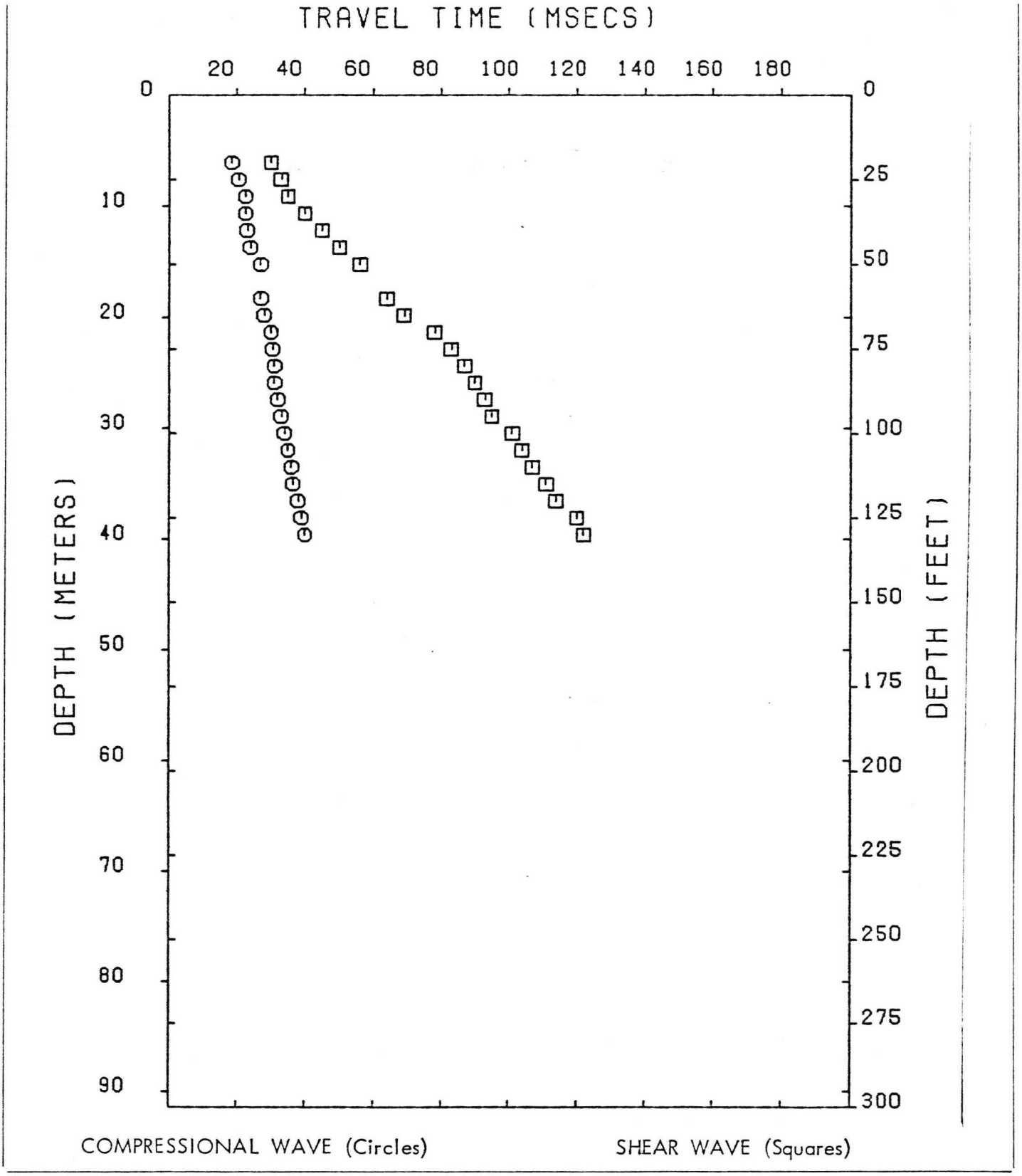


DOWNHOLE TRAVEL TIME PROFILE - BORING 5

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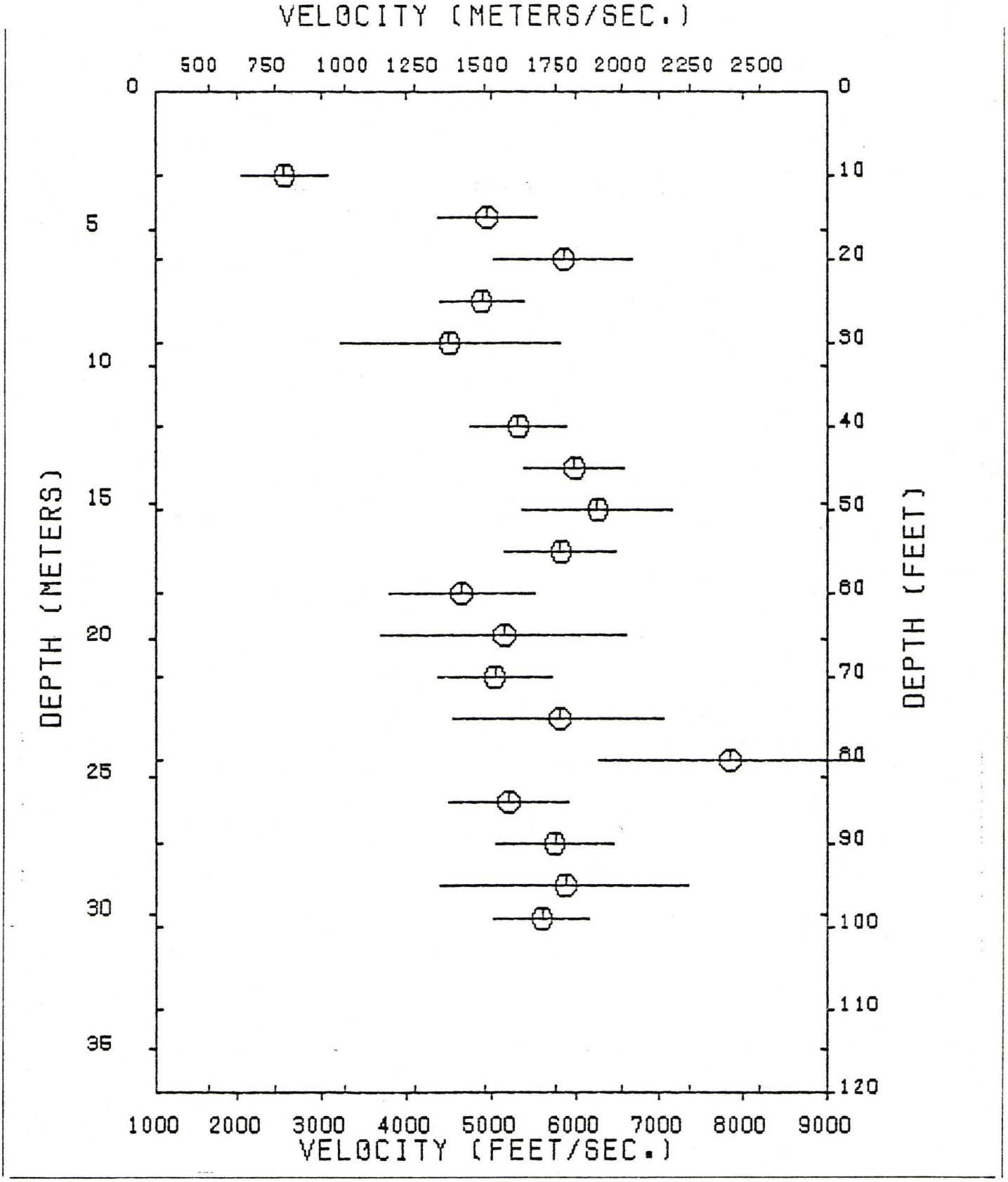
Figure No.
C-2




COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 5

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83-1101

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Figure No.
C-3

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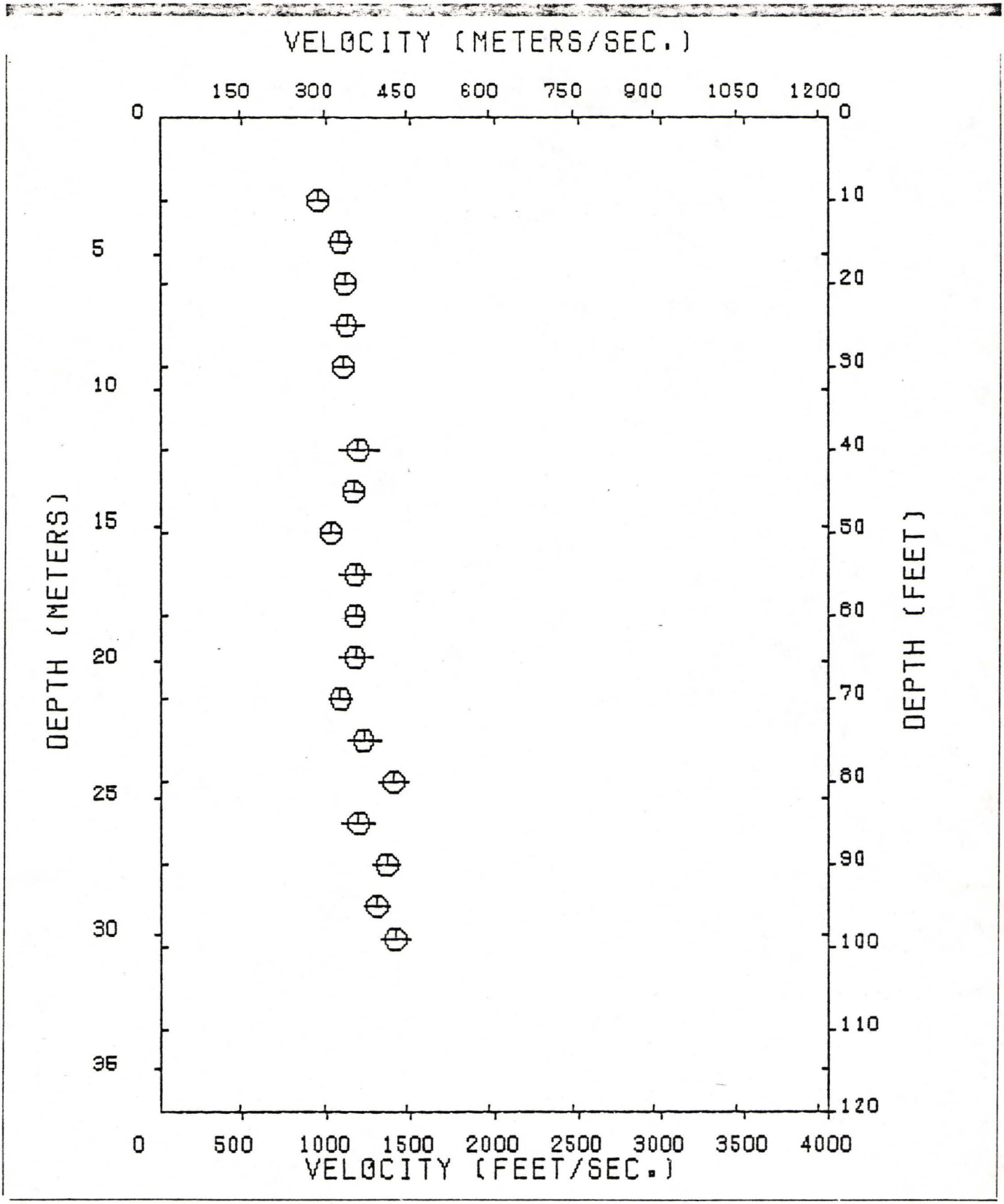


SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 5

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83-1101

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Figure No.
C-4

# Appendix D

# Gas Chromatographic Analysis

# APPENDIX D GAS CHROMATOGRAPHIC ANALYSIS

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

## D.2 FIELD PROGRAM

Specific hydrocarbon and non-hydrocarbon gases were collected during the 1981 investigation at shallow depths in Boring CEG-2, located 2,000 feet east of Union Station. 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 drawn 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 drawn 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. 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.

## D.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 $5 \times 10^{-10}$ g/ml of propane at 150°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 D.

### 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°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 or 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.

Both the volume method and weight method were used in our analyses of the data for this project. The results of one method provide a check of the other.

## D.4 RESULTS

The chromatogram for Boring CEG-2 is attached. The results of the analyses, reported as parts per million, are given in Table D-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 D-1 also identifies (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 D-2 and D-3 are included in Table D-1 for convenience. Table D-2 indicates the limits of flammability for the gases. Table D-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 D-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 smalled near some of the open boreholes and from the water pumped from the subsurface; the gas is highly soluble in water (Table D-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 D-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-2 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 D-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 sandstone, and no impermeable barrier or lithology exists between the water and the gases. Although small amounts of hydrocarbon gases can be absorbed in the water, the limit of saturation for these gases is extremely low, not exceeding 65 ppm (Table D-4). Among the non-hydrocarbon gases, only carbon dioxide and hydrogen sulfide are significantly soluble (1449 ppm and 3375 ppm, respectively; Table D-4). Because only small amounts of gas can be present in the water, only small amounts can come 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 or 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.

## D.5 CONCLUSIONS

In areas between the oil fields, such as Union Station, we may expect to find subsurface gas either within the Puente formation or trapped within the alluvium below the ground water table. 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 or 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.

Boring 4 in 1981

# TABLE D-1 Summary of Data from Gas Chromatograms

| GAS | Lower Limit of Flammability* (ppm) | TUNNEL CLASSIFICATION** GASSY 5% LEL (ppm) | TUNNEL CLASSIFICATION** EXTRA HAZARDOUS 20% LEL (ppm) | BORING NUMBER 1 Air (ppm) | 1 Cased Piezometer Sampled @ 10' Water Level @ 10' (ppm) | 2 Air (ppm) | 2 Cased Piezometer Sampled @ 18' Water Level @ 20' (ppm) | 10 Air (ppm) | 10 Cased Piezometer Sampled @ 15' Water Level @ 20' (ppm) | 11 Air (ppm) | 11 Cased Piezometer Sampled @ 40' Water Level @ 45' (ppm) | 12 Air (ppm) | 12 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) | 25 Air (ppm) | 25 Cased Piezometer Sampled @ 12' Water Level @ 15' (ppm) | 25A Air (ppm) | 25A Cased Piezometer Sampled @ 13' Water Level @ 15' (ppm) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Methane | 50,000 | 2,500 | 10,000 | - | 100 | - | 200 | ✓ | - | | | - | trace | - | a trace<br>b -<br>c - | trace | trace | - | - | - | trace |
| Ethane | 30,000 | 1,500 | 6,000 | trace† | 500 | - | 500 | - | - | | | - | 2,000 | 100 | a trace<br>b trace<br>c trace | 100 | 1,800 | - | - | 150 | 550 |
| Propane | 21,200 | 1,060 | 4,240 | - | trace | - | trace | - | - | | | - | - | - | a trace<br>b -<br>c - | - | - | - | - | trace | - |
| n-Butane | 18,600 | 930 | 5,720 | - | trace | - | trace | - | - | | | - | trace | - | a trace<br>b -<br>c - | - | trace | - | trace | trace | - |
| Isobutane | 18,000 | 900 | 5,600 | - | trace | - | trace | - | - | | | - | 150 | - | a trace<br>b -<br>c - | - | trace | - | trace | trace | - |
| n-Pentane | 14,000 | 700 | 2,800 | - | trace | - | trace | - | - | | | - | trace | - | a trace<br>b -<br>c - | - | - | - | trace | trace | - |
| Isopentane | 13,200 | 660 | 2,640 | - | trace | - | trace | - | - | | | - | trace | - | a trace<br>b -<br>c - | - | - | - | trace | trace | - |
| $C_6$* | ‡ | - | - | 800 | 1,000 | 1,600 | 1,600 | 400 | 1,200 | | | 2,000 | 4,500 | 500 | a 500<br>b 2,000<br>c trace | 100 | 1,000 | 5,500 | 5,500 | 2,500 | 2,500 |
| Nitrogen | - | - | - | 772,000 | 771,000 | 770,000 | 770,000 | 770,000 | 770,000 | | | 770,000 | 766,000 | 770,000 | a 770,000<br>b 770,000<br>c 770,000 | 770,000 | 770,000 | 769,000 | 769,000 | 768,000 | 768,000 |
| Oxygen | - | - | - | 200,000 | 200,000 | 200,000 | 200,000 | 201,000 | 200,000 | | | 200,000 | 199,000 | 201,000 | a 201,000<br>b 200,000<br>c 201,000 | 201,000 | 200,000 | 200,000 | 200,000 | 200,000 | 200,000 |
| Carbon monoxide | 125,000 | 6,250 | 25,000 | - | trace | - | trace | - | - | | | trace | trace | trace | a trace<br>b -<br>c - | - | trace | - | trace | trace | trace |
| Carbon dioxide | - | - | - | 27,000 | 28,000 | 28,000 | 28,000 | 28,000 | 28,000 | | | 28,000 | 28,000 | 28,000 | a 28,000<br>b 27,000<br>c 28,000 | 28,000 | 27,000 | 27,000 | 27,000 | 29,000 | 27,000 |
| Hydrogen sulfide | 43,000 | 2,175 | 8,700 | trace | trace | - | trace | - | trace | | | trace | trace | trace | a trace<br>b trace<br>c 100 | trace | trace | trace | trace | trace | trace |

| GAS | THRESHOLD LIMIT VALUES OF SELECTED NON-HYDROCARBON GASES ppm § | ppm §§ | BORING NUMBER 1 Air (ppm) | 1 Cased Piezometer @ 10' (ppm) | 2 Air (ppm) | 2 Cased Piezometer @ 18' (ppm) | 10 Air (ppm) | 10 Cased Piezometer @ 15' Water Level @ 20' (ppm) | 11 Air (ppm) | 11 Hole (ppm) | 19 Air (ppm) | 19 Hole (ppm) | 21 Air (ppm) | 21 Hole (ppm) | 22 Air (ppm) | 22 Hole (ppm) | 25 Air (ppm) | 25 Hole (ppm) | 25A Air (ppm) | 25A Hole (ppm) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Carbon monoxide | 50 | 100; 200; 400; 1,200; 2,000 | - | trace | - | trace | - | - | | | trace | trace | trace | a trace<br>b -<br>c - | - | trace | - | trace | trace | trace |
| Carbon dioxide | 5,000 | 5,000, 50,000; 90,000 | 27,000 | 28,000 | 28,000 | 28,000 | 28,000 | 28,000 | | | 28,000 | 28,000 | 28,000 | a 28,000<br>b 27,000<br>c 28,000 | 28,000 | 27,000 | 27,000 | 27,000 | 29,000 | 27,000 |
| Hydrogen sulfide | 10 | 10; 100; 200 | trace | trace | - | trace | - | trace | | | trace | trace | trace | a trace<br>b trace<br>c 100 | trace | trace | trace | trace | trace | trace |
| Oxygen | # | - | 200,000 | 200,000 | 200,000 | 200,000 | 201,000 | 200,000 | | | 200,000 | 199,000 | 201,000 | a 201,000<br>b 201,000<br>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 B, 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'

CCI/ESA/GRC

## TABLE D-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 D-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 D-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.

