# SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
# METRO RAIL PROJECT

# GEOTECHNICAL INVESTIGATION REPORT VOLUME I

PREPARED BY


Converse Ward Davis Dixon
Earth Sciences Associates
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**General Geotechnical Consultant**


**November, 1981**

RTD

**ConverseWardDavisDixon**
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December 21, 1981

WBS 12AAC
Audit No. 2256

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

Attention: Mr. Richard Gallagher
Manager/Chief Engineer

Gentlemen:

This letter transmits our final report of the geotechnical investigation for the Metro Rail Project, prepared in accordance with the Agreement dated November 3, 1980, between Converse Ward Davis Dixon and the Southern California Rapid Transit District. The findings and conclusions developed during this investigation are presented in two separate volumes:

Volume I: Report - interpretive information, data analyses, conclusions and recommendations

Volume II: Appendices - supportive information for Volume I and results of field/laboratory programs.

We appreciate the assistance and guidance provided by Mr. James E. Crawley, Deputy Chief Engineer, and his staff, and the District's Board of Special Geotechnical Consultants. We also want to acknowledge the dedicated efforts of each member of the Converse Team, particularly Howard A. Spellman, Project Manager.

Respectfully submitted,

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SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT

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# Section 1.0
# Executive Summary

# Section 1.0: Executive Summary

## 1.1 GENERAL

The results of the geotechnical investigation indicate that the proposed alignment for the 18-mile Metro Rail Project is favorable for modern economical tunnel construction. Sixty percent of the planned route traverses competent soil materials (Old Alluvium) and soft-rock formations most suitable for excavation by tunnel boring machines (TBMs). Approximately 15 to 20 percent of the route is expected to encounter hard-rock formations of the Santa Monica Mountains which may be excavated either by conventional drilling/blasting methods or by hard-rock TBMs. The remaining 20 to 25 percent of the alignment will penetrate soil materials which may require special excavation procedures using TBMs or excavation by cut-and cover methods.

The findings and conclusions developed during this geotechnical study are presented in two separate volumes: Volume I, "Geotechnical Investigation Report", presents interpretive information, data analyses, conclusions and recommendations. Volume II, "Appendices", contains the detailed results of field and laboratory programs, and provides information that supports the conclusions and recommendations presented in Volume I.

## 1.2 INTRODUCTION

The proposed 18-mile Metro Rail Project route extends from Union Station in downtown Los Angeles to Lankershim Boulevard in North Hollywood. The prime purpose of this investigation and report is to provide comprehensive information on subsurface soil, bedrock and ground water conditions along the alignment. This information will be used by engineers in preparing designs and as an aid to potential construction contractors. This report addresses the suitability of geologic formations as they will affect station excavations, as well as shallow and deep tunnels in various formations.

Project design requirements and structural features have not been established. The tunnel could be up to 35 feet in diameter if two lines are contained in one tunnel. Seventeen stations are planned at about one mile intervals, with stations up to 800 feet long. Generally, stations and tunnels will extend 30 to 200 feet below the existing ground surface; the tunnel under the Santa Monica Mountains will exceed 200 feet in depth.

## 1.3 DESCRIPTION OF EXPLORATION AND TESTING PROGRAM

Field exploration consisted of subsurface drilling, sampling and testing, combined with geophysical testing along the proposed alignment. Simultaneously with and following the field investigation, an extensive laboratory program, consisting of over 1,100 individual tests, was conducted. The purpose of the field and laboratory investigation was to develop a comprehensive understanding of the subsurface conditions at selected locations along the proposed route, and to establish the physical properties of the various soil, rock, water and gas samples obtained during this investigation. The following field and laboratory work tasks were accomplished:

- drilled, sampled and cored 41 test borings, ranging from 100 to 400 feet in depth;
- obtained, identified and classified numerous soil and bedrock samples, and measured the presence of in situ gases;
- monitored in situ water pressure tests to determine the permeability characteristics of the bedrock materials;
- installed permanent ground water monitoring devices and measured water levels in test borings;
- electrically logged all 41 core holes to provide geophysical information;
- performed three miles of surface seismic refraction surveys to assist in locating faults and depth of alluvium;
- conducted downhole geophysical logging at 17 proposed station sites to develop vertical profiles of dynamic properties for soft-ground materials;
- obtained 220 surface micro-gravity readings at four locations to more accurately define major fault locations;
- conducted crosshole geophysical logging at 10 selected station sites to develop horizontal profiles of dynamic properties for soft-ground materials;
- tested samples in the laboratory to determine the static and dynamic engineering properties of representative soil and rock materials;
- tested gas, oil and water samples in the laboratory to determine their physical characteristics;
- examined the petrographic features of the basalt and hard sandstone bedrock samples for detailed classification and identification.