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

# Gas Chromatography
Boring No. ____

## Ryland-Cummings, Inc.

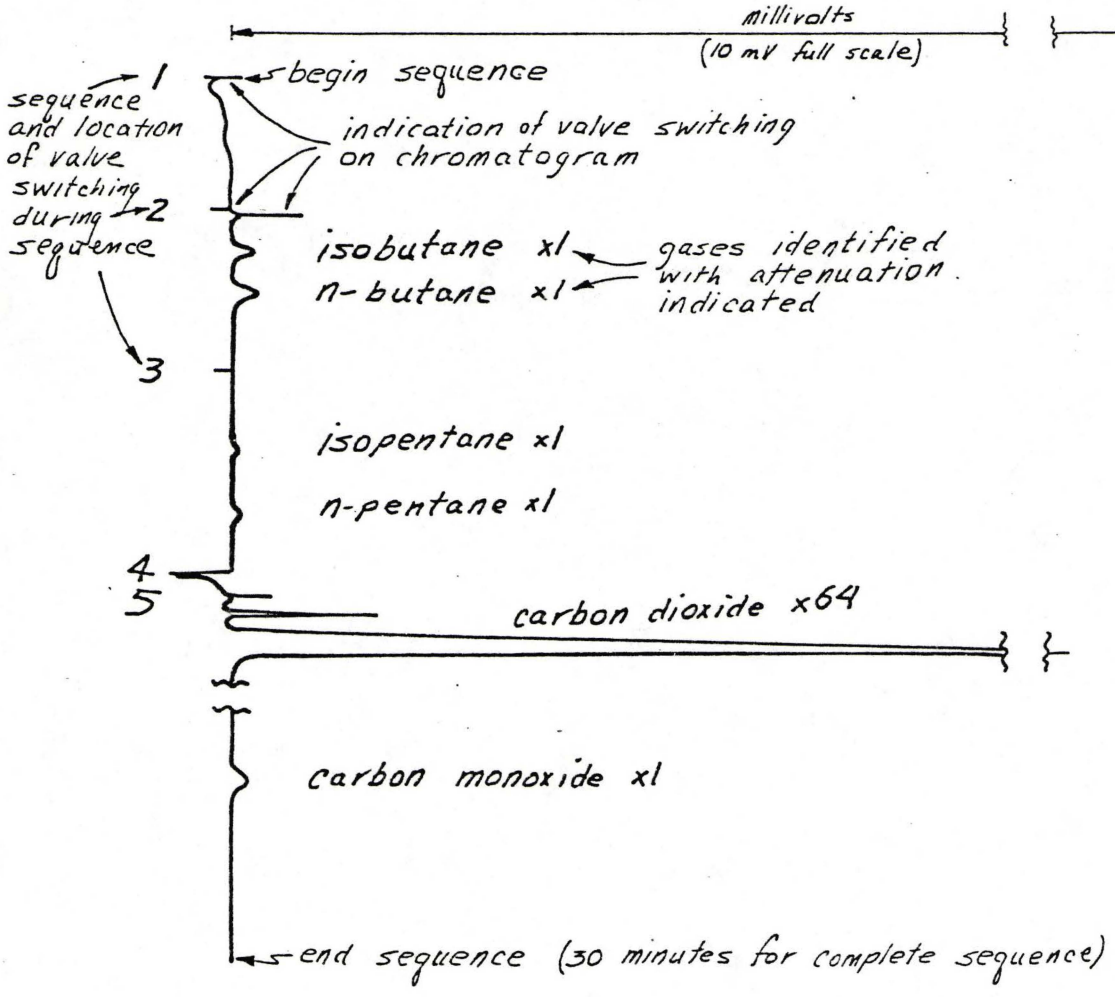
Date Sampled _____ Tested by _____

Depth of Sample _____ Column Temp. _____ Chart Speed _____

Formation _____ Helium _____

Sample Size _____ Attenuation Range _____

## LEGEND



ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

# Gas Chromatography
## Boring No. 1

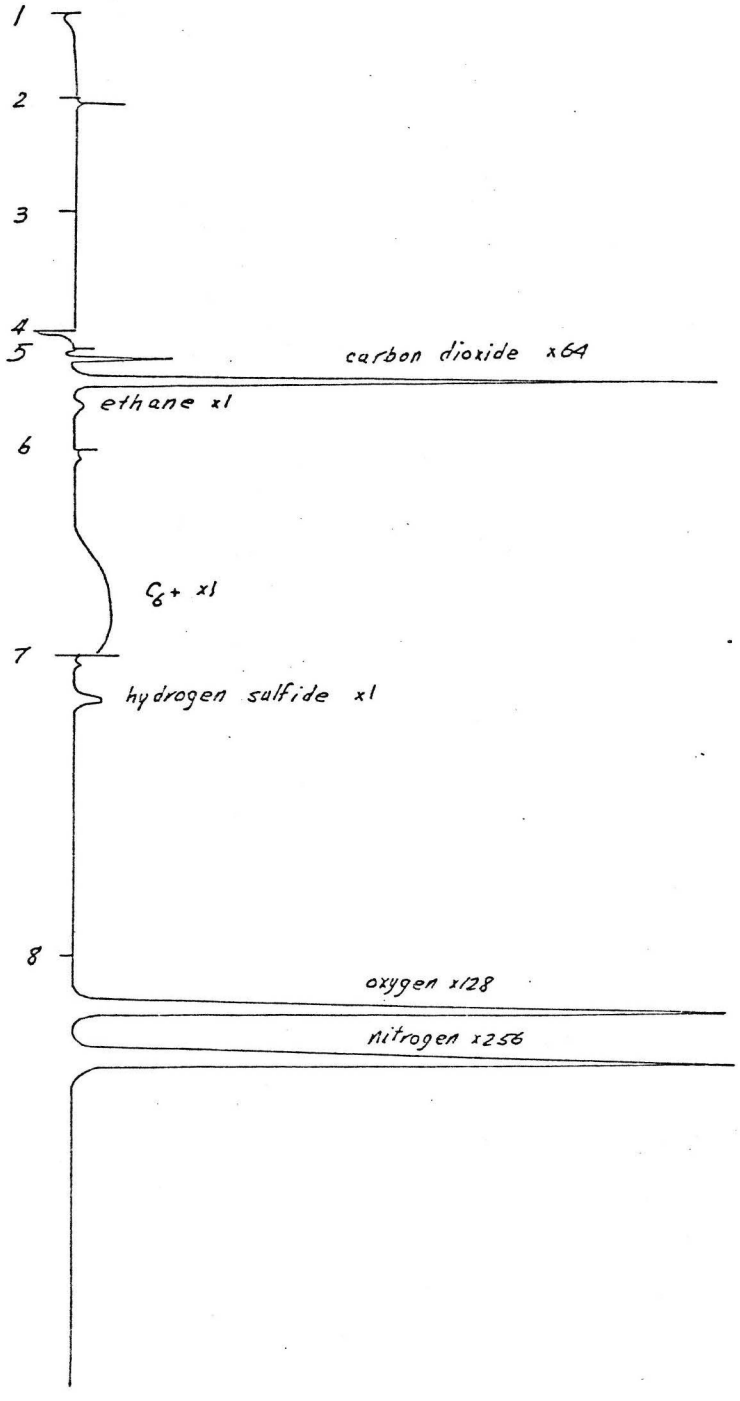
## Ryland-Cummings, Inc.

Date Sampled 2/12/81 Tested by DC SLR

Depth of Sample Air Column Temp. 70°C Chart Speed 0.5 in/min

Formation Helium 30 ml/min flow rate @ 60 psig

Sample Size 10 ml Attenuation Range as shown



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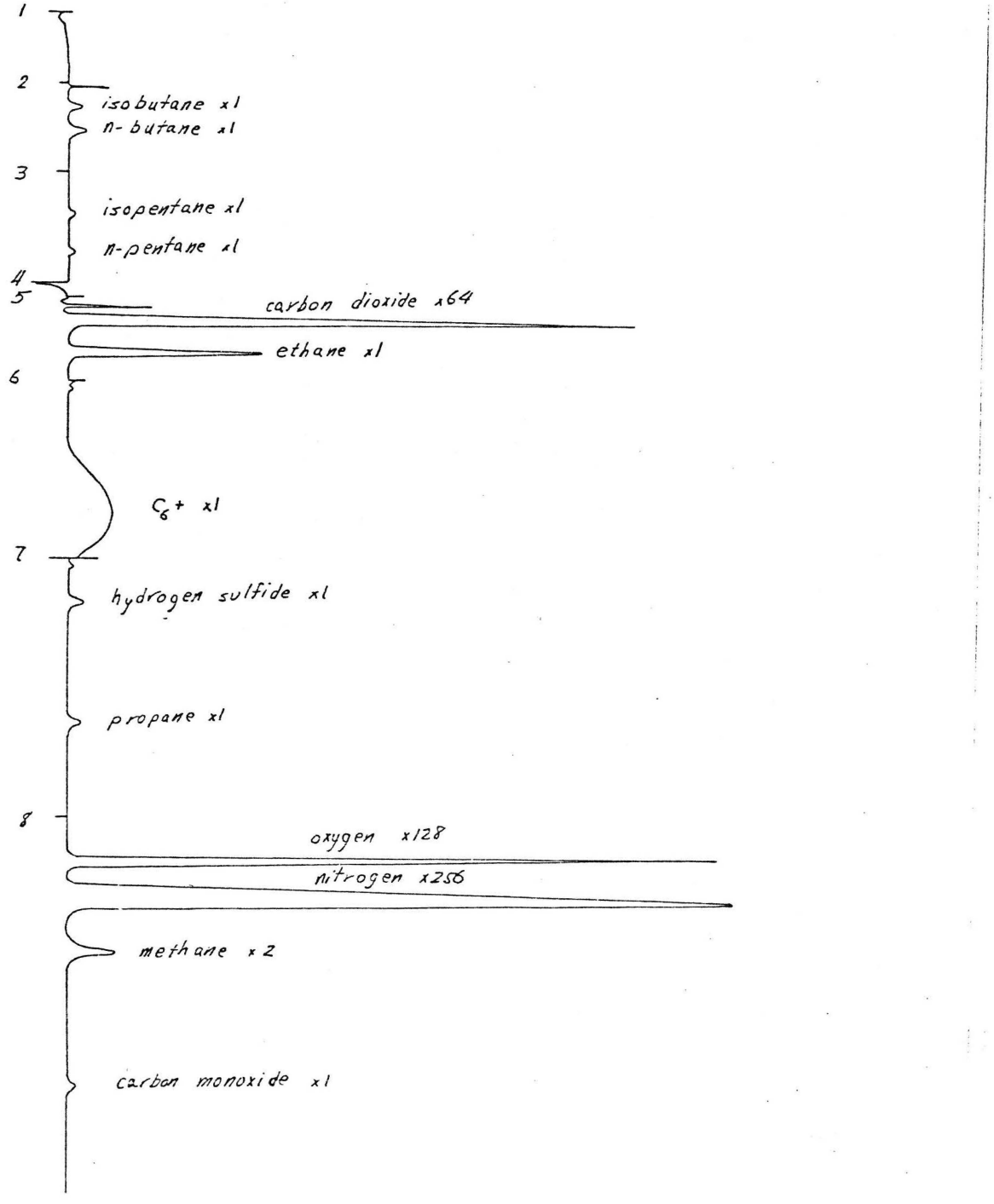
Geo/Resource Consultants

# Gas Chromatography
Boring No. 1

## Ryland-Cummings, Inc.

Date Sampled 2/12/81 Tested by DC SLR
Depth of Sample 10 ft water @ 10 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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# Gas Chromatography
## Boring No. 2

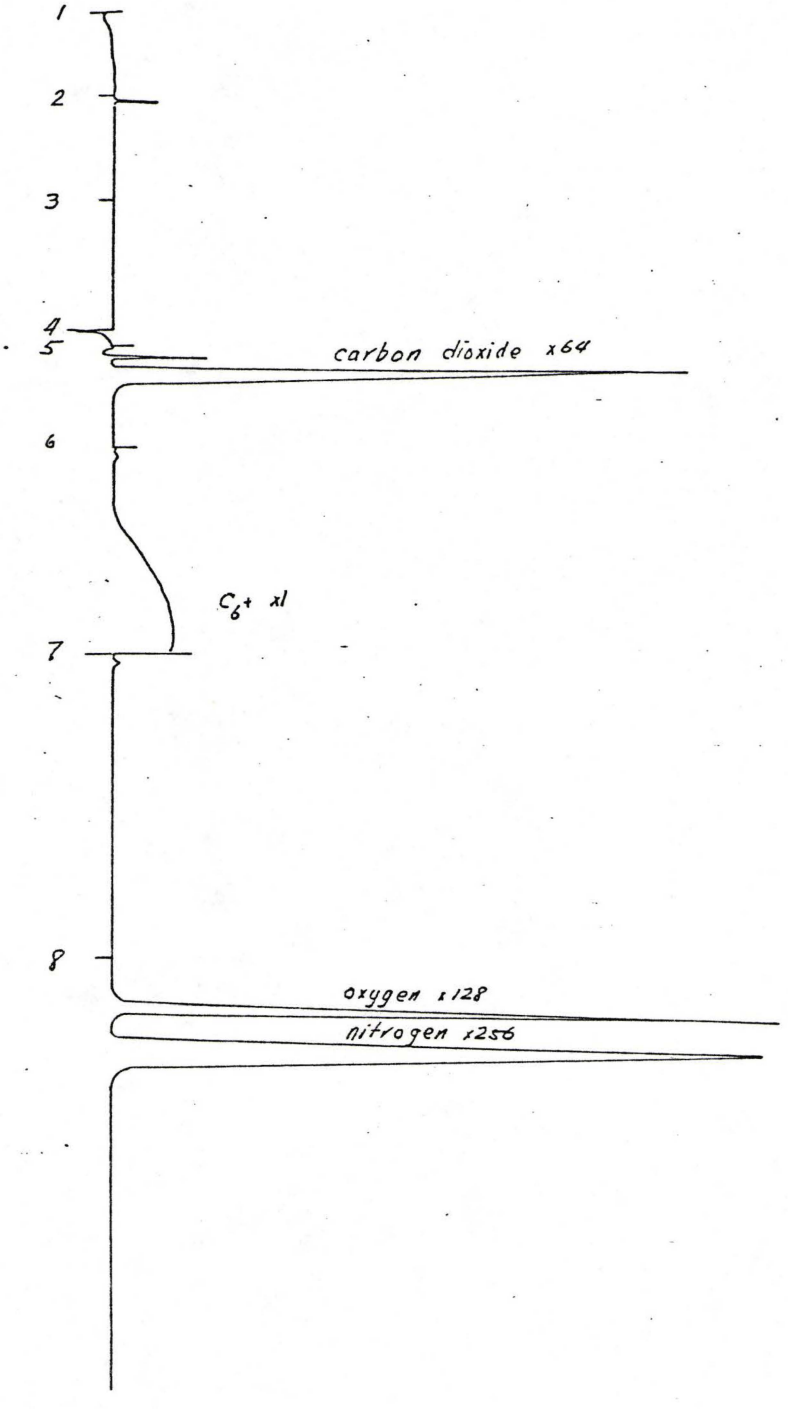
**Ryland-Cummings, Inc.**

Date Sampled 2/14/81 Tested by DC

Depth of Sample Air Column Temp. 70°C Chart Speed 0.5 in/min

Formation Helium 30 ml/min flow rate @ 60 psig

Sample Size 10 ml Attenuation Range as shown



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Ryland-Cummings, Inc.

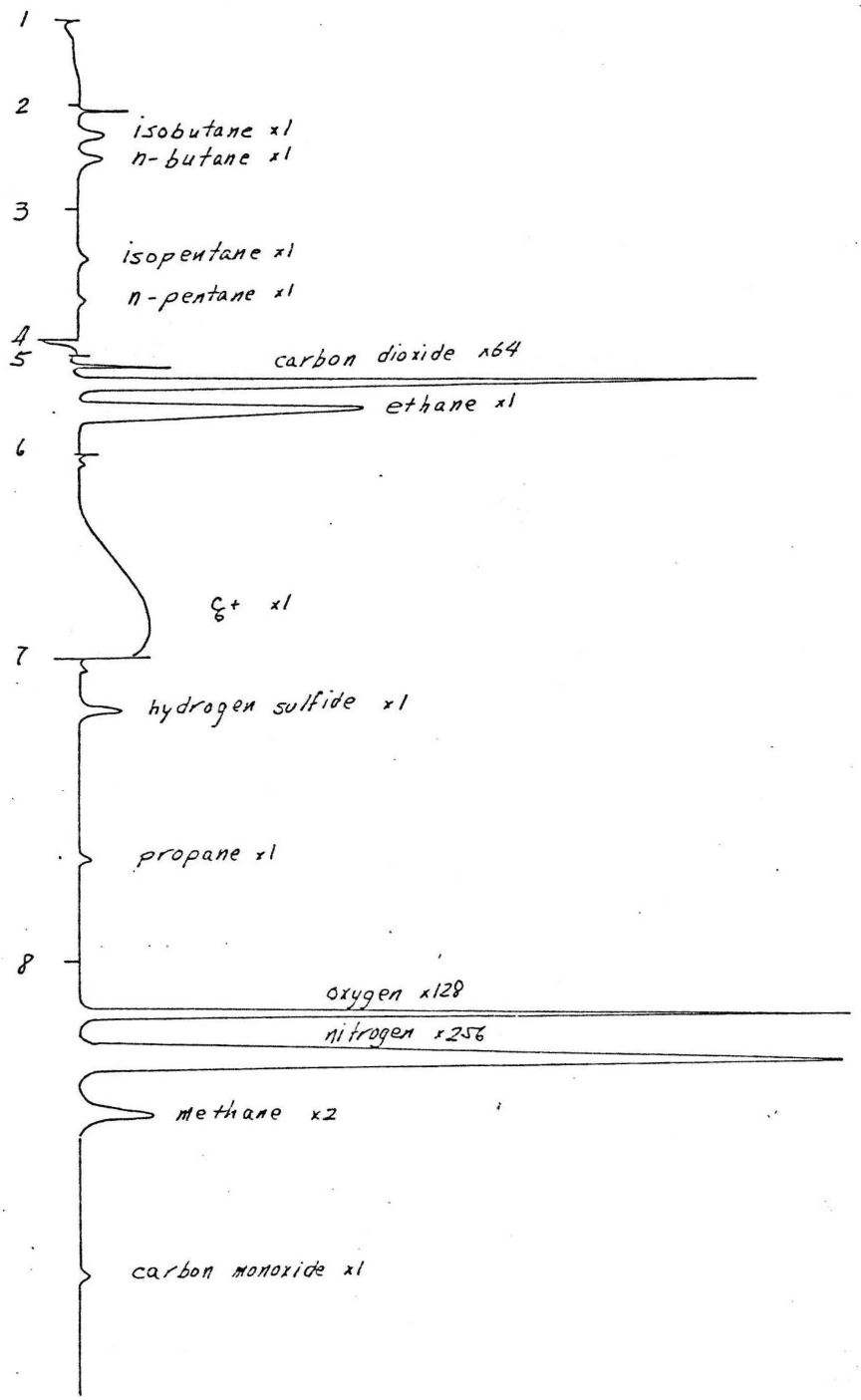
# Gas Chromatography
Boring No. 2

Date Sampled 2/14/81 Tested by DC

Depth of Sample 18 ft water @ 20 ft Column Temp. 70°C Chart Speed 05 in/min

Formation cased piezometer Helium 30 ml/min flow rate @ 60 psig

Sample Size 10 ml Attenuation Range as shown



# Appendix E

# Water Quality Analysis

# APPENDIX E. WATER QUALITY ANALYSIS

## E.1 RESULTS

Water samples were taken from Borings CEG-4 and 6. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for future tunneling activities. Chemical constituents tested are attached.

## E.2 FIELD PROGRAM

The borehole was flushed and established as a piezometer. At a later date (often several weeks) the established piezometer hole was again flushed and cleaned out. Upon achieving a clean hole, water samples were collected with an air-lifting procedure from various depths within the borehole. The water samples were collected in sterilized one-quart glass containers which were properly identified and marked in the field. The water samples taken were delivered to Jacobs Laboratories for testing.

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Earth Sciences Associates
Geo/Resource Consultants

Jacobs Laboratories

Water Quality

April 6, 1981

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

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

Attention: Buzz Spellman

Report of Chemical Analysis

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

Cation/Anion balance was not acheived on many of the samples due to the presence of an unmeasured cation, probably aluminum or barium. This fact is reflected in the large difference between the milliequivalents of total hardness, (Milligrams $CaCO_3/1 \div 50$ = 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. P81-02-186-7

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-20-81

Sample labeled: HOLE #4-2"

Conductivity: 8,450 μ mhos/cm

Turbidity: NTU

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

| | Milligrams per liter (ppm) | Milli-equivalents per liter |
|---|---|---|
| **Cations determined:** | | |
| Calcium, Ca | 76 | 3.80 |
| Magnesium, Mg | 64 | 5.27 |
| Sodium, Na | 1,800 | 78.30 |
| Potassium, K | 18 | 0.46 |
| | Total | 87.83 |
| **Anions determined:** | | |
| Bicarbonate, as $HCO_3$ | 404 | 6.62 |
| Chloride, Cl | 2,800 | 79.18 |
| Sulfate, $SO_4$ | 79 | 1.65 |
| Fluoride, F | 0.6 | 0.03 |
| Nitrate, as N | 0.3 | 0.02 |
| | Total | 87.50 |
| Carbon dioxide, $CO_2$, Calc. | 15 | |
| Hardness, as $CaCO_3$ | 453 | |
| Silica, $SiO_2$ | 39 | |
| Iron, Fe | 0.08 | |
| Manganese, Mn | < 0.01 | |
| Boron, B | 7.0 | |
| Total Dissolved Minerals, (by addition: $HCO_3$ -> $CO_3$) | 5,085 | |

Converse Ward Davis Dixon

Lab No. P81-02-186-2

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-20-81

Sample labeled: HOLE #6-2"

Conductivity: 30,000 μ mhos/cm

Turbidity: NTU

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

| | Milligrams per liter (ppm) | Milli-equivalents per liter |
|---|---|---|
| **Cations determined:** | | |
| Calcium, Ca | 1,055 | 52.75 |
| Magnesium, Mg | 210 | 17.28 |
| Sodium, Na | 6,450 | 280.58 |
| Potassium, K | 38 | 0.97 |
| | | Total 351.58 |
| **Anions determined:** | | |
| Bicarbonate, as $HCO_3$ | 230 | 3.77 |
| Chloride, Cl | 12,255 | 345.60 |
| Sulfate, $SO_4$ | 27 | 0.56 |
| Fluoride, F | 0.4 | 0.02 |
| Nitrate, as N | 0.5 | 0.04 |
| | | Total 349.99 |
| Carbon dioxide, $CO_2$, Calc. | 10 | |
| Hardness, as $CaCO_3$ | 3,500 | |
| Silica, $SiO_2$ | 39 | |
| Iron, Fe | 0.08 | |
| Manganese, Mn | 0.64 | |
| Boron, B | 38 | |
| Total Dissolved Minerals, (by addition: $HCO_3$ -> $CO_3$) | 20,230 | |

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# Appendix F

# Geotechnical Laboratory Testing

# APPENDIX F GEOTECHNICAL LABORATORY TESTING

## F.1 INTRODUCTION

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

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

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

The laboratory test data from the present investigation are presented in Table F-2, while data from the 1981 geotechnical investigation are presented in Table F-3. The geologic units listed in these tables are described in Section 5.0 of the report.

### F.1.1 Data Analysis

The summary of laboratory test results is presented in Table F-1. Figures F-1 and F-2 summarize strength and modulus data for granular alluvium existing at the site.

Data from the various tests were organized by test type and geologic unit. Where the numbers were 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:

$$m_s = a_1 \times a_2 \times \ldots \times a_n)^{1n}$$

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.

## F.2 INDEX AND IDENTIFICATION

### F.2.1 Visual Classification

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

### F.2.2 Grain-Size Distribution

Grain-size distribution tests were performed on representative samples of the geologic units to assist in the soils classification and to correlate test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D-422-63 test method. Combined sieve and hydrometer analyses were performed on selected samples which had a significant percentage of soil particles passing the No. 200 sieve. Results of these analyses are presented in the form of grain-size distribution or gradation curves on Figures F-3 through F-5.

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.

### F.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 F-6 and on Tables F-2 and F-3.

### F.2.4 Moisture Content

Moisture content determinations were performed on selected soil samples to assist in their classification and to evaluate ground water location. The testing procedure was a modified version of the ASTM D-2261 test method. Test results are presented on Tables F-2 and F-3.

### F.2.5 Unit Weight

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

The test procedure entailed measuring specimen dimensions with a precision ruler or micrometer. Weights of the sample were than determined at natural moisture content. Total unit weight was computed directly from data obtained from the two previous steps. Dry density was calculated from the moisture content found in Section F.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables F-2 and F-3.

## F.3 ENGINEERING PROPERTIES: STATIC

### F.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 F-2 and F-3.

F.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:

F.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 to more fully utilize the available samples. 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 sometimes repeated a third time at a still higher confining pressure, and the sample was loaded until failure occurred.

F.3.2.2 Unconsolidated Undrained (UU), Quick (Q) Triaxial 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.
- The specimen was loaded at a controlled rate of strain to failure, or to 15% to 20% axial strain. The specimen was neither saturated nor consolidated prior to loading.

Results of the triaxial compression tests are presented in Figures F-9 through F-10.

F.3.3 Direct Shear

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

Each test specimen was trimmed, soaked and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until a maximum shear strength was developed. Fine-grained samples were allowed to consolidate prior to shearing. The maximum developed shear strengths are summarized on Tables F-2 and F-3.

Progressive direct shear tests were performed on selected undisturbed samples of coarse-grained material. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. A new normal load was then applied, and the specimen was sheared a second time. This process was repeated for several different normal loads. Results of the progressive direct shear tests are summarized on Tables F-2 and F-3.

F.3.4 Swell

No swell tests were performed for this design unit.

F.3.5 Consolidation

Consolidation tests were performed on selected undisturbed soil samples. Test specimens were 1-inch thick by 2.42-inch diameter and were obtained either directly from a Converse ring sample or by trimming a 3-inch diameter Shelby tube sample.

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

Results of consolidation tests on the undisturbed samples are presented on Figures F-7 and F-8.

F.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 F-2 and F-3.

F.3.7 Porosity

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

$$e = \frac{1-Vs}{Vs} \text{ where } Vs = \frac{\gamma_d}{G \times \gamma_w} \text{ and } n = \frac{e}{1+e}$$

$\gamma_w$ = unit weight of water
$\gamma_d$ = unit dry weight of soil
G = specific gravity of soil solids.

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

## F.4 ENGINEERING PROPERTIES: DYNAMIC

### F.4.1 Resonant Column

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

#### F.4.1.1 Sample Preparation and Handling

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

#### F.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures, ($\sigma 3c$), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressures and progressing to the higher confining pressures. At each confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures F-13 and F-14.

F.4.1.3 Apparatus

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

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

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

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

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

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

F.4.1.4 Data Reduction

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

F.4.2 Cyclic Triaxial Compression

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

F.4.2.1 Sample Preparation and Handling

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

F.4.2.2 Test Conditions and Parameters

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

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

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

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

F.4.2.3 Apparatus

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

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

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

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

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

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

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

F.4.2.4 Data Reduction

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

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

Graphs of the test results appear on Figures F-11 and F-12.

F.4.3 Dynamic Triaxial Compression

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

F.4.3.1 Sample Preparation and Handling

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

F.4.3.2 Test Conditions and Parameters

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

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

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

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

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

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

F.4.3.3 Apparatus

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

F.4.3.4 Data Reduction

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

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

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

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

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

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

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

Results of the dynamic triaxial tests are presented in Figures F-15 and F-16.

TABLE F-1 Summary of Laboratory Test Results - Union Station A-135

| Unit | | Total Density (pcf) | Dry Density (pcf) | Moisture Content (%) | Unconfined Compressive Strength (ksf) | Triaxial Tests: Direct Shear Test Peak Shear Strength ϕ (deg) | Direct Shear Test Peak Shear Strength c (ksf) | Peak Undrained Strength ϕ (deg) | Peak Undrained Strength c (ksf) | Peak Effective Strength ϕ (deg) | Peak Effective Strength c (ksf) | Youngs Modulus E/ 3c (ksf/ksf) | Permeability (cm/sec) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Fill | Number of Tests | 4 | 4 | 4 | 2 | -0- | | -0- | | -0- | | -0- | -0- |
| | High | 137 | 115 | 32 | 2.6 | - | - | - | - | - | - | - | - |
| | Low | 117 | 89 | 9 | 2.4 | - | - | - | - | - | - | - | - |
| | Mean | 126 | 107 | 19 | - | - | - | - | - | - | - | - | - |
| | Std. Deviation | ±8.2 | ±12.1 | ±9.8 | - | - | - | - | - | - | - | - | - |
| Sand and Gravel | Number of Tests | 9 | 9 | 9 | -0- | 15 D.S. | | -0- | | 3 Tx | | 3 C | 2 |
| | High | 142 | 130 | 17 | - | 47° | 1.9 | - | - | - | - | 600 | 6.2 E-4 |
| | Low | 125 | 115 | 8 | - | 36° | .4 | - | - | - | - | 270 | 5.4 E-6 |
| | Mean | 137 | 123 | 21 | - | 36° | .7 | - | - | 39° | .1 | 365 | 1.1 E-4 |
| | Std. Deviation | ±6.6 | ±5.6 | ±4.7 | - | - | | - | - | - | - | - | - |
| Sand and Silty Sand | Number of Tests | 14 | 14 | 14 | -0- | 12 D.S. | | -0- | | 3 Tx | | 1 C 3 Tx | 2 |
| | High | 140 | 115 | 29 | - | 32° | 1.0 | - | - | - | - | 390 | 6.1 E-4 |
| | Low | 105 | 95 | 9 | - | 32° | 0 | - | - | - | - | 210 | 4.0 E-4 |
| | Mean | 125 | 104 | 20 | - | 32° | .5 | - | - | 37° | .6 | 270 | - |
| | Std. Deviation | ±9.0 | ±6.7 | ±6.2 | - | - | - | - | - | - | - | - | - |
| Claystone/ Siltstone | Number of Tests | 8 | 8 | 8 | 5 | 3 D.S. | | -0- | | -0- | | -0- | -0- |
| | High | 129 | 105 | 33 | 12.6 | - | - | - | - | - | - | - | - |
| | Low | 117 | 88 | 23 | 1.4 | - | - | - | - | - | - | - | - |
| | Mean | 124 | 98 | 26 | 8.7 | 27° | 1.2 | - | - | - | - | - | - |
| | Std. Deviation | ±3.9 | ±5.6 | ±3.7 | ±4.4 | - | - | - | - | - | - | - | - |

TABLE F-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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5-1 | C-1 | 2 | Silty Fine to Coarse Sand | F | 115 | 9 | | | | | | | | | | | | |
| | C-2 | 6 | Silty Fine Sand | $A_1$ | 95 | 11 | | | | | | | | | | | | |
| | C-3 | 30 | Fine to Medium Sand | $A_1$ | 117 | 17 | | | | | | | | | | | | |
| | C-5 | 46 | Silty Fine to Medium Sand | $A_1$ | 122 | 15 | | | | | | | | | | | | |
| | C-6 | 46 | Fine to Coarse Sand to Silty Fine to Medium Sand | $A_1$ | 127 | 12 | | | | | | | | | | | | |
| | C-7 | 66 | Silty Fine Sand to Fine Sand | $A_1$ | 115 | 22 | | | | | | | | | | | | |
| - | - | | Remolded Sand & Gravel | | 121 | 4 | | | 4.5E-4(10)<br>8.1E-4(25)<br>6.4E-4(40) | | | | | | X | | | X |
| 5-2 | C-1 | 1 | Clayey Fine to Medium Sand to Clayey Silt | F | 114 | 20 | | | | 2.40 | | | | | | | | |
| | C-2 | 21 | Fine to Coarse Sand | $A_1$ | 117 | 8 | | | 3.5E-4(15) | | 39.7 | .17 | | | X | | | |
| | C-3 | 45 | Fine to Coarse Sandy Gravel | $A_1$ | 127 | 8 | | | | | 42.3 | .76 | | | X | | | |
| | C-4 | 70 | Fine to Medium Sand | $A_1$ | 112 | 17 | | | | | 30.2 | .19 | | | X | | X | |
| | C-5 | 81 | Silty Fine Sand | $A_1$ | 103 | 24 | | | | | | | | | | | | |
| 5-3 | C-1 | 6 | Clayey Silt | F | 89 | 32 | | | | 2.48 | | | | | | | | |
| | C-2 | 51 | Fine Sandy Clayey Silt | $A_2$ | 110<br>100 | 17<br>26 | | | 0000<br>0000 | | 31.0 | .42 | | | X | | | |

TABLE F-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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | C-3 | 71 | Fine to Medium Sand | $A_1$ | 108 | 20 | | | 5.4E-4(10)<br>6.1E-4(25) | | | | | | X | | | X |
| | C-4 | 80 | Silty Fine Sand to Fine Sandy Silt | $A_1$ | 97 | 27 | | | | | | | | | | | | |
| 5-4 | C-1 | 1 | Silty Sand with Debris | F | 109 | 15 | | | | | | | | | | | | |
| | C-2 | 11 | Fine Sand | $A_1$ | 102 | 9 | | | | | | | | | | | | |
| | C-3 | 21 | Fine to Coarse Sand & Gravel | $A_1$ | 115 | 9 | | | | | 40.0 | .17 | | | X | | | |
| | C-4 | 31 | Fine to Medium Sand with Clay Lenses | $A_1$ | 109 | 19 | | | | | | | | | | | | |
| | C-5 | 51 | Fine to Coarse Sand with Clay & Silt Lenses | $A_1$ | 105 | 25 | | | | | 26.3 | .17 | | | | | X | |
| | C-6 | 61 | Fine to Medium Sand | $A_1$ | 117 | 14 | | | | | | | | | | | | |
| | C-7 | 71 | Silty Fine Sand to Fine Sandy Silt | $A_1$ | 96 | 29 | | | | | | | | | | | | |
| | C-8 | 79 | Fine to Coarse Sandy Gravel | $A_1$ | 129 | 10 | | | | | | | | | | | | |
| 5-5 | C-1 | 5 | Sandy Silt to Silty Sand | $A_1$ | 99 | 18 | | | | | | | | | | | | |
| | C-2 | 15 | Fine to Coarse Sandy Gravel | $A_1$ | 123 | 13 | | | | | | | | | | | | |
| | C-3 | 25 | Silty Fine Sand with Clayey Silt Lenses | $A_1$ | 86 | 32 | | | | | 30.8 | .11 | | | X | | | |
| | C-4 | 36 | Fine to Coarse Sand & Gravel | $A_1$ | 130 | 9 | | | | | 48.5 | 1.15 | | | X | | X | |

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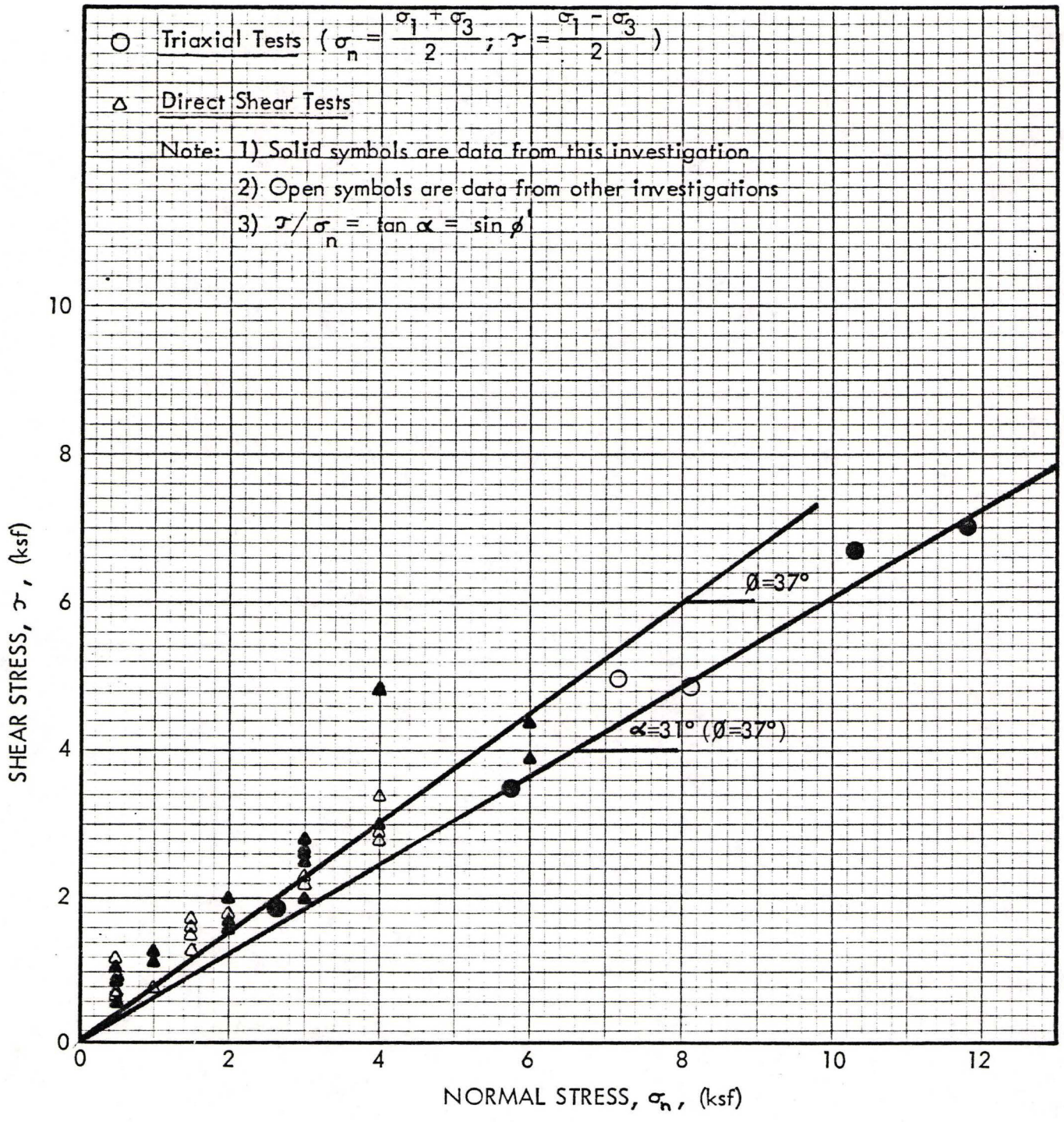
TABLE F-2 LABORATORY TEST DATA

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | C-5 | 60 | Fine to Medium Sand | $A_1$ | 105 | 23 | | | | | 31.5 | .12 | | | X | | X | |
| | C-6 | 71 | Silty Fine Sand | $A_1$ | 99 | 27 | | | | | | | | | | | | |
| | C-7 | 99 | Siltstone | C | 97 | 26 | | | | 9.01 | | | | | | | | |