## 1.4 PROJECT GEOLOGY

The tunnel alignment will traverse three physiographic features: the Los Angeles Basin, the Santa Monica Mountains, and the San Fernando Valley. The Santa Monica Mountains are comprised mainly of hard rock of the Topanga Formation, consisting of basalt, sandstone and conglomerate. The Los Angeles Basin and San Fernando Valley contain Young and Old alluvial deposits underlain by the Fernando, Puente and San Pedro formations.

For the purpose of this report, similar hard rock and soft-ground formations along the alignment have been grouped into units. These units have engineering properties which are expected to exhibit similar behavior during excavation. The soft-ground category includes Old alluvial deposits, and soft rock such as siltstone and sandstone of the Fernando, Puente and San Pedro formations. Hard rock formations consist of the basalt, sandstone and conglomerate of the Topanga Formation. A summary of the geologic features encountered during this investigation are presented on the "Geologic Profile" in Drawing 2 at the end of this volume.

## 1.5 GEOLOGIC FEATURES OF ENGINEERING SIGNIFICANCE

Based on our evaluation of the subsurface conditions and engineering properties along the proposed alignment, we have divided the tunnel route into "reaches" having geologic similarities as follows:

° Reach 1 - East Portal to Hollywood Freeway
Deep, loose to dense stream-deposited Young Alluvium (soil) of the modern Los Angeles River channel was encountered with soft-rock siltstone and sandstone near the surface at each end of this reach. The permanent ground water level was found at a depth of about 25 feet. Oil was observed in the eastern portion, and a gassy to potentially gassy ground classification is recommended.

° Reach 2 - Hollywood Freeway to Harbor Freeway
Deep, loose to dense Young Alluvium of the ancestral Los Angeles River channel and soft-rock formations were encountered. A relatively deep (below 90 feet) permanent ground water level was measured. Minor amounts of oil were detected, resulting in a classification of potentially gassy ground.

° Reach 3 - Harbor Freeway to Normandie Avenue
Shallow, dense Old Alluvium was found over soft rock. There was an upper perched water table in the Old Alluvium, and a deeper permanent ground water level in the rock. No oil was encountered, but this is potentially gassy to gassy ground.

° Reach 4 - Normandie Avenue to La Brea Avenue
Shallow to deep, dense Old Alluvium overlies dense saturated sand of the San Pedro Formation which is underlain by soft rock. A shallow perched water table was measured in the upper alluvium along with a deeper (below 130 feet) permanent ground water level in the soft rock. Oil was not found, but this is potentially gassy ground.

° Reach 5 - La Brea Avenue to Melrose Avenue
Deep, dense Old Alluvium overlies dense, oil-saturated sand (La Brea Tar Pits area), which is underlain by soft rock of the Fernando Formation. Shallow perched water was found in the alluvium, with a deep, permanent ground water level in the soft rock. Oil and gassy ground was found throughout the reach. Three inactive faults are located in the Salt Lake oil field, and the potentially active Santa Monica fault zone will be crossed near Melrose Avenue.

° Reach 6 - Melrose Avenue to Yucca Street
Deep, dense Young and Old Alluvium overlies soft rock to depths greater than 200 feet. The permanent water table was not encountered in the test borings. Shallow water levels were contained in the upper alluvium, and neither oil nor gas was encountered. This reach is bounded by the Santa Monica fault zone on the south, and the Hollywood fault on the north at the base of the Santa Monica Mountains.

° Reach 7 - Yucca Street to Universal City Station
This entire reach within the Santa Monica Mountains consists of a thin layer of weathered bedrock over hard rock. The sedimentary rocks on both sides of the mountain are not as hard those in the central portion. Neither oil nor gas was encountered. The tunnel alignment will pass through several faults that may contribute to ground water inflow to the tunnel.

° Reach 8 - Universal City Station to North Portal
Deep, loose to dense Young and Old Alluvium of the modern Los Angeles River channel was underlain by soft rock at depths greater than the test borings (200 feet). A deep permanent ground water level (below 110 feet) and nongassy ground conditions are anticipated.

The tunnel alignment will pass near or penetrate numerous geologic faults, all but two of which are judged to be inactive. The Hollywood fault (vicinity of Yucca Street) is judged to be active, and the Santa Monica fault (vicinity of Melrose Avenue) is thought to be potentially active. The project is 30 miles from the nearest point of the active San Andreas fault. Although a great earthquake is expected to occur on the San Andreas during the project life, its impact may not be as significant as a future event on a local fault. This subject will be addressed in detail in a separate seismology study.

## 1.6 PREVIOUS TUNNELING EXPERIENCE IN AREA

More than 60 tunnels, with a total accumulated length of over 50 miles, have been bored within the Los Angeles city limits. This history of local tunneling experience was reviewed, with particular attention given to six case histories that involved similar geologic formations or subsurface conditions. Emphasis was placed on tunnels excavated by TBM or where gas and/or oil conditions were encountered. Case histories included:

° Metropolitan Water District (MWD) San Fernando Tunnel
° MWD Newhall Tunnel
° City of Los Angeles-La Cienega-San Fernando Valley Relief Sewer Tunnel
° MWD Hollywood Tunnel
° Los Angeles County Flood Control District (LACFCD) Sacatella Tunnel
° MWD Tonner Tunnel.

Geologic conditions, overall excavation progress and construction methods were summarized for each case history. Similar geologic and excavation conditions were noted and then compared to conditions expected to be encountered along the proposed alignment. These case histories indicate rapid and economical progress can be made by tunnel boring machines (TBM) on the Metro Rail alignment. For example, excavation experience in Old Alluvium at the San Fernando Tunnel resulted in record advances using a TBM with a digging spade. A total of 3,500 feet of tunnel was excavated in one month, and 277 feet during one three-shift day. When ground water was encountered, advance rates reduced to about 60 feet per three-shift day.

Specific tunnel excavation problems experienced on these previous projects are discussed in detail. In most instances, a majority of these problems (gas/oil, ground water, caving, slow progess) were anticipated or could have been anticipated by the tunneling contractor prior to construction.

## 1.7 ANTICIPATED GROUND BEHAVIOR IN UNDERGROUND CONSTRUCTION

Underground construction refers to tunnels and mined stations excavated below ground with minimal disruption to surface facilities. Based on this geotechnical investigation, favorable excavation conditions are anticipated for most of the proposed route for both tunnels and mined stations. For soft-ground materials, the most economical and practical means of advancing the tunnel is by a fully shielded TBM, utilizing precast concrete elements to form both the initial support and permanent lining placed in one construction operation.

A soft-ground TBM will be favorable for tunnels that pass through Old Alluvium and soft rock. This soft-ground condition will include most of the alignment, with the exceptions of: 1) hard rock (Reach 7) through the Santa Monica Mountains, and 2) the loose, wet alluvium at the Los Angeles River crossings. A hard-rock TBM or excavation by drilling and blasting methods will probably be required through the Santa Monica Mountains. In addition, such a hard-rock TBM must be capable of coping with rock that ranges locally from soft to hard (mixed face condition) and varying ground water inflows.

Design selection of deep (below 100 to 150 feet) tunnel profiles will require consideration of local subsurface conditions to determine the need for constructing some stations by mining methods. It is feasible to mine stations in the Old Alluvium and soft rock, as well as in the hard rock of the Santa Monica Mountains.

TBM advance rates are expected to average from 20 to 50 feet per day (hard-rock formations, fault zones) to 70 to 100 feet per day (Old Alluvium and soft-rock formations). Oil and gas zones are expected to reduce excavation rates. On any tunnel project, there will be days during which no advancement is made due to unforeseen circumstances.

Estimates of TBM excavation rates are based on the results of this geotechnical investigation, on previous tunneling experience in the Los Angeles area and on judgment. Other variables, in addition to subsurface geologic conditions, can affect machine excavation rates, including skill of the operator and equipment reliability. Our estimated excavation rates assume the use of experienced crews, average machine downtime, and three eight-hour shifts per day.