TABLE F-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 | | Particle Size Cumulative % Passing Sieve No. | | | Unconfined Compression Strength (psi) | Kv, Coefficient of Permeability (cm/sec) | Coefficient of Cc (in/in per Log Cycle) | Specific Gravity | Porosity (n) | Undrained Quick Direct Shear | | One-Dimensional Swell % | Cyclic Triaxial (Liquefaction) | Dynamic Triaxial (Stress/Strain) | Resonant Column Test | Triaxial Compression |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | LL | PI | 4 | 40 | 200 | | | | | | ϕ, deg | c, ksf | | (ksf Normal Load) | | | |
| 4 | S1 | 118 | Clayey siltstone, massive | C | 89 | 31 | | | | | | 60.5 | | | | | | | | | | | |
| | S2 | 135 | Silty claystone, massive | C | 107 | | | | | | | 139 | | | | | | | | | | | |
| | S3 | 148 | Silty claystone, massive | C | 105 | 20 | | | | | | 34.1 | | | | | | | | | | | |
| 5 | PB3 | 90 | Claystone | C | | | 49 | 18 | | | | | | | | | | | | | | | |
| | PB15 | 119 | Micaceous claystone | C | | | 40 | 13 | | | | | | | | | | | | | | | |
| | PB26 | 148 | Claystone, bedded | C | | | 40 | 13 | | | | | | | | | | | | | | | |
| 6 | S1 | 21 | Gray medium to coarse sand | $A_1$ | 107 | 18 | | | 75 | 32 | 3 | | 5.4E-6 | | | 36.4 | | | | X | | | |
| | S2 | 41 | Gray fine to medium sand | $A_1$ | 99 | 21 | | | 94 | 67 | 10 | | 4.0E-4 | | | 40.8 | | | | X | | | |
| | C1 | 80 | Silty claystone | C | 94 | 29 | | | | | | | | | | | 27 | 1.2 | | | | | |
| | C1 | 80 | Silty claystone | C | 105 | 23 | | | | | | | | | | | | | | | X | | |
| | S3 | 94 | Clayey micaceous siltstone | C | 100 | 25 | 42 | 18 | | | | 62.5 | | | | | | | | | | | |
| | S3 | 95 | Clayey micaesous siltstone | C | 94 | 27 | | | | | | | | | | | | | | | X | | |
| | S4 | 97 | Claystone, massive, silt pockets | C | 102 | 23 | | | | | | 87.6 | | | | | | | | | | | |
| | S5 | 104 | Claystone, massive, silt pockets | C | 88 | 33 | | | | | | 80.9 | | | | 37.3 | | | | | | | |
| | S6 | 124 | Claystone, bedded thin sandstone | C | 103 | 22 | | | | | | 9.6 | | | | | | | | | | | |
| | S7 | 139 | Gray fine sand | C | 112 | 22 | | | | | | | | | | | | | | | X | | |
| | S7 | 141 | Gray fine sand | | 113 | 16 | | | | | | | | | | | 23.5 | 0.65 | | | | | |




# SUMMARY OF EFFECTIVE STRENGTH DATA - GRANULAR ALLUVIUM

UNION STATION
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METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-1

Converse Consultants — Geotechnical Engineering and Applied Sciences

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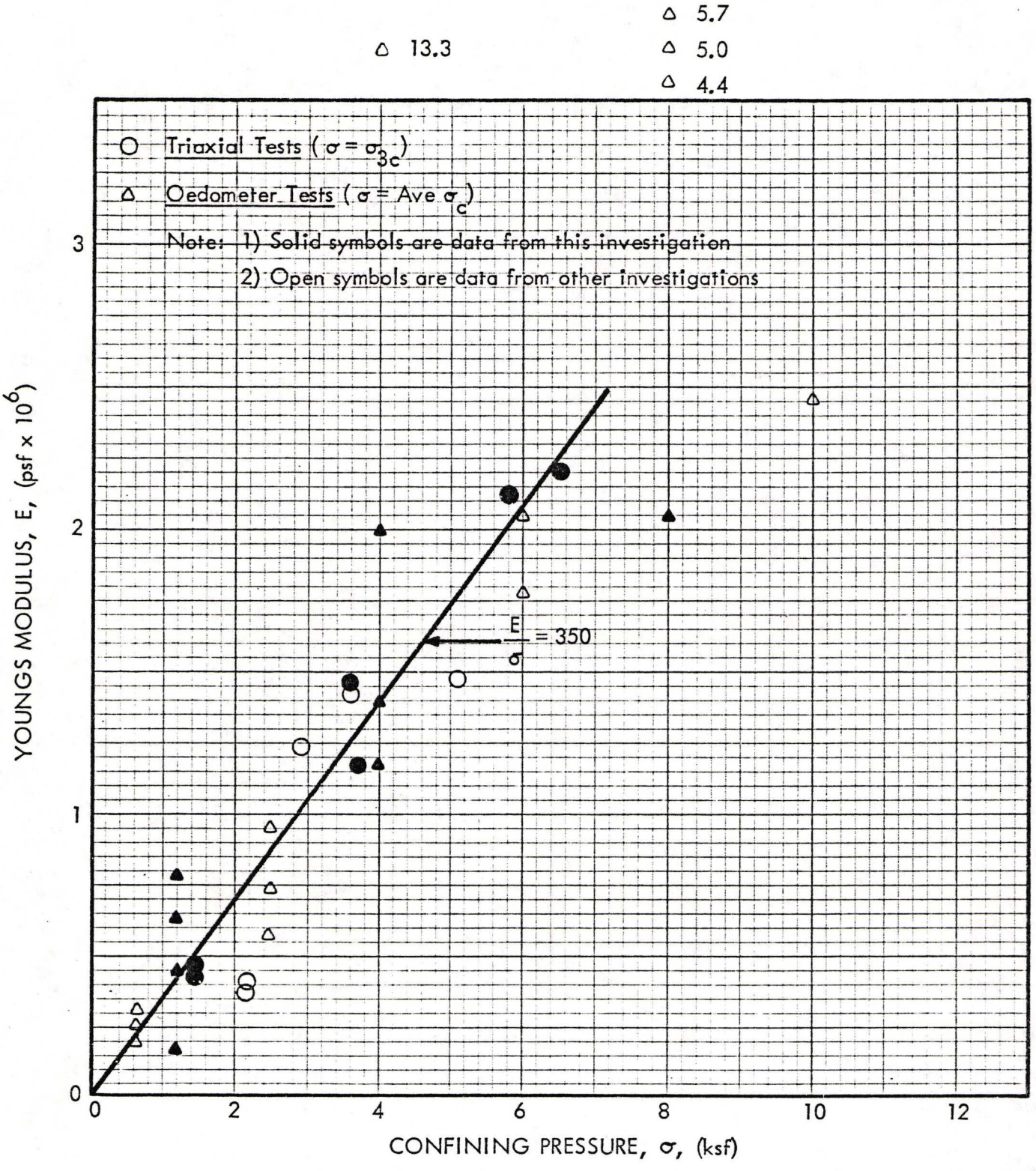


## SUMMARY OF MODULUS DATA - GRANULAR ALLUVIUM

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Project No.
83-1101

Converse Consultants — Geotechnical Engineering and Applied Sciences

Figure No.
F-2

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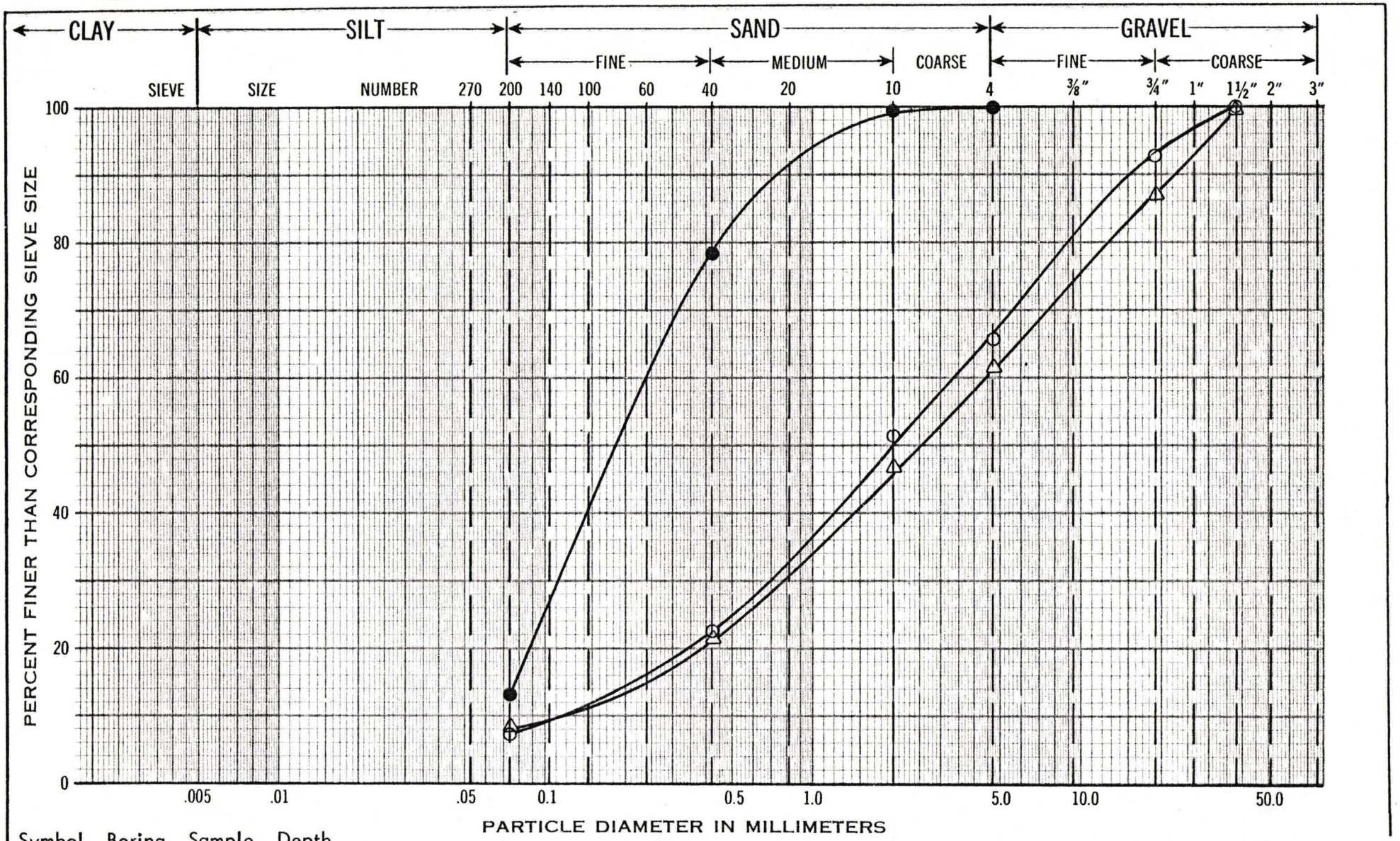


| Symbol | Boring | Sample | Depth |
|---|---|---|---|
| ● | 5-3 | C-3 | 71' |
| ○ | 5-4 | C-3 | 21' |
| △ | 5-5 | C-3 | 26' |
| △ | 5-5 | C-4 | 36' |

## GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
F-3

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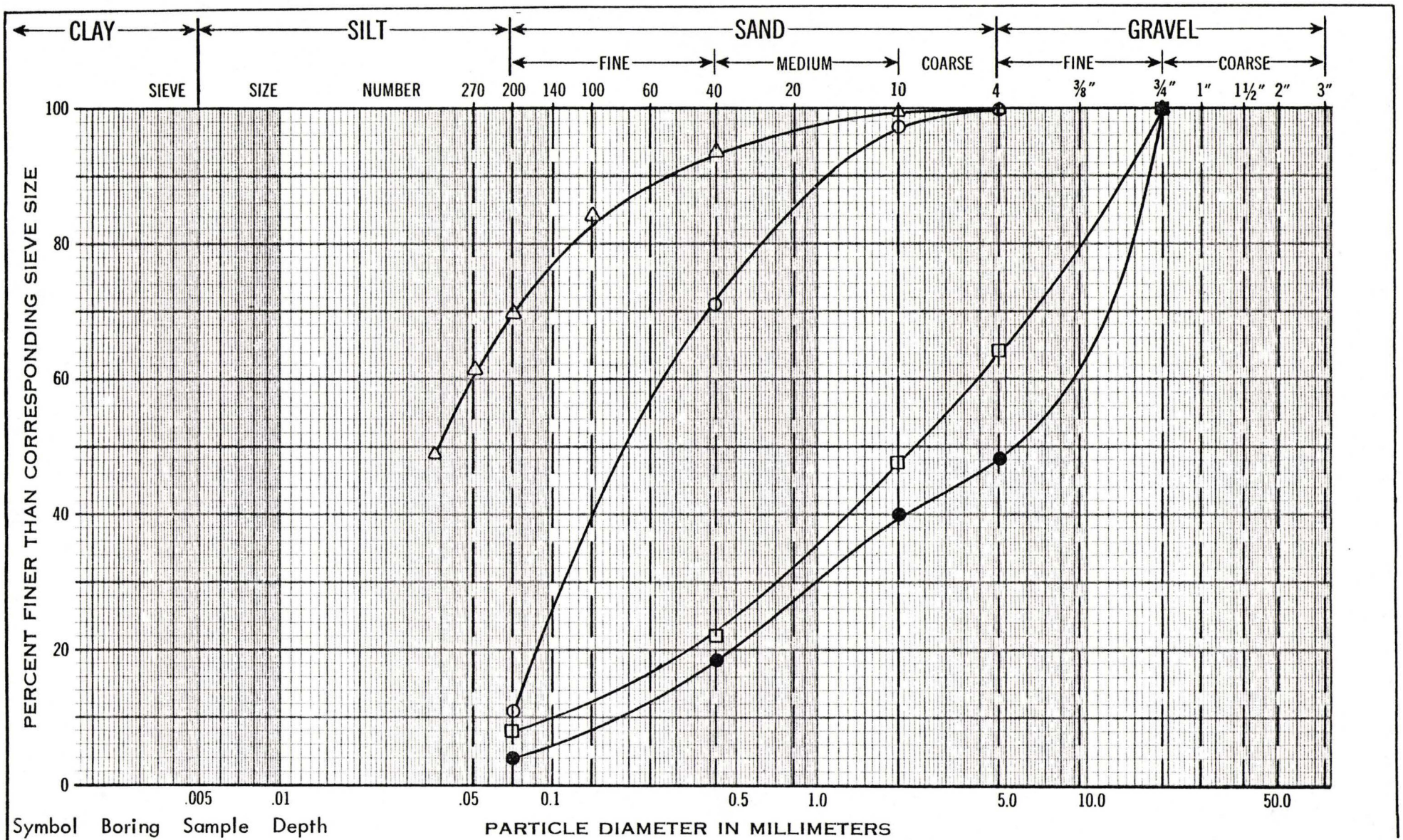
FORM NO. [redacted]1 Approved for publication _____ by _____




| Symbol | Boring | Sample | Depth |
|---|---|---|---|
| ● | 5-2 | C-2 | 21' |
| ● | 5-2 | C-3 | 46' |
| ○ | 5-2 | C-4 | 70' |
| ○ | 5-5 | C-5 | 60' |
| △ | 5-3 | C-2 | 51' |
| □ | Remolded Sample (Sand & Gravel) | | |

# GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
F-4

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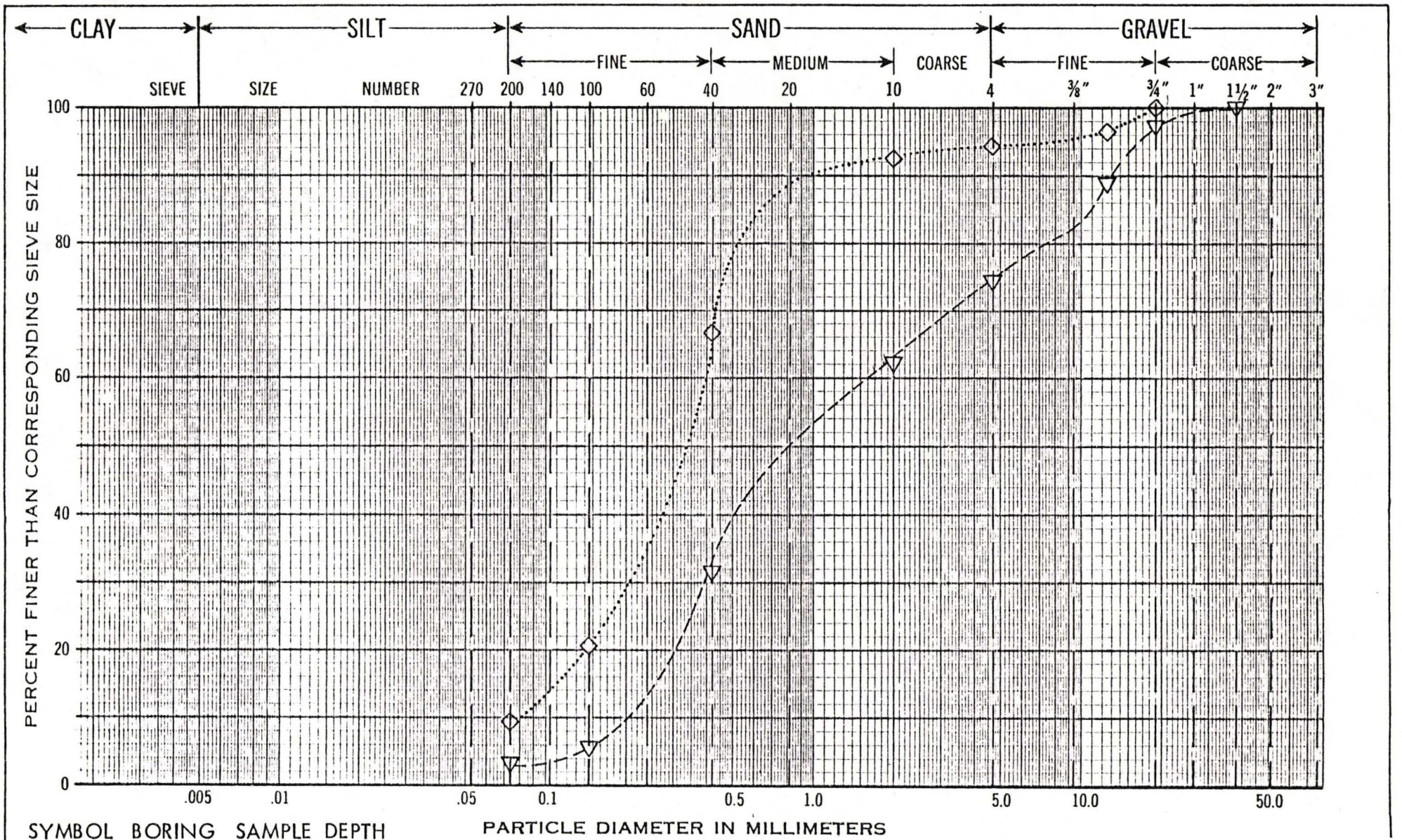


| SYMBOL | BORING | SAMPLE | DEPTH |
|---|---|---|---|
| --▽--- | 6 | S-1 | 21' |
| ·····◇····· | 6 | S-2 | 42' |

## GRAIN-SIZE DISTRIBUTION CHART

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Project No.
83-1101

Figure No.
F-5

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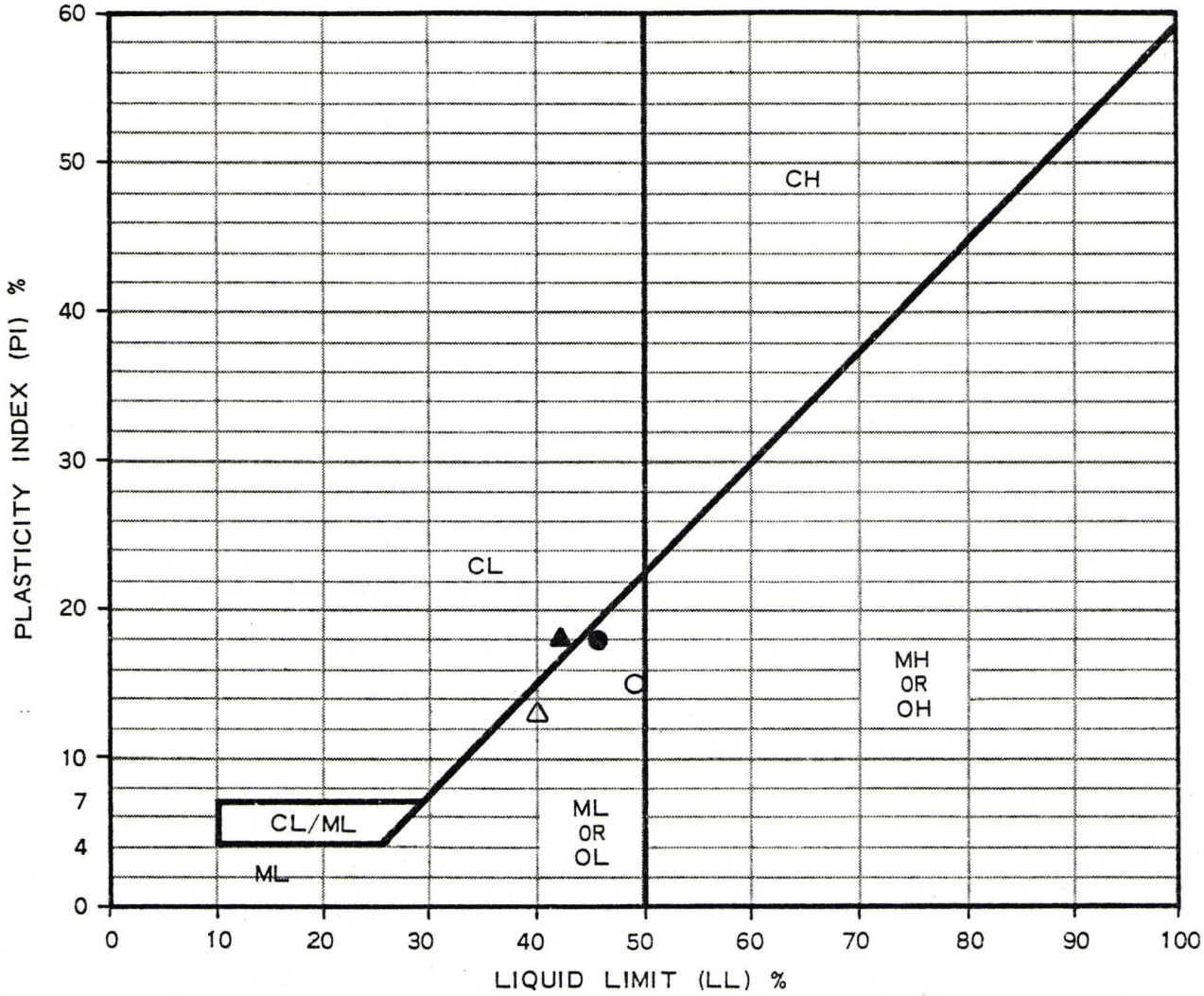
11-894