## 1.8 ANTICIPATED GROUND BEHAVIOR IN SURFACE EXCAVATIONS

Surface excavations most likely to be associated with this project will include cut-and-cover stations and line segments, ventilation shafts and portals.

Owing to the limited available construction space and nature of the near-surface soils, virtually all surface excavations will have to be shored. Shored excavations to depths of approximately 80 feet can be designed and constructed along a majority of the alignment, using proven construction practices. Exceptions will include the wet alluvial deposits in Reach 1 and tar sands in Reach 5. For shored excavations in excess of 80 feet, special design will be required.

Existing structures along the alignment in close proximity to any proposed shallow tunnel and station excavations may require consideration of the following:

- relocation or razing of existing structure prior to making the new excavation
- construction of temporary or permanent walls between existing structure and new excavation
- underpinning or building a new foundation under an existing structure prior to proceeding with the proposed excavation.

## 1.9 DESIGN CONSIDERATIONS

Geotechnical design parameters and criteria have been developed that are suitable for current design, evaluating alignment and grade alternatives, and cost estimation. The major design parameters and criteria address the following:

- engineering properties of geologic soil and rock materials
- foundations for all structures
- surface excavations for underground structures
- design of underground structures by mining methods
- ground water considerations.

In general, soils underlying the proposed station locations will be dense, providing excellent foundation support for the station structures. Possible exceptions would be the soils in Reaches 1, 6 and 8 where the upper 40 feet may be less dense and will require either a continuous mat or deep foundation system for heavy concentrated loads. Surface facilities can generally be supported on spread footings, except in Reaches 1, 6 and 8 where heavy concentrated loads may require a deep foundation system such as piles.

Tar sands found at a depth of 40 to 50 feet in Reach 5 present special engineering considerations because current practice in Los Angeles precludes supporting foundation loads on or within tar sand materials. However, it is reasonable to assume that stations could be founded in these materials if future detailed studies of final structures are performed.

Design criteria are presented for underground structures constructed in surface excavations and include stations, cut-and-cover line segments and shafts. In addition to criteria for permanent structures, general criteria have been developed for temporary conditions such as: shoring systems, wall pressures, wall embedment, strut considerations, tieback considerations and bottom heave in excavations and shafts.

Design criteria have also been formulated for mined underground structures including bored tunnels and deep stations. These criteria for permanent structures in soil deposits or soft rock include: high loads, bending moments, buckling, loads from parallel tunnels and surcharge from surface structures.

Ground water conditions along the alignment are complex. For current design purposes, qualitative ground water design criteria have been developed based upon water level measurements available at this time.

## 1.10 SPECIFIC SUBSURFACE PROBLEMS IN DESIGN AND CONSTRUCTION

There are a few specific or local subsurface problems which will require special design considerations. These are as follows:

- high ground water table in loose alluvium in Reaches 1, 6 and 8
- tar sands in Reach 5
- seismic design considerations for the entire alignment (this will be the subject of a separate study)
- corrosion potential of ground water
- gas hazards, particularly in Reach 5 and eastern part of Reaches 1 and 3
- oil seepage, particularly in Reach 5 and eastern part of Reach 1
- loose Los Angeles River channel deposits in Reaches 1, 2 and 8
- crossing fault zones in blocky and seamy rock in Reach 7.

With proper engineering design and construction techniques, these conditions can be successfully solved.

# Section 2.0
# Introduction

# Section 2.0: Introduction

## 2.1 SITE LOCATION AND DESCRIPTION

The proposed 18-mile Metro Rail Project is mainly in the City of Los Angeles, California. The route extends from the downtown Los Angeles Union Station area to North Hollywood (Figure 2-1) via:

- East Portal, across the Los Angeles River, west on Macy Street to Broadway Street,
- Broadway Street to 7th Street,
- west on Wilshire Boulevard to Fairfax Avenue,
- north on Fairfax Avenue to Fountain Avenue,
- east on Fountain Avenue to Cahuenga Boulevard,
- north on Cahuenga Boulevard beneath the Santa Monica Mountains,
- terminating at Lankershim Boulevard along Chandler Boulevard and the Southern Pacific Railroad tracks (for future extension westward via railroad right-of-way).

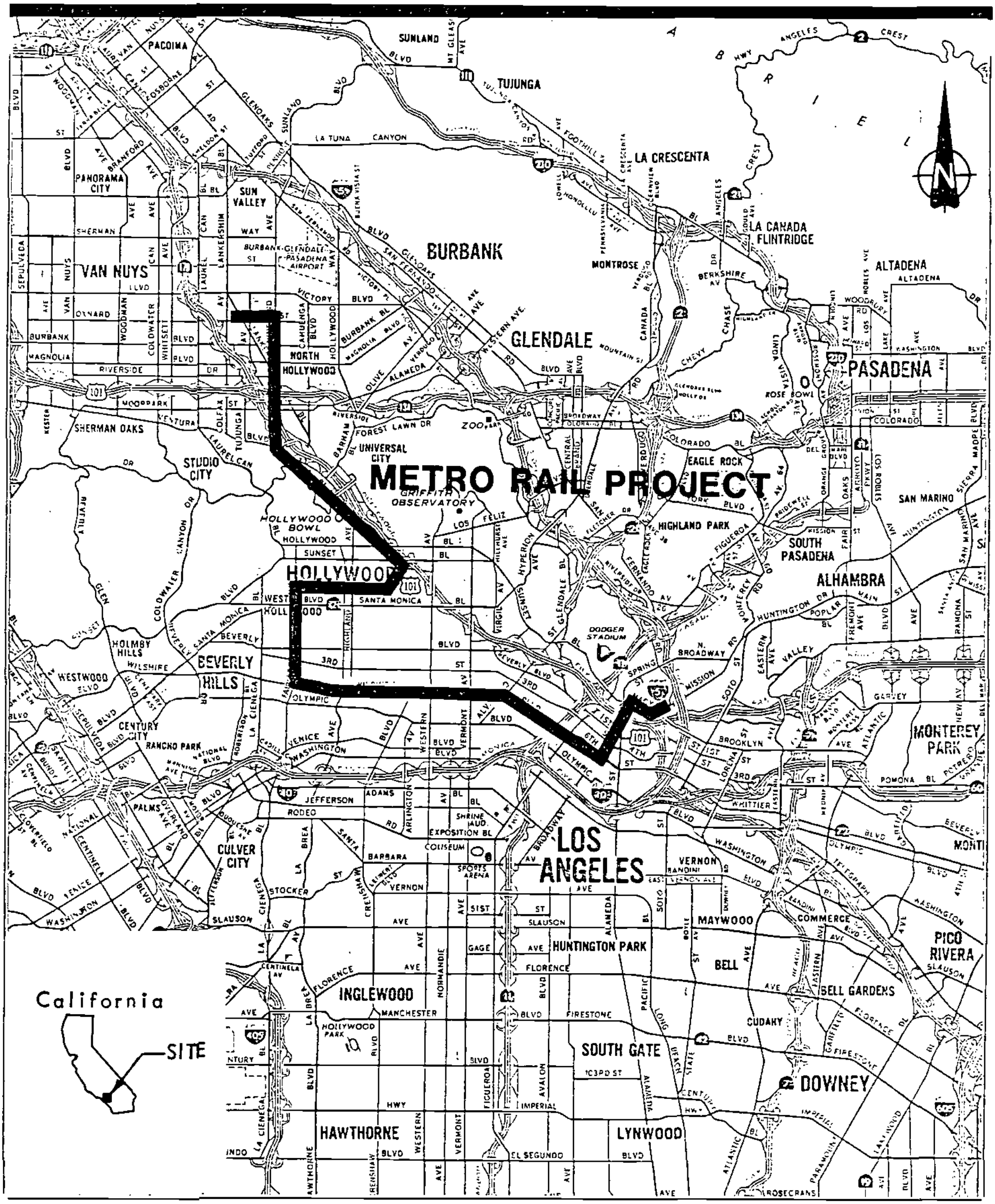
## 2.2 PROJECT PARAMETERS

At present, the project design requirements and structural features have not been established by the SCRTD or its design engineers. For purposes of geotechnical evaluation of line and station structures, the following dimensions are considered likely:

- line: 18- to 20-foot diameter opening, if a single-track, double-line configuration is selected
- line: 30- to 35-foot diameter opening, if a double-track system is selected
- station: 30 to 200 feet deep, 500 to 800 feet long, 50 to 100 feet wide
- station: construction may use both cut-and-cover and underground excavation (mining) methods.