502




| Symbol | Classification and Source | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index (%) | % Passing No. 200 Seive |
|---|---|---|---|---|---|
| ○ | BH 5, 90' (ML) | 49 | 34 | 15 | - |
| ● | BH 5, 119' (ML) | 46 | 28 | 18 | - |
| △ | BH 5, 148' (ML) | 40 | 27 | 13 | - |
| ▲ | BH 6, 94' (CL) | 42 | 24 | 18 | - |

# PLASTICITY CHART

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Project No.
83-1101

Figure No.
F-6

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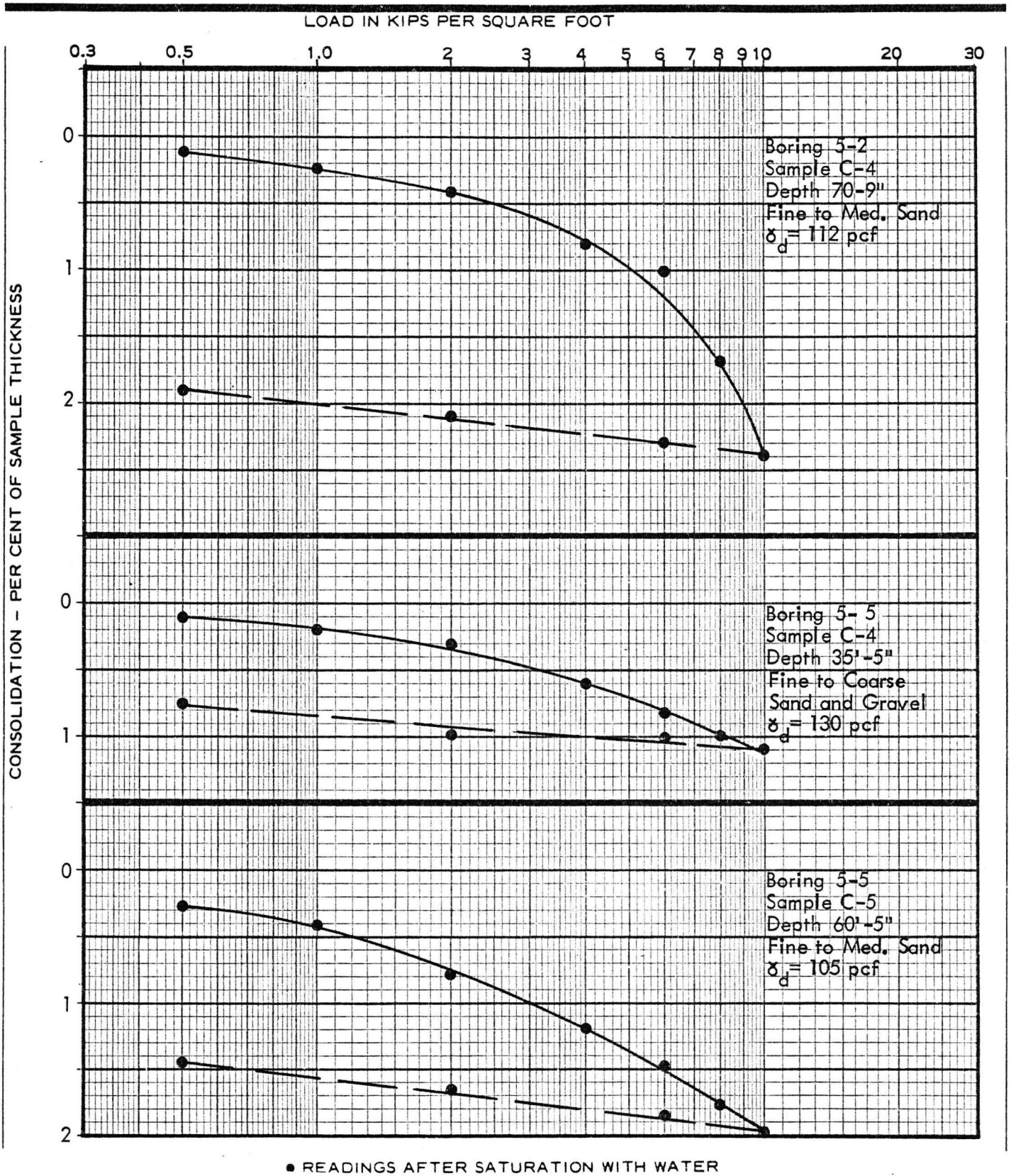
206




## CONSOLIDATION TESTS

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-7

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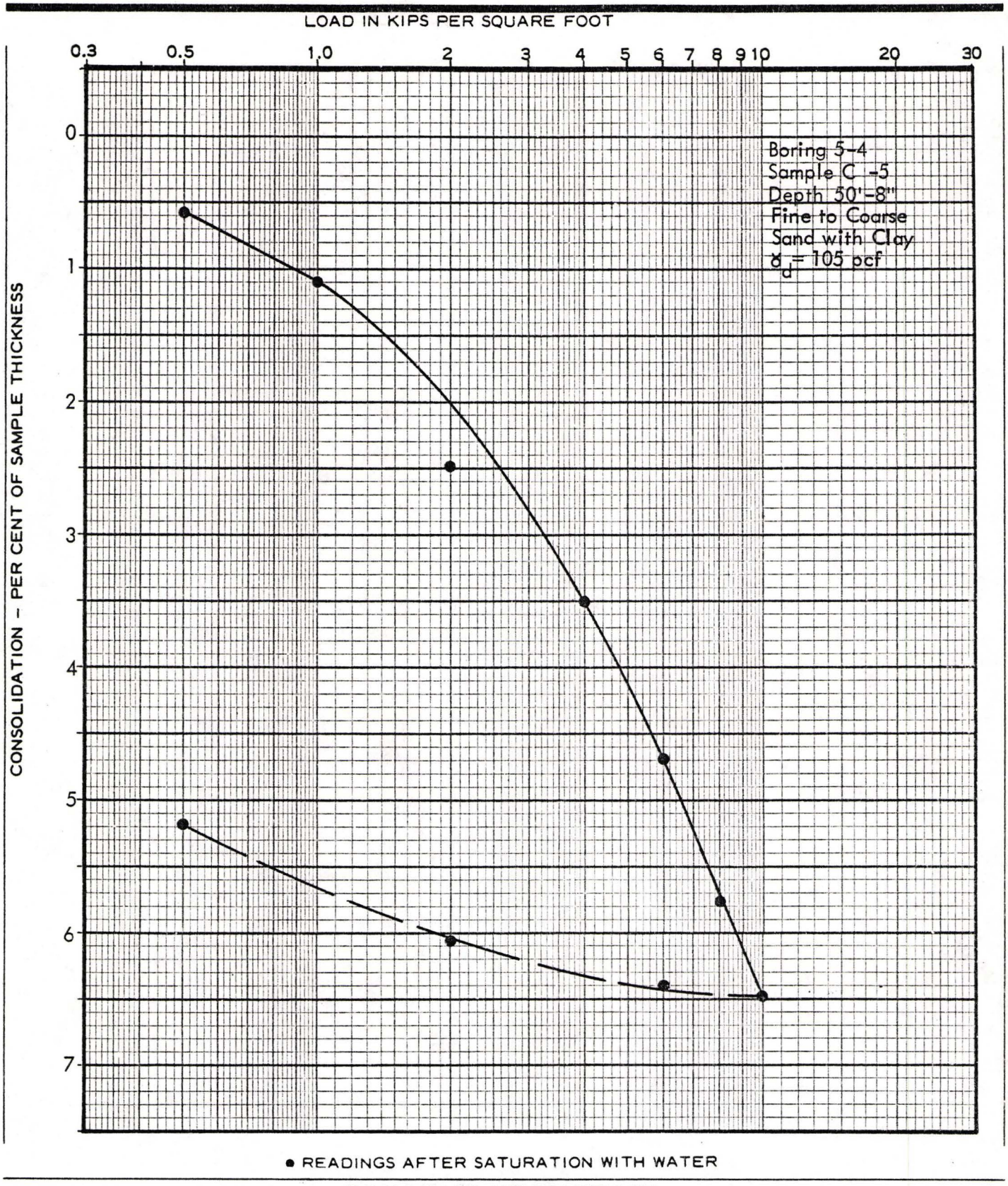


# CONSOLIDATION TESTS

UNION STATION
Southern California Rapid Transit District
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Project No.
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Figure No.
F-8

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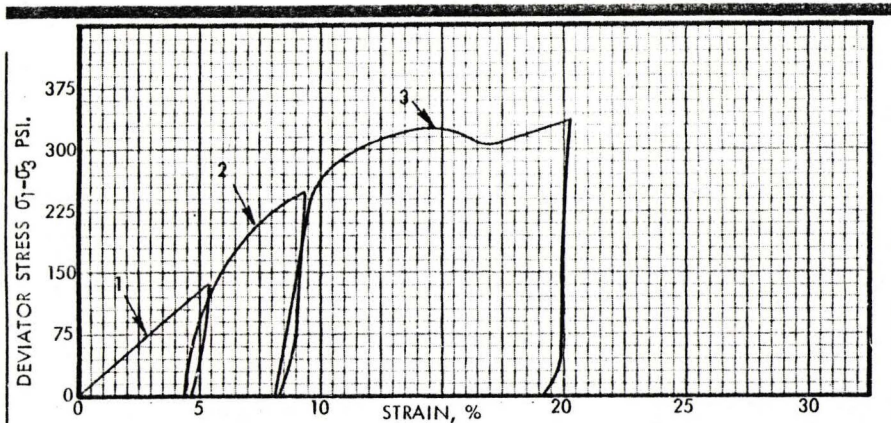
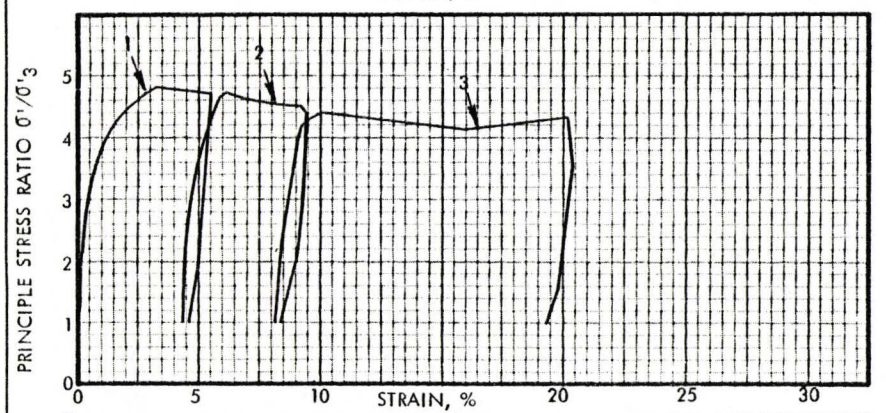
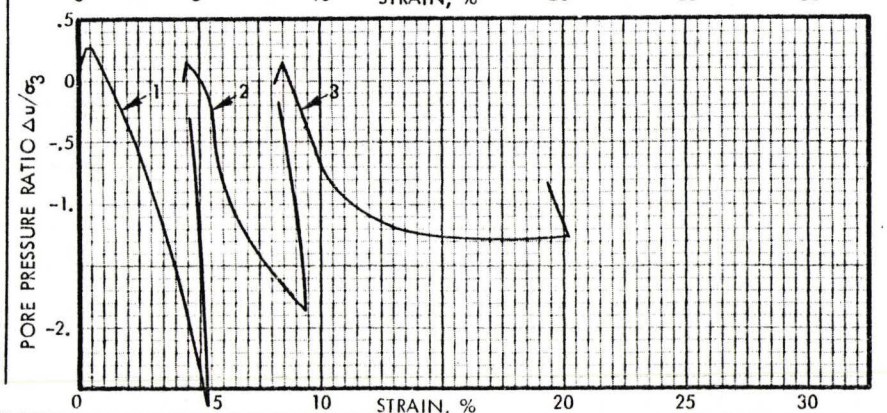
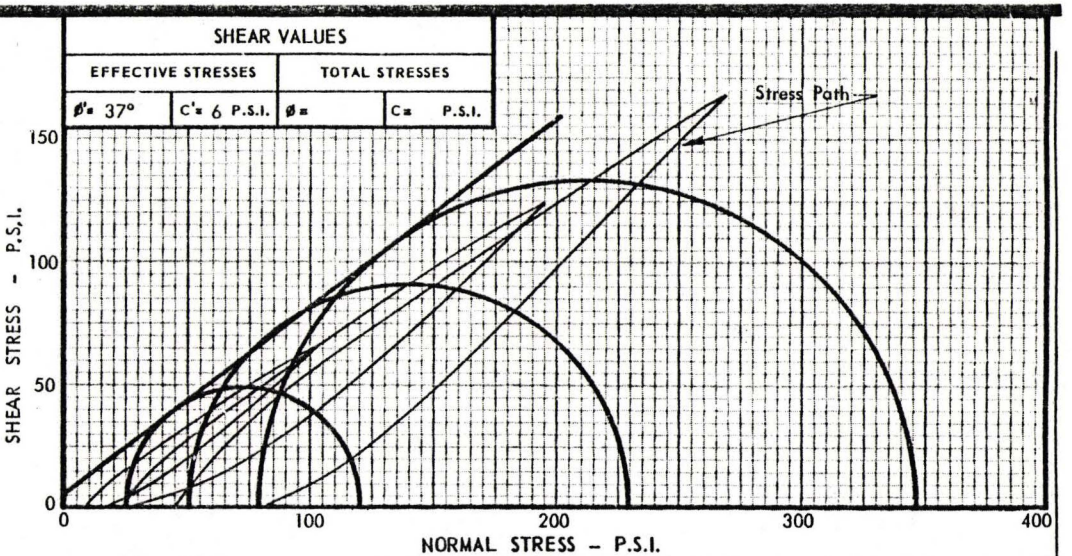
208




| SHEAR VALUES | | | |
|---|---|---|---|
| EFFECTIVE STRESSES | | TOTAL STRESSES | |
| $\phi'$= 37° | C'= 6 P.S.I. | $\phi$= | C= P.S.I. |




| SPECIMEN NUMBER | SPECIMEN LOCATION | | INITIAL SPECIMEN DATA | | | | | TYPE OF SAMPLE |
|---|---|---|---|---|---|---|---|---|
| | BORING NUMBER | DEPTH IN FEET | SOIL CLASSI-FICATION | LENGTH IN INCHES | DIAMETER IN INCHES | DRY DENSITY (P.C.F.) | MOISTURE CONTENT IN PERCENT | |
| C-3 | B5-3 | 70'-9" | SP | 4.974 | 4.42 | 108 | 20 | Undisturbed |

| SYMBOLS | SPECIMEN NUMBER | APPLIED LATERAL PRESSURE $\sigma_3$ (P.S.I.) | MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.) | TEST VALUES AT FAILURE - MAXIMUM $\sigma'_1/\sigma'_3$ | | BACK PRESSURE (P.S.I.) | TYPE OF TEST |
|---|---|---|---|---|---|---|---|
| | | | | EFFECTIVE LATERAL PRESSURE $\sigma'_3$ (P.S.I.) | MAJOR EFFECTIVE STRESS $\sigma'_1$ (P.S.I.) | | |
| 1 | C-3 | 10 | (135) | 24.9 | 120 | 50 | CU Progressive |
| 2 | C-3 | 25 | (250) | 49.4 | 230 | 50 | " " |
| 3 | C-3 | 45 | (338) | 79.2 | 348 | 50 | " " |

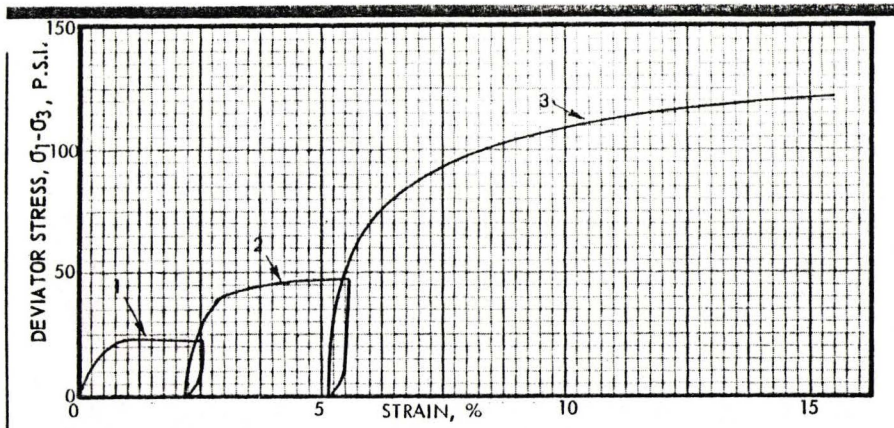
TRIAXIAL COMPRESSION TEST

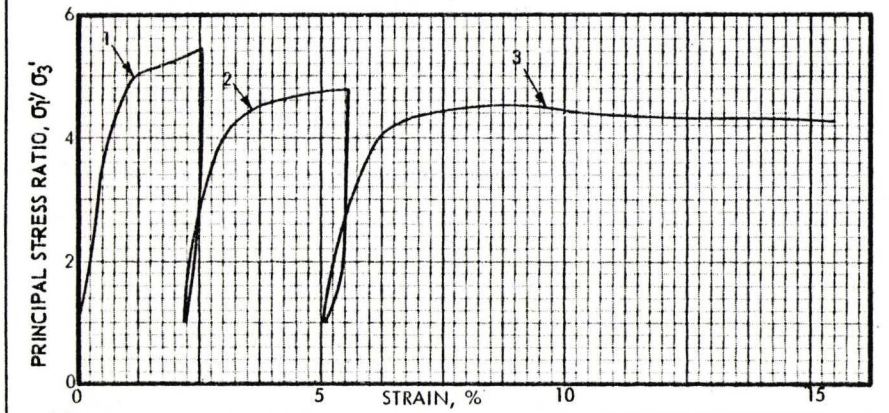
UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