This report addresses the following alternative grades:

- cut-and-cover
- shallow tunnel 30 to 100 feet deep
- deep tunnel greater than 100 feet deep.




SITE LOCATION MAP

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
80-1280

CWDD/ESA/GRC

Geotechnical Consultants

Figure No.
2-1

## 2.3 PURPOSE OF INVESTIGATION

The purpose of this geotechnical investigation and report was to conduct a subsurface investigation with field and laboratory testing of materials, interpret the data and prepare a comprehensive report of subsurface and surface conditions anticipated during construction along the proposed Metro Rail Project route. This information is to be used by design firms in preparing engineering designs.

## 2.4 SCOPE OF WORK

The scope of work for this investigation included the following tasks to assess geologic conditions along the proposed alignment:

- Literature search, including logs of existing borings
- Review of aerial photographs
- Geologic borings (41, for a total of 8,491 linear feet)
- Water pressures tests (16)
- Piezometer installations (23)
- E-logs (41): Self Potential and Resistivity, Natural Gamma-ray, Casing Collar Locator, Neutron, Caliper and Density
- Seismic refraction profiling (13,560 linear feet)
- Micro-gravity surveys (220 points at four locations)
- Downhole surveys (all 17 station sites)
- Crosshole surveys (10 selected station sites)
- Gas analysis (9 tests)
- Petroleum composition analysis (12 analyses)
- Water quality analysis (34 samples)
- Petrographic analysis (36 thin sections)
- Static laboratory tests
- Dynamic laboratory tests
- Data analyses
- Development of conclusions and recommendations
- Preparation of this report and appendices.

## 2.5 (LIMITATIONS) PHILOSOPHY OF INVESTIGATION

Ideally, an investigation of a construction site should yield precise, three-dimensional locations of rock and soil units, locations and extent of discontinuities, quantitative appraisals of the physical and chemical properties of the rock and soil units, and appraisal of the materials in and near discontinuities or other rock or soil properties. Estimates should be made of amounts and distribution of ground water, gas, oil, of rock or ground water chemistry, and of the possibilities of faulting and seismic activity. Such data will permit adequate, economical engineering planning and design and will be of great practical value during construction. In actual practice, however, exact prediction of all the geologic conditions that will be encountered during construction is almost never realized. For that reason, the geologist and geotechnical engineer should be given the opportunity to study and interpret for the design engineer the geologic features revealed during project construction.

The planned Metro Rail Project is an example of a major underground project where surface studies and numerous subsurface borings provide data that give some indication of subsurface conditions and engineering properties of soils and rocks. The findings can be used to assess site geology so as to permit reasonable engineering design and cost estimates.

## 2.6 ADDITIONAL STUDIES NECESSARY FOR FINAL DESIGN

This report is to assist the SCRTD and its design engineers in preparing current design concepts. Upon selection of appropriate line(s) and grade(s), additional geotechnical studies may be necessary to develop final design plans. Specific site problems envisioned at this time, worthy of additional studies, are presented in Section 10.0.

## 2.7 INSPECTION OF BORING SAMPLES

Samples recovered from borings are stored in a storage building at Converse Ward Davis Dixon, Inc. (CWDD), 126 West Del Mar Boulevard, Pasadena, California 91105 (213) 795-0461. As required in our contract, these samples will be stored at this location for four years; i.e., from December 1, 1980 to December 1, 1984. Inspection of samples by the designers and/or prospective construction contractors is encouraged. Authorization for inspection of the samples should be made through the Chief Engineer's office, Southern California Rapid Transit District, 425 South Main Street, Los Angeles, California 90013. Upon SCRTD authorization, arrangement for inspecting the boring samples can be made with CWDD.

## 2.8 INTERPRETATION OF GEOPHYSICAL DATA

Seismic refraction surveys, micro-gravity surveys, downhole and crosshole surveys, and geophysical borehole logging (E-logging) were performed in conjunction with other subsurface investigations. Drawings 1 and 3 show the locations at which these geophysical investigations were made and the basic data obtained are presented and interpreted in Appendices B, C, D and E. Calculations necessary to interpret these basic geophysical data should also be made by prospective bidders. Our report interprets data from the survey; however, because of the particular deductive nature of the interpretations and conclusions reached in this report, and the selective methods used, prospective bidders are encouraged to make their own interpretations of the basic data.