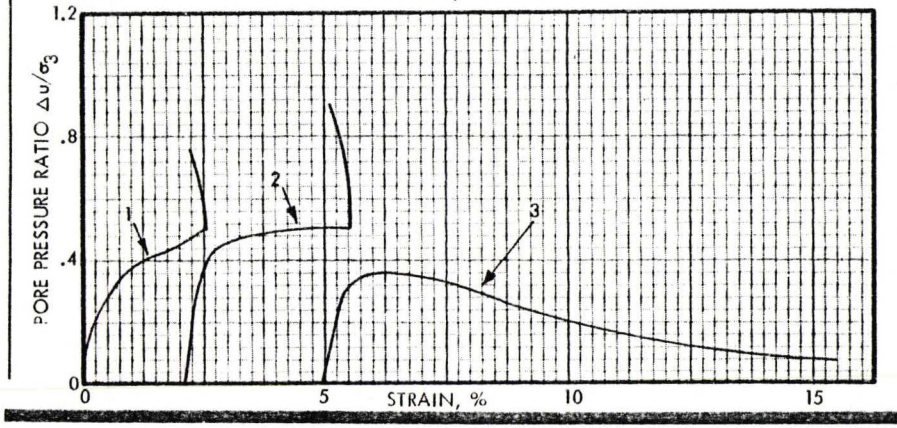
Converse Consultants — Geotechnical Engineering and Applied Sciences

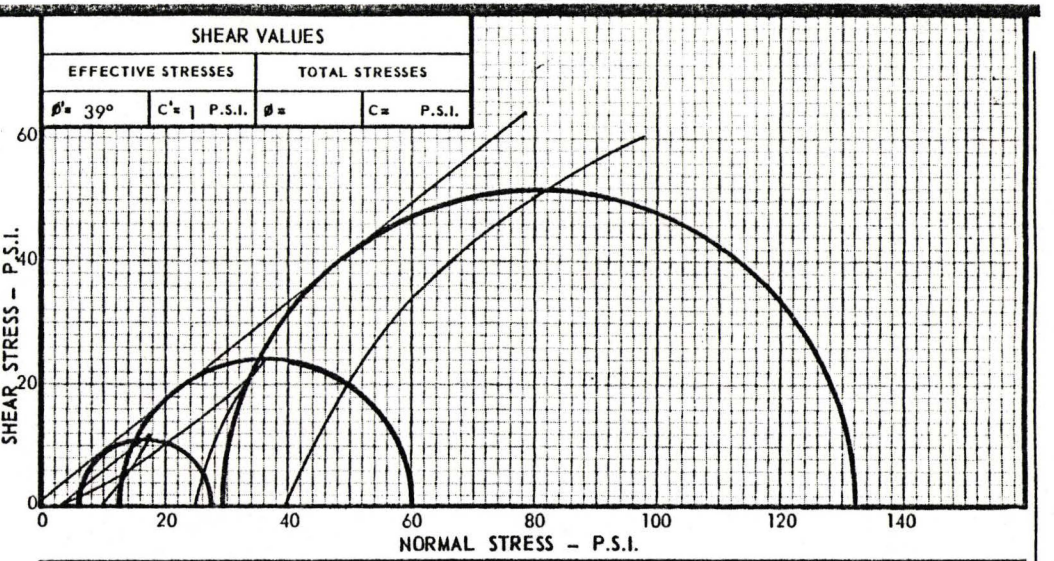
Scale: As Shown
Date: 9-23-83
Prepared by: RG
Checked by: JDH
Approved by: JAD
Project No.: 83-1101
Figure No.: F-9

607




| 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 | |
| | | | gravelly sand | 9.03 | 3.95 | 121 | 3.7 | Remolded |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| SYMBOLS | SPECIMEN NUMBER | APPLIED LATERAL PRESSURE $\sigma_3$ (P.S.I.) | MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.) | TEST VALUES AT FAILURE – MAXIMUM $\sigma_1'/\sigma_3'$ | | BACK PRESSURE (P.S.I.) | TYPE OF TEST |
|---|---|---|---|---|---|---|---|
| | | | | EFFECTIVE LATERAL PRESSURE $\sigma_3'$ (P.S.I.) | MAJOR EFFECTIVE STRESS $\sigma_1'$ (P.S.I.) | | |
| 1 | | 10 | 23.7 | 5.0 | 27.2 | 80 | CU Progressive |
| 2 | | 25 | 47.5 | 12.6 | 60.1 | 80 | " " |
| 3 | | 40 | 121.8 | 29.6 | 132.6 | 95 | " " |
| | | | | | | | |

TRIAXIAL COMPRESSION TEST

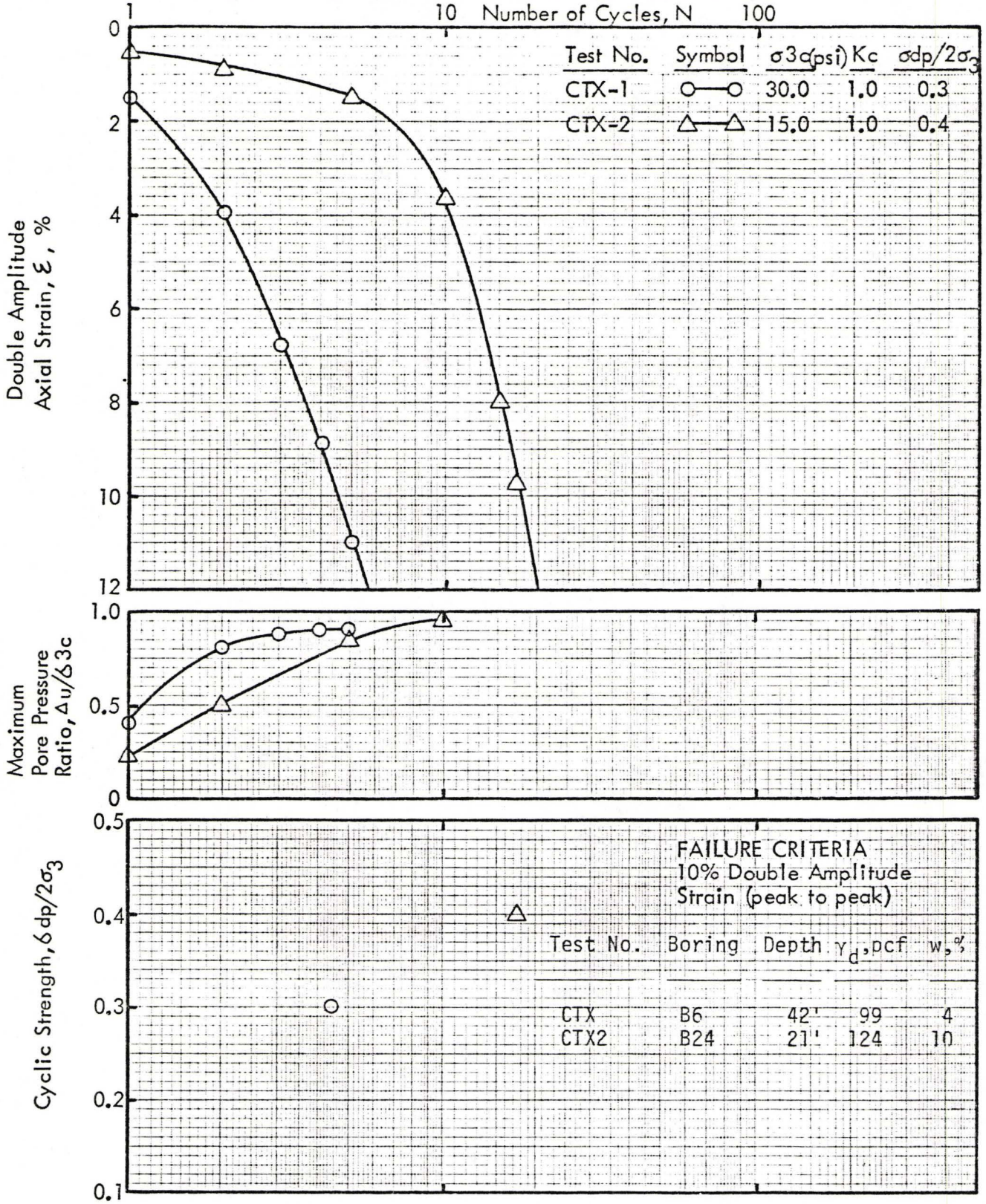
UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale As Shown
Date 9-23-83
Prepared by RG
Checked by JDH
Approved by JAD

Project No. 83-1101
Figure No. F-10

012




CYCLIC TRIAXIAL COMPRESSION TEST RESULTS

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-11

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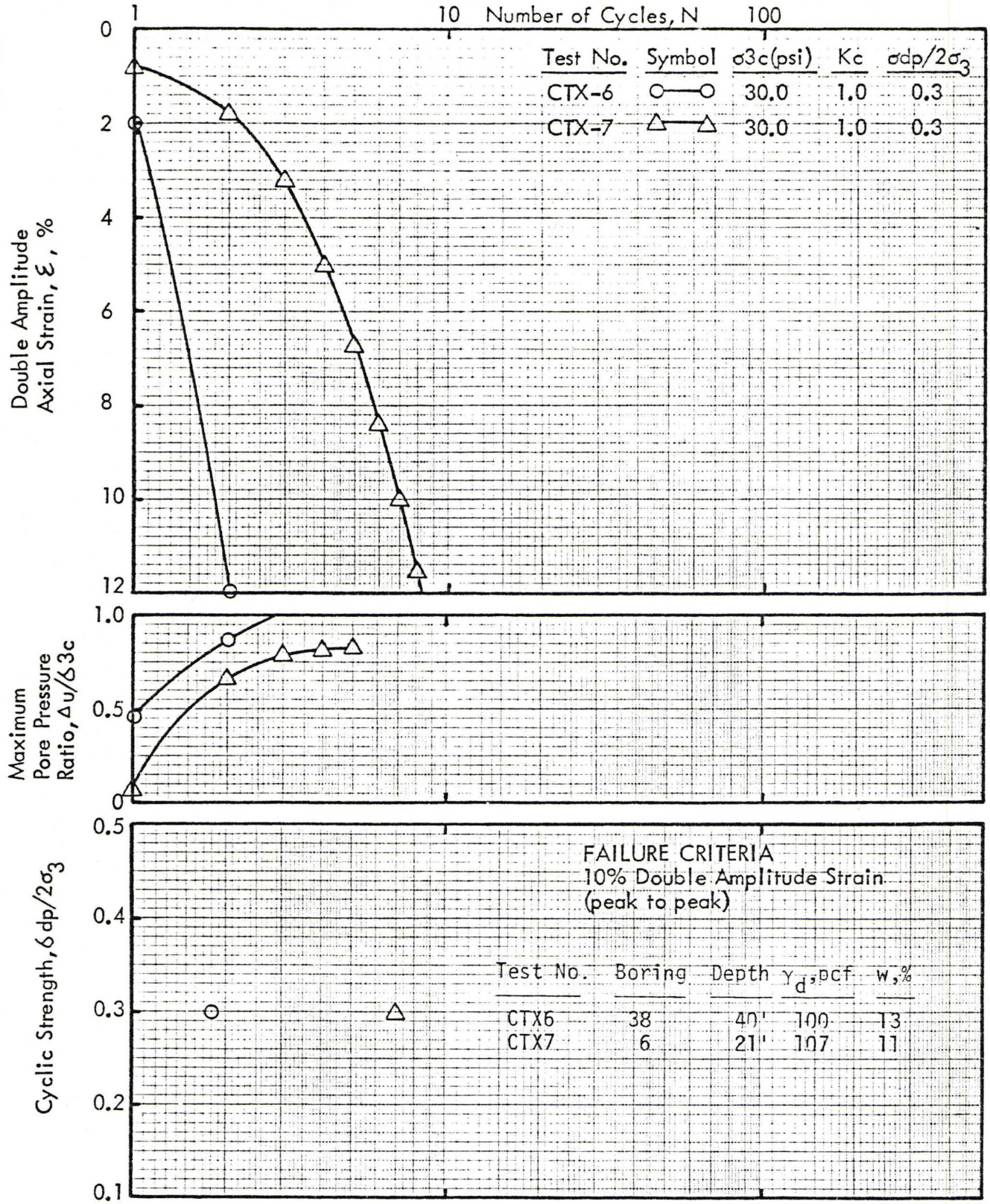
FORM NO. D5




CYCLIC TRIAXIAL COMPRESSION TEST RESULTS

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-12

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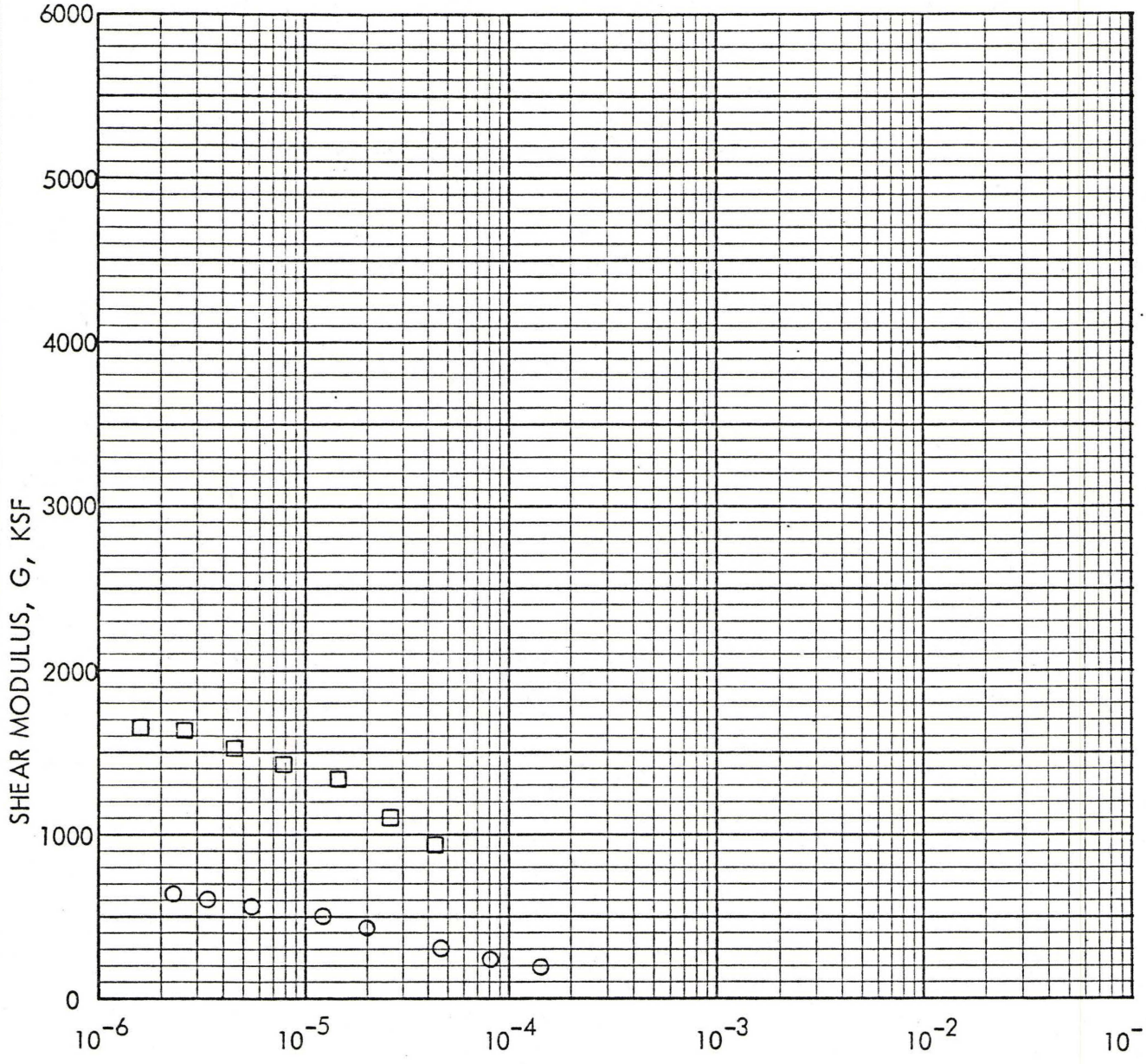
FORM NO. D55

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS




| BORING | SAMPLE | DEPTH(FT) | $\gamma_d$(PCF) | $w_o$(%) | $\bar{\sigma}_c$(PSI) | SYMBOL |
|---|---|---|---|---|---|---|
| 6 | S-3 | 95 | 94 | 27 | 15 | ○ |
| | | | | | 50 | □ |

Sample Description: Brown Clayey Siltstone, firm

RESONANT COLUMN TEST

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-13

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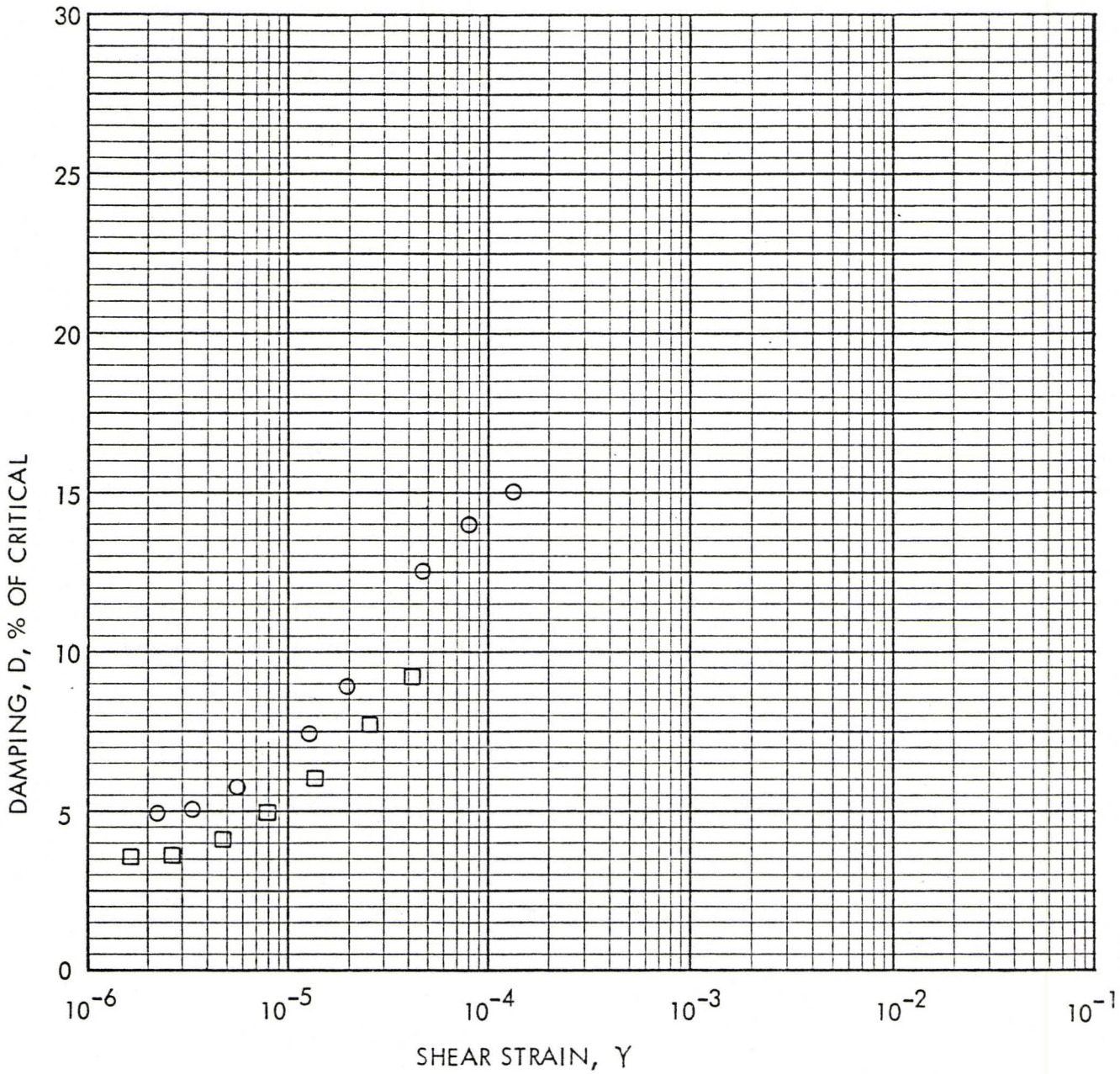
Z13

STRAIN DEPENDENT DAMPING




| BORING | SAMPLE | DEPTH(FT) | $\gamma_d$(PCF) | $w_o$(%) | $\bar{\sigma}_c$(PSI) | SYMBOL |
|---|---|---|---|---|---|---|
| 6 | S-3 | 95 | 94 | 27 | 15 | ○ |
| | | | | | 50 | □ |

Sample Description: Brown Clayey Siltstone, firm

RESONANT COLUMN TEST

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

CWDD/ESA/GRC Geotechnical Consultants

Figure No.
F-14

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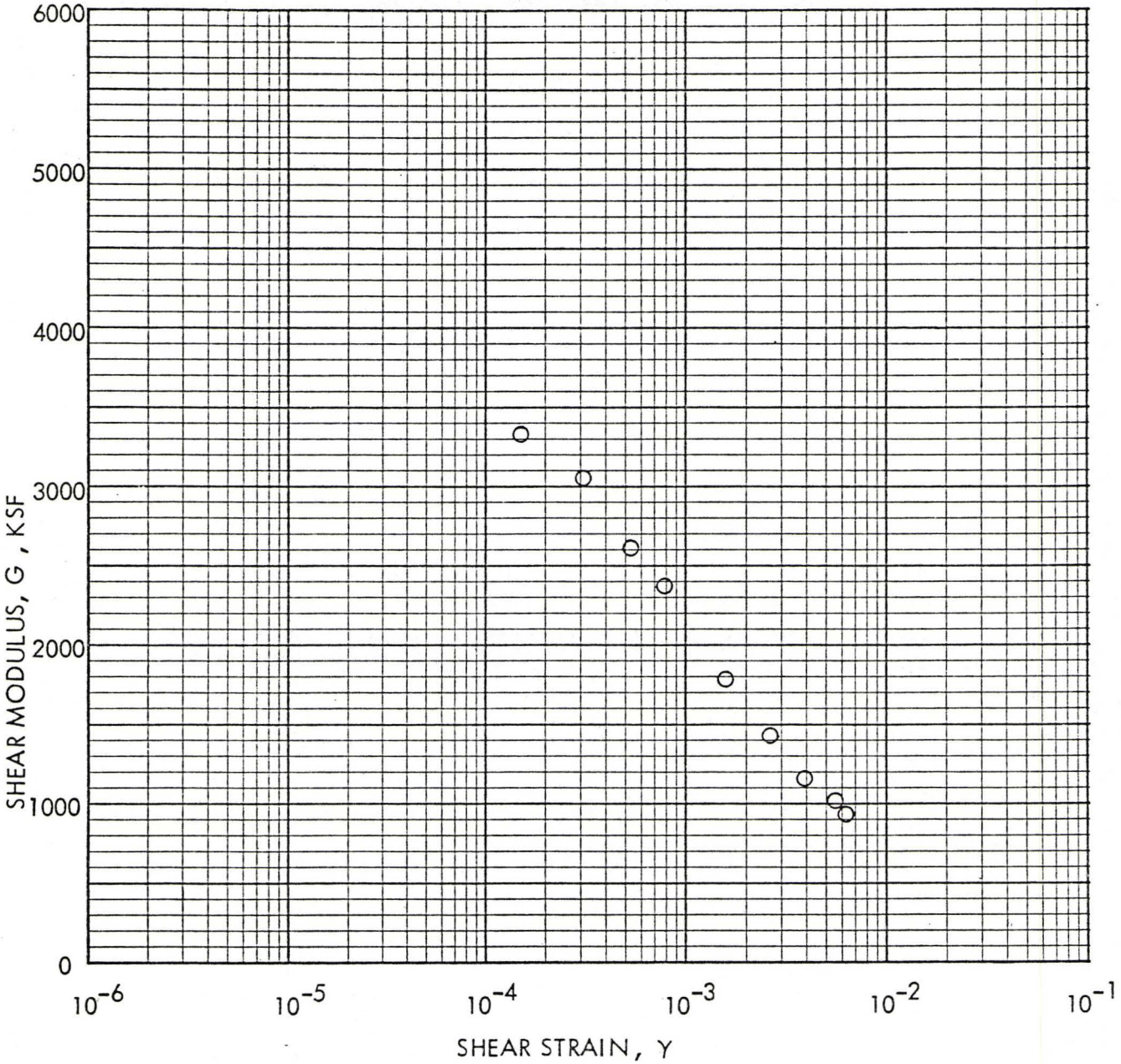
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS




| BORING | SAMPLE | DEPTH(FT) | $\gamma_d$(PCF) | $w_o$(%) | $\bar{\sigma}_c$(PSI) | SYMBOL |
|---|---|---|---|---|---|---|
| 6 | S-7 | 139 | 112 | 22 | 50 | O |

Sample Description: Gray-Brown Silty fine Sand; very dense

DYNAMIC TRIAXIAL TEST

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Figure No.
F-15

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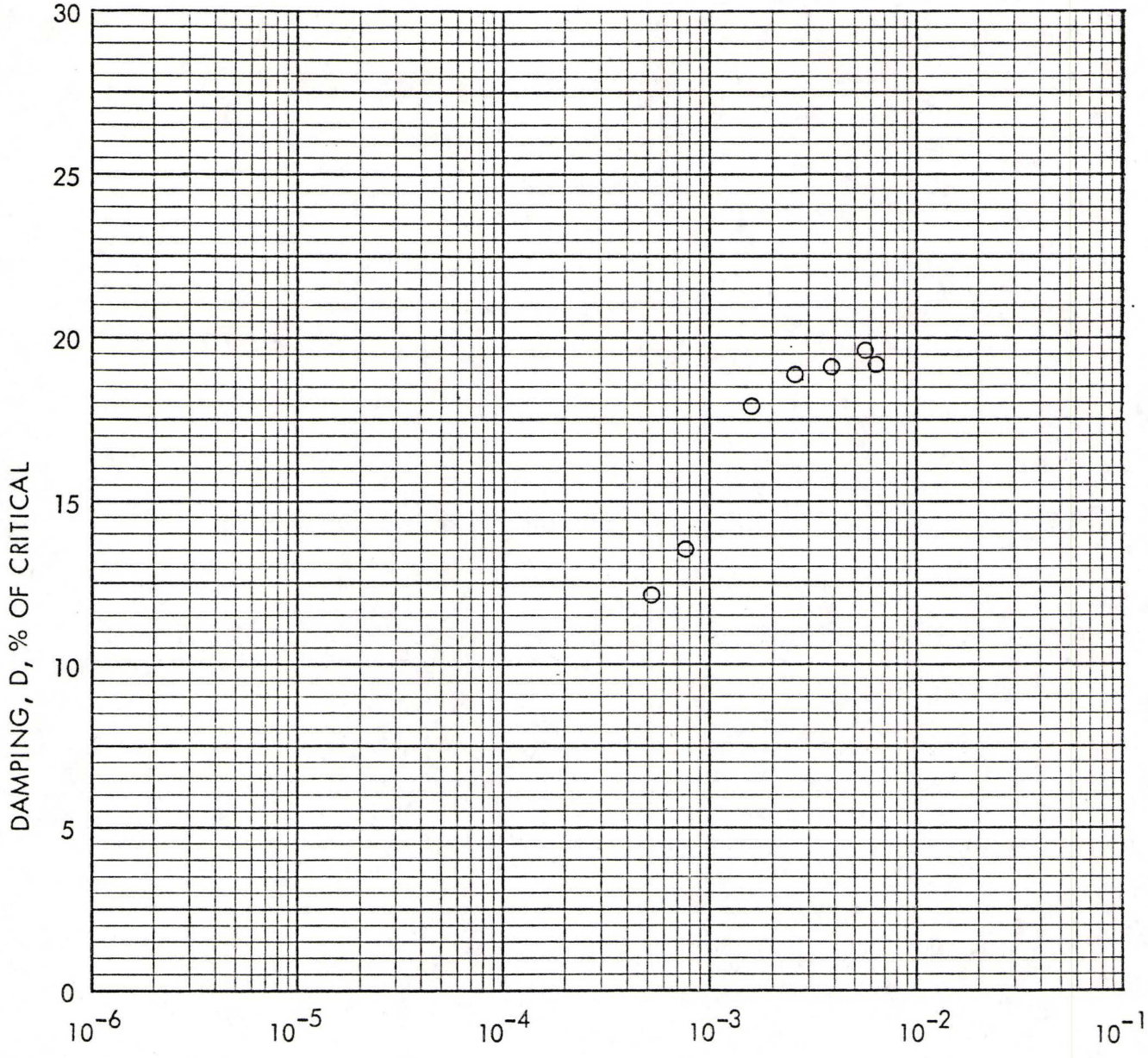
STRAIN DEPENDENT DAMPING




| BORING | SAMPLE | DEPTH(FT) | $\gamma_d$(PCF) | $w_o$(%) | $\bar{\sigma}_c$(PSI) | SYMBOL |
|---|---|---|---|---|---|---|
| 6 | S-7 | 139 | 112 | 22 | 50 | O |

Sample Description: Gray-Brown Silty fine Sand; very dense

DYNAMIC TRIAXIAL TEST

UNION STATION
Southern California Rapid Transit District
METRO RAIL PROJECT

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83-1101

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Figure No.
F-16

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# Appendix G

# Technical Considerations

APPENDIX G TECHNICAL CONSIDERATIONS

## G.1 SHORING PRACTICES IN THE LOS ANGELES AREA

### G.1.1 General

Deep excavations for building basements in the Los Angeles area are commonly supported with soldier piles with tieback anchors. Although not excavated into the type of dense sands and gravels anticipated at the Union Station site, three case studies of deep tieback excavations in the Los Angeles area are summarized below as background information. Of these, the Century City site which was underlain by dense granular alluvium is most similar to the Union Station site conditions.

### G.1.2 Atlantic Richfield Project (Nelson, 1973)

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

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

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

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

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

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

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

- The garage excavation (when 65 feet deep) survived the February 9, 1971 San Fernando earthquake (6.4 Richter magnitude) without detectable movement. The excavation is about 20 miles from the epicenter and experienced an acceleration of about 0.1g. 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.

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

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

- Basic subsurface materials were stiff clays and dense silty sands and sands. The permanent ground water table was below the level of the excavation, although minor seeps from perched ground water were encountered.

- Shoring system consisted of steel WF soldier piles placed in 36-inch diameter drilled holes spaced 6 feet on center.

- As the excavation proceeded, pneumatic concrete was placed incrementally in horizontal strips to create the finished exterior wall. The concrete which was shot against the earth acted as the lagging between soldier piles.

- Tieback anchors consisted of high-capacity 12- and 16-inch diameter friction anchors.

- Actual load imposed on the wall by the adjacent bridge was computed and added to the design wall pressures as a triangular pressure distribution.

- Maximum horizontal deflection at the top of the wall was 3 inches, while the typical deflection was less than 1 inch. Adjacent to the existing bridge, the deflections were essentially zero, with the tops of most of the soldier piles actually moving into the ground due to the high pre-stress loads in the anchors.

- Survey of the bridge pile caps indicated practically no movement.

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

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

- Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20° to 40°. Although the permanent ground water level was below the excavation level, perched zones of significant water seepage were encountered.

- Shoring system consisted of steel WF soldier piles placed in 20-inch diameter drilled holes spaced at 6 feet on center.

- Tieback anchors consisted of high-capacity friction anchors.

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

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

### G.1.5 Design Lateral Load Practices

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

According to Terzaghi and Peck's rules, the design pressure in granular soils would be equal to 0.65 times the active earth pressure. Assuming a friction angle of 37-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.

## G.2 SEISMICALLY INDUCED EARTH PRESSURES

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

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

Monobe-Okabe (1929) 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:

TABLE G-1

SHORING LOADS IN LOS ANGELES AREA

| PROJECT LOCATION | EXCAVATION DEPTH (ft) | SOIL CONDITIONS | ACTUAL DESIGN PRESSURE (P) | EQUIVALENT DESIGN PRESSURE (P') |
|---|---|---|---|---|
| Broadway Plaza Near 7th/Flower Station | 15 to 30 | Fill over Alluvium Sands | 19.0H | 15.2H |
| 500 South Hill | 25 | Fill over Sands & Gravel | 22.0H | 17.6H |
| Tishman Building Near CEG-14 | 25 | Alluvium-Clays, Sand, Silt | 19.0H | 15.2H |
| Equitable Life Near CEG-14 | 55 | Alluvium Sand/Siltstone | 20.0H | 17.5H |
| Arco Near CEG-9 | 70 to 90 | Alluvium over Claystone | 16.0H | 12.0H |
| Century City Near CEG-20 | 70 to 110 | Alluvium-Clays & Sands | 18.0H | 14.4H |
| St. Vincent's Hospital Near 3rd & Lankershim | 70 | Thin Alluvium over Puente | 15.0H | 12.0H |
| Oxford Plaza Near 7th/Flower | 40 | Fill & Alluvium over Siltstone | 21.0H | 16.8H |
| Bank Building 2nd & San Pedro | 40 | Alluvium (including Sand & Gravel over Siltstone) | 20H | 17.5H |

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

Note:

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

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

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

$$P_{AE} = 12\gamma H^2(1-k_v)K_{AE}$$

Where:

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

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

$\gamma$ = unit weight of soil
$\phi$ = angle of internal friction of soil
$i$ = angle of soil slope to horizontal
$\beta$ = angle of wall slope to vertical
$k_h$ = horizontal earthquake coefficient
$K_v$ = vertical earthquake coefficient
$\delta$ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for $K_{AE}$ then becomes,

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

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

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

Where:

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

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

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

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

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

In a saturated soil medium the change in water pressure during an earthquake has usually been established on the basis of the method of analysis originally developed by Westergaard (1933). His method of analysis was intended to apply to the hydrodynamic forces acting on the fact of a concrete dam during an earthquake. However, it was used by Matsuo and O'Hara (1960) to determine the dynamic water pressure (due to the pore fluid within the soil) acting on quay walls during earthquakes, and has been used by various other engineers for evaluating dynamic water pressures acting on retaining walls backfilled with saturated soil. Unless the soil is extremely porous, it is difficult to

visualize that the pore water can actually move in and out quick enough for it to act independently of the surrounding soil media. For most natural soils, the soil and pore water would move together in phase during the duration of the earthquake such that the dynamic pressure on the wall would be due to the combined effect of the soil and water. Thus, the total weight of the saturated soil should be used in calculating dynamic earth pressure values.

The recommended permanent wall uniform earthpressure (8H) presented in Figure 6-6 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 exceedance 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 increases 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 of about 0.23g (using the recommended procedures). Data from Part I Seismological Investigation indicates the 0.23g peak acceleration to have a probability of exceedance less than 5% during an average two-year period (a reasonable construction period). The average recurrence of this ground motion level was indicated to be about 100 to 150 years. Based on consideration of the above, the 6H uniform seismic pressure was recommended for design of the temporary wall (see Figures 6-2).

## G.3 LIQUEFACTION ANALYSES

### G.3.1 Introduction

The procedures used in this study to evaluate liquefaction potential are based mainly on field observations of the performance of soils during previous earthquakes. The field observations made at the Union Station site during this and the previous geotechnical investigation (1981 Geotechnical Investigation Report) that were used to establish the liquefaction potential of the various soils include:

- Standard Penetration Test (SPT) resistance
- Shear wave velocity measurements
- Observed behavior of soil in the large diameter borehole.

In addition to the field observations listed above, gradations of the soils obtained from the field were compared with gradations of materials which have liquefied during past earthquakes and which are considered most susceptible to liquefaction in laboratory tests.

Each of the field observations (and comparisons) is described in the following text. It should be noted that the observations which have been made in the field only provide a basis upon which to judge the liquefaction potential of the various soils. Our conclusions regarding the liquefaction potential of the soils are generally supported by these observations. However, our conclusions are also based on engineering judgement.

G.3.2 Standard Penetration Resistance

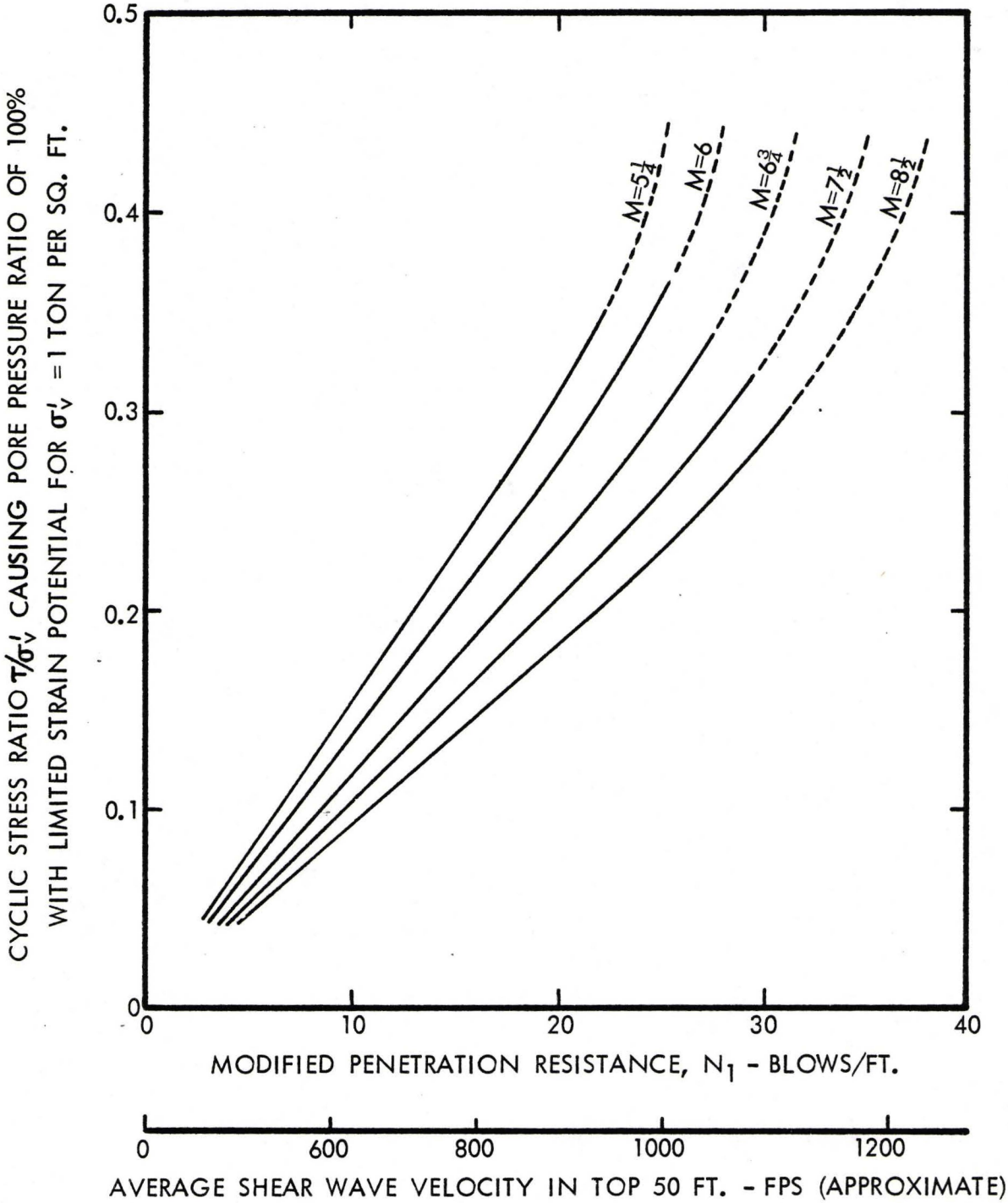
The use of the Standard Penetration Test (SPT) in estimating the liquefaction potential of saturated cohesionless soil deposits has been the topic of many previous investigations. Results of these investigations have recently been summarized by Seed et al (1983). Basically, the method utilizes empirical relationships which have been developed from a comprehensive collection of SPT blow count data obtained from sites where evidence of liquefaction or no liquefaction was known to have taken place during past earthquakes. Empirical relationships that have been recently proposed by Seed et al (1983) are shown in Figure G-1.