## 2.9 PREVIOUS INVESTIGATIONS

Specific previous investigations reviewed for this report, performed by others on or near the proposed Metro Rail Project, are:

- Kaiser Engineers, 1962, Test boring program, rapid transit system backbone route: 93 borings by Raymond Concrete Pile Division, Raymond International, Inc. (on proposed Metro Rail alignment)
- LeRoy Crandall and Associates, 1968, Report of preliminary foundation investigation, proposed rapid transit system for the Southern California Rapid Transit District: 1 boring (on proposed Metro Rail alignment)
- Yerkes, R.F., Tinsley, J.C., and Williams, K.M., 1977, Geologic aspects of tunneling in the Los Angeles area: US Geological Survey, Map MF-866
- Woodward-Clyde Consultants, 1977, Report of drilling services for Rapid Transit Starter Line: 10 borings.

# Section 3.0
# Description of Exploration and Testing Program

# Section 3.0: Exploration and Testing Program

## 3.1 DRILLING EXPLORATION PROGRAM

The drilling exploration program was performed for the SCRTD (WBS/12AAC) under the 1980 geotechnical contract, Audit No. 2256. A detailed description of the Metro Rail Project drilling exploration program, boring logs, colored core photographs, water pressure tests, ground water-level monitoring, and piezometer installation is presented in Appendix A, Volume II. Associated field explorations, performed in conjunction with the boring program were: E-logging, gravity surveys, downhole and crosshole surveys, gas and petroleum analyses, water quality analyses and petrographic analyses. These items are discussed briefly in this section, and in detail in Appendices A through G, and J, Volume II.

The following tables are in Appendix A, Volume II:

- Table A-1 Summary of Boring Data
- Table A-2 Correlation of N-Values and Consistency/Compactness of Soil Obtained in the Field
- Table A-3 Bedrock Description Terms
- Table A-4 Water Pressure Tests
- Table A-5 Ground Water Monitoring Summary.

### 3.1.1 Borings

Forty-one geologic borings were drilled at designated locations shown on Drawing 1 (in pocket), along the 18-mile alignment, by Pitcher Drilling Co. and P.C. Exploration, Inc. The 41 geologic borings varied from 100 to 400 feet deep, with most being approximately 200 feet in depth, for a total of 8,491 linear feet of drilling. In addition, 20 borings, 100 feet deep, were drilled for downhole placement of geophysical crosshole geophones at 10 selected station locations.

The drilling and coring of the 61 borings took place between December 1, 1980, and March 17, 1981. Although three additional borings were added to the drilling (including a 400-foot continuous hard-rock NX core hole), the program was successfully completed 14 days ahead of schedule. Sample and core recovery from the 41 geologic borings was very good. Sample recovery from the Converse ring standard penetration, Split Spoon and Pitcher Barrel samplers collectively averaged 77.6%. Copies of the geologists' original field boring logs, explanation sheets for descriptions and definition of symbols used on the boring logs, water-pressure test data, piezometer installation, and ground water monitoring and core photography are included in Appendix A, Volume II. A total of 1,650 linear feet of NX coring was performed in nine borings using wire line and a 5-foot long single tube core barrel; core recovery averaged 91.6%.

### 3.1.2 Drilling Contractors and Equipment

Pitcher Drilling Company of East Palo Alto, California, supplied three Failing 1500 rotary wash rigs, each operated by a two-man crew. For the hard-rock reaches of the alignment near the flanks and the interior of the Santa Monica Mountains, a CME-55 wireline core rig, owned and operated by P.C. Exploration, Inc., Roseville, California, was used. Although Pitcher Drilling Company was hired specifically to drill and sample the alluvium and soft formation reaches of the alignment, the firm also did a minor amount of NX coring in Borings 34 and 35 using a conventional 10-foot Christensen core barrel.

P.C. Exploration, Inc. was contracted to specifically core the hard rock formations. However, near the ground surface in the Santa Monica Mountains where rock quality did not permit succesful core recovery, samples were successfully obtained using a Pitcher Barrel sampler.

### 3.1.3 Soil Classification

All soil types were classified using the Unified Soil Classification System (USBR, 1974). Based on the characteristics of the soil, this system indicates the behavior of the soil as an engineering construction material. Using the Unified Soil Classification System, which is based primarily on laboratory test results, the site geologists arrived at a classification in the field that would be the same as the one obtained in the laboratory. Although particle size distribution estimates were based on volume rather than weight, the field estimates were within an acceptable range of accuracy.