While results of the Standard Penetration Test have been generally accepted as a good index upon which to estimate the liquefaction potential of saturated sand deposits, it should be noted that the SPT results cannot be utilized to evaluate the liquefaction potential of soils containing gravels, cobbles or boulders. Since much of the Young Alluvium which underlay the Union Station contains gravel and cobbles, the SPT blow counts recorded in the soils cannot be specifically relied upon. However, for those soils which did not include significant percentages of gravel-sized particles (namely the finer gravelly sand units within the main gravelly sand unit and the lower sands), SPT blow count data were utilized along with the relationships shown in Figure G-1. In general, the SPT blow count measurements taken in the finer-grained Young Alluvium are greater than 100 blows per foot indicating that these soils are generally dense to very dense. These blow counts along with the relationship shown in Figure G-1 suggest that liquefaction of the soil deposits during strong earthquake ground shaking would be highly unlikely.

G.3.3 Shear Wave Velocity Measurements

Cross-hole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project tunnel alignment were performed as part of the initial 1981 geotechnical investigation. One of the cross-hole surveys was performed within 1,000 feet of the center of the proposed Union Station site in Boring CEG-5. Shear wave velocities measured in the Young Alluvium (approximately the upper 85 feet of the borehole) range between 950±50 fps to 1,400±90 fps. Most of the shear wave velocities measured, however, were about 1,100 fps.

While shear wave velocity in the past has not been as widely accepted as SPT blow count data for estimating the liquefaction potential of a soil deposit, it has received recent attention (Seed et al, 1983). Figure G-1 suggests that liquefaction will never occur during any earthquake if the shear wave velocity in the upper 50 feet of soil exceeds 1,200 fps. Since the shear wave velocities measured close to the Union Station site are approximately 1,100 fps, this is an indication that liquefaction at the site would probably be unlikely.




(after Seed, 1983)

**CORRELATION BETWEEN PENETRATION RESISTANCE AND FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS**

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Figure No.
G-1

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G.3.4 Observed Soil Behavior

Observations made during the field exploration program may also provide information which may be useful in determining whether or not the saturated cohesionless soils found at the site are susceptible to liquefaction. The logging of the large diameter boring 6A, which was drilled approximately 550 feet west of boring 5-1 allowed the following observations to be made:

- The soils between the ground surface and a depth of 12 feet belled and caved during logging.
- Between depth of 15 to 22 feet the soils only experienced slight caving.
- Between depth of 22 to 55 feet the hole did not cave.

These observations are somewhat remarkable in that the soils in this borehole were generally granular and seepage was noted in the hole at a depth of about 18 feet.

If the soils which make up the Young Alluvium were in fact loose (and therefore more susceptible to liquefaction) one would have expected excessive caving of the walls of the borehole. Instead, the wall stood unsupported (even with groundwater seepage) suggesting that the soils are at least medium dense and compact. It is our judgement that the behavior of the soils in this borehole are generally indicative of a soil that would not have a tendency to liquefy during strong earthquake ground shaking.

G.3.5 Gradational Characteristics

Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compliation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figure G-2. The ranges shown in this figure have been complied by Lee and Fitton (1968), Seed and Idriss (1967), Kishida (1969), and Youd (1982) and appear to indicate that the soils types most susceptible to liquefaction consist of primarily poorly grade silty sands and sandy silts. It is important to note that all the gradation ranges shown in Figure G-2 have less than 10 percent by weight clay size particles (i.e., particles less than 0.002 mm) suggesting that clayey (cohesive) soils have a low liquefaction potential. Gradation characteristics typical of gravels and gravelly soils are also absent from Figure G-2 suggesting, in part, that these types of soils may not be capable of developing high excess pore pressure either because they are capable of draining rapidly during the cyclic loading or because these types of materials are usually more efficiently packed (i.e., denser) in situ than soils that consist of uniformily-sized particles. While the liquefaction potential of a soil is dependent on many factors other than gradation (such as the relative density of the soil, the intensity and duration of cyclic loading, among others), comparisons of the gradation characteristics of a soil with those ranges shown in Figure G-2 provides a useful guide in establishing the liquefaction potential of a soil.

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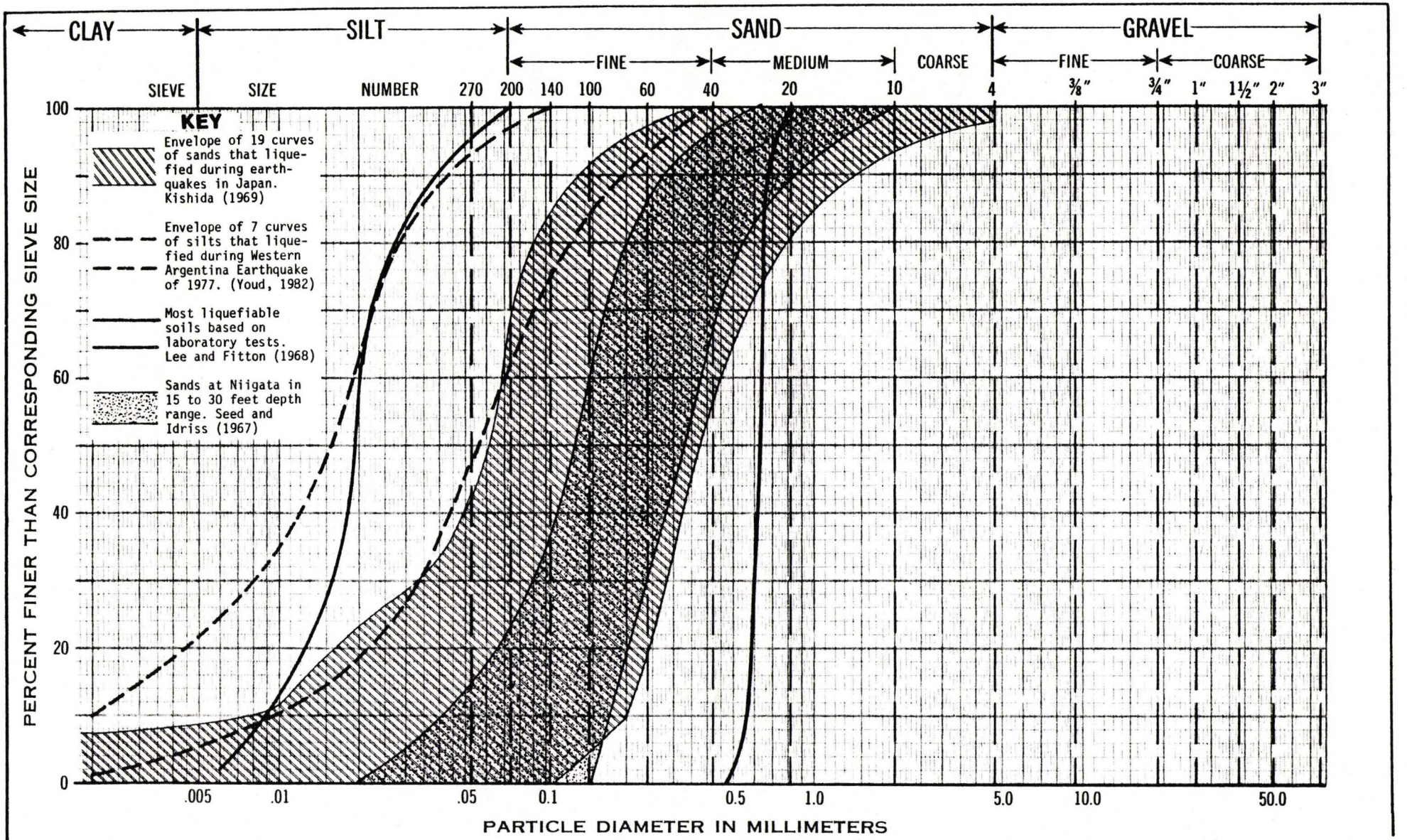


# GRADATIONS OF SOILS CONSIDERED SUSCEPTIBLE TO LIQUEFACTION

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Project No.
83-1101

Figure No.
G-2

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The gradation characteristics of the various soils which comprise the Young Alluvium were compiled from laboratory tests performed during this and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" sandy soils shown in Figure G-2 are presented in Figure G-3.

This figure indicated that five of the 11 samples tested fall within the range of gradations of soils considered "susceptible" to liquefaction. However, it should be noted that three of these samples were obtained near the "lower" sand unit which had high SPT blow counts. The remaining two samples were obtained from samples taken from the gravelly sands. The comparisons shown in Figure G-3 indicates that, on the basis of gradation alone, there appears to be some soils present at the site which may be considered liquefiable.

G.3.6 Conclusions

Based on the above considerations and comparisons, it is our judgement that the Young Alluvium soil deposits found at the Union Station site would not be subject to liquefaction during strong ground shaking produced at the site by the postulated earthquake motions.

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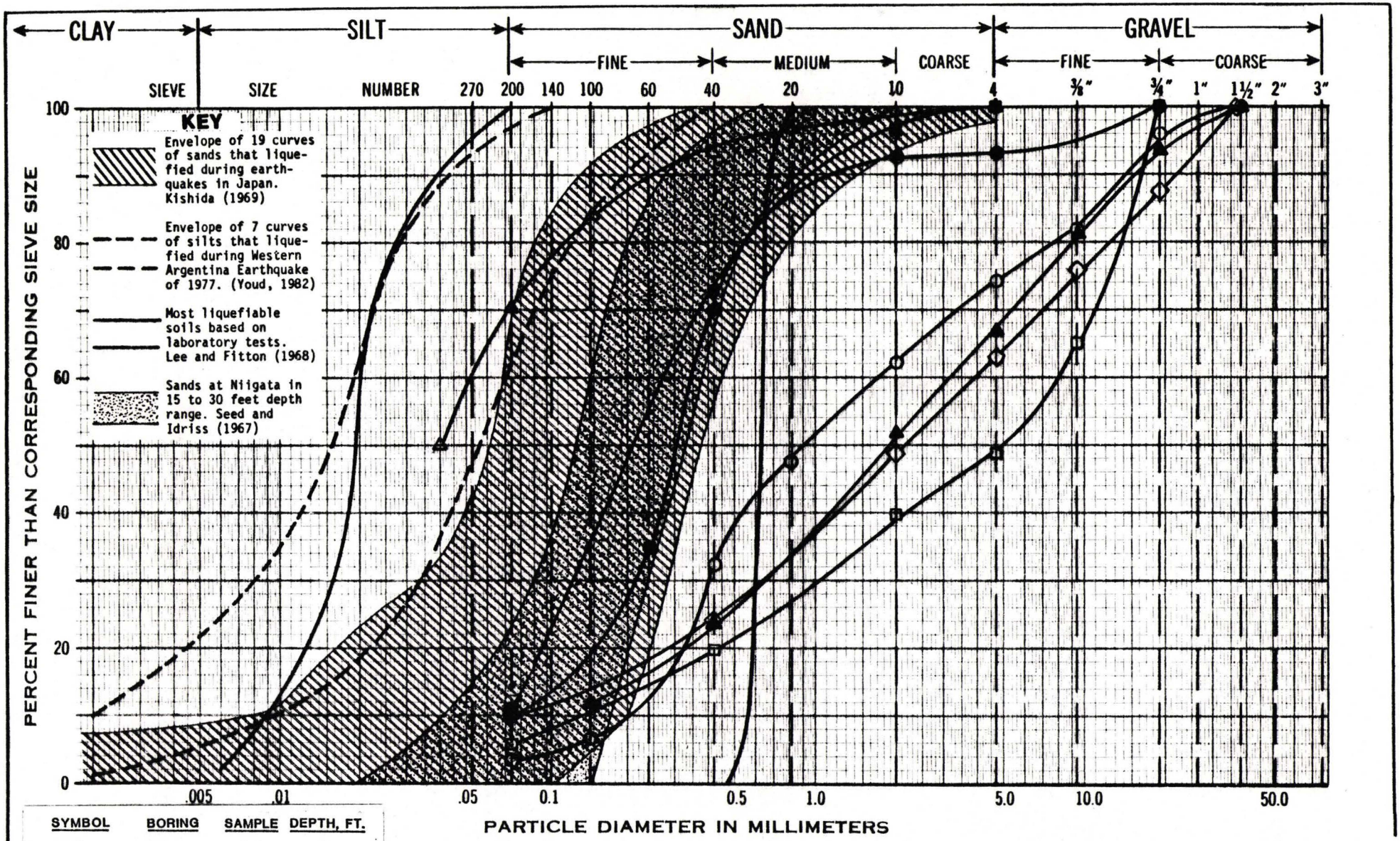


| SYMBOL | BORING | SAMPLE | DEPTH, FT. |
|---|---|---|---|
| ○ | CEG 6 | S-1 | 21 |
| ● | CEG 6 | S-2 | 42 |
| □ | 5-2 | C-2 | 21 |
| | 5-2 | C-3 | 46 |
| ■ | 5-2 | C-4 | 70 |
| | 5-5 | C-5 | 60 |
| △ | 5-3 | C-2 | 51 |
| ▲ | 5-4 | C-3 | 21 |
| ◇ | 5-5 | C-3 | 26 |
| | 5-5 | C-4 | 36 |

# COMPARISONS OF GRADATIONS

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G-3

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# Appendix H

# Earthwork Recommendations

APPENDIX H - 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 6.5 respectively.

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

° Minor Construction Excavations: Temporary dry excavations for foundations or utilities may be made vertically to depths up to 5 feet. For deeper dry excavations in existing fill or natural materials up to 15 feet, excavations should be sloped no steeper than 1:1 (horizontal to vertical) or shored. Recommendations for major sloped excavations are presented in Section 6.5.

° Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed prior to backfilling the walls. Onsite soils or imported granular soils should be placed and compacted in accordance with "Recommended Specifications for Fill Compaction" in this appendix. 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.

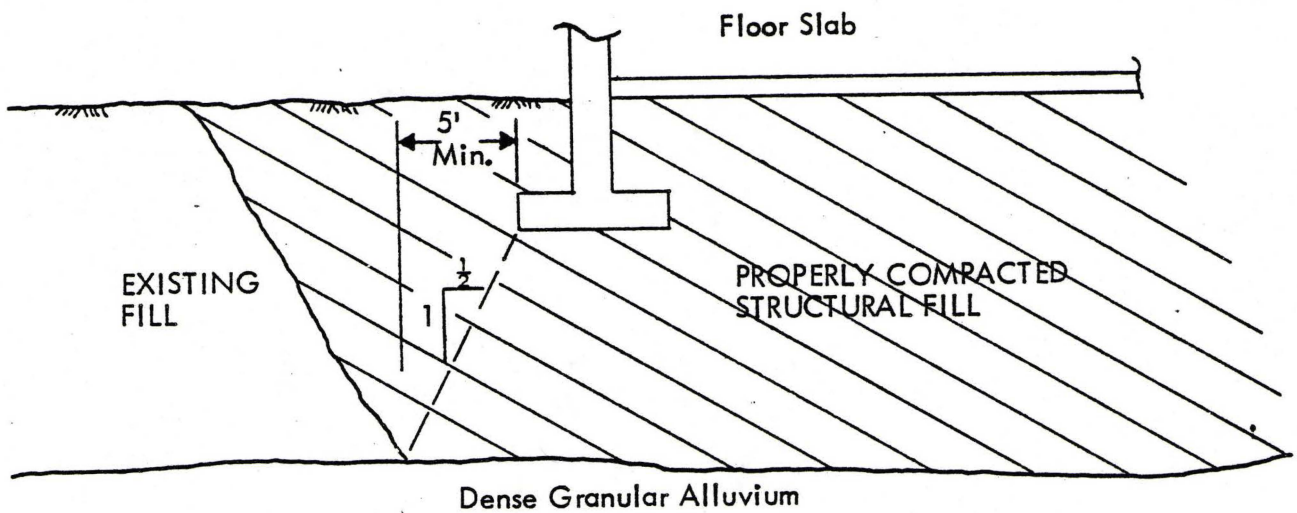
  Oversized fill consisting of cobbles and boulders greater than 6 inches but less than 3 feet may be placed in the deeper portions of structural fill at depths greater than 8 feet below rough grade and at least 5 feet (horizontally and vertically) away from existing or proposed structures. All oversized fill materials should be properly windrowed to reduce the potential for voids in the fill after placement. All oversized fill should be placed and compacted in accordance with "Recommended Specifications for Placement of Oversized Material".

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

and densification procedures to render an opinion as to the adequate densification of the pea gravel.

If granular backfill or pea gravel is placed in an area of surface drainage, the backfill should be capped with at least 18 inches of relatively impervious type soil; i.e., 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 should be excavated and replaced with structural fill. If existing fill soils are encountered in near surface subgrade areas, these materials should be excavated and replaced with properly compacted 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 should be placed and compacted in accordance with "Structural Fill and Backfill".

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

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

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

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

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

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

° Recommended Specifications for Placement of Oversized Material: The following specifications are recommended to provide a basis for quality control during placement of oversized material during grading:

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

2. The exposed natural soil surface shall be scarified and compacted to at least the specified density, with adjustment of the moisture content when necessary prior to placement of fill.

3. Non-organic oversized material such as cobbles and boulders shall be placed in windrows in the deep fill areas such that not oversized material lies within 8 feet of the finish grade or 5 feet of existing or proposed structures. The maximum dimension of oversized material shall not exceed 3 feet. Oversized debris shall be placed in windrows approximately 3 feet wide and 2 to 3 feet in height. Windrows should be spaced horizontally at least 10 feet apart.

4. Approved onsite or imported soils shall be placed and compacted in lifts between and around the windrowed material in accordance with applicable specifications for soil fill. The windrows shall be track-rolled with a CAT-D8H dozer or similar equipment and flooded repeatedly with water to the satisfaction of the soils engineer as the soil fill is placed and compacted around the windrows.

5. Soil fill placement and compaction shall continue to a level at least 2 feet above the windrowed material prior to placing additional oversized material. Subsequent windrows shall be staggered between the underlying windrows such that new windrows do not overlie the previous.

6. The soils engineer shall observe the placement, flooding and track-rolling of the windrowed material as well as the placement and compaction of the soil fill around the windrowed material. Where inadequate flooding, track-rolling or compactive effort is indicated, additional effort shall be applied as necessary.