### 3.1.4 Ground Water Level Monitoring and Piezometer Installation

A total of 6,047 linear feet of 2-inch plastic ABS pipe was installed in 31 of the 41 geology borings. In addition, 906 linear feet of 1-inch PVC plastic pipe was installed in 11 of these borings, alongside the 2-inch pipe, to monitor perched water levels. When double piezometers were installed in a boring, the two piezometers were separated vertically by a 3- to 5-foot bentonite seal placed below the upper piezometer and above the highest perforation of the lower piezometer. Several water-level readings were taken, the last in March, 1981 (see Table A-5, Appendix A, Volume II).

### 3.1.5 Water Pressure Testing

Water pressure and holding tests were performed in 16 of the 41 geology borings. Similar results were obtained using both pneumatic and push packers in the Puente and Fernando formations. Because of the more massive and clayey nature of the Fernando Formation, slightly better data were gathered. The fine sand interbeds of the Puente Formation tended to wash out around the packers at pressures between 20 to 60 psi. However, enough reliable data were obtained to demonstrate that both the Puente and Fernando formations are very tight and, in almost all cases, would take less than 1 gpm over a 20-foot test interval.

### 3.1.6 Color Photographs of Core Samples

Color photographs were taken of all samples of the Puente and Fernando formations extruded from the Pitcher Barrel, and NX core from Reach 7 through the Santa Monica Mountains (see Appendix A, Volume II).

## 3.2 E-LOGGING

All 41 geologic borings were electric-logged. A full suite, consisting of spontaneous potential and resistivity, natural gamma, casing collar locator, caliper, neutron and density logs, was performed in each boring. A detailed description of E-logging and a complete set of E-logs are presented in Appendix B, Volume II. When the various field and laboratory data were being compiled and synthesized in the office, the logs were used frequently to:

- facilitate interpretation of surface geophysics
- verify ground water depths
- check the field geologists' boring log descriptions
- prepare the Geologic Profile (Drawing 2) and Engineering Geology Profiles (Drawings 4 through 12) for this report.

A Model 3200 Gearhart Owen skid-mounted downhole logging unit, transported in a modified 3/4 ton-truck, was used for all E-logging. All downhole sondes were slim line (1 5/8-to 1 11/16-inch diameter) Gearhart Owen tools. The radioactive source used for the neutron devices was a Gulf Nuclear 3-curie AmBe; for the density tools, a 125-millicurie Cs source was used.

## 3.3 SEISMIC REFRACTION SURVEY

Details of the seismic refraction survey are presented in Appendix C, Volume II. A total of 52 seismic refraction lines, with a combined shot length of 13,560 linear feet, was recorded in six general locations along the proposed Metro Rail Project alignment. These lines were recorded during the months of February and March 1981, to evaluate the depth to and velocity characteristics of various subsurface materials, and to supplement information from exploratory borings. For the purposes of this report, a seismic refraction line consists of 12 in-line geophones spaced at equal intervals and monitored simultaneously with shots off both ends and optionally at the center of each line. Shot length is the distance between the two outermost shots recorded for each line.

Profiles showing subsurface velocity zones were constructed from interpretations of the data, and are presented in Figures C-1 through C-23, and maps showing the locations of seismic refraction lines are presented on Figures C-24 through C-29A (Appendix C, Volume II). Six areas were surveyed, as shown on Drawing 3 (in pocket).

The method and equipment used for the seismic refraction survey was the same for all 52 seismic refraction lines recorded. A 1980 SIE Model RM-49S 12-channel Geophysical Amplifier and Interval Stacker system was used in conjunction with an SIE 27-trace VRO 6 Dry-write Oscillograph. The vehicle used for housing and transporting equipment was a 1 1/4-ton four-wheel-drive van equipped with photographic darkroom and shock-mounted instrument bays. Due to the high level of ambient background noise present in the urban environment surveyed, hours of peak traffic were avoided, and most of the seismic refraction lines were recorded between the hours of 11 p.m. and 8 a.m. The energy source used was a Betsy Seisgun Source which consisted of a 21 mm inverted industrial shotgun that fired 3-ounce lead slugs into the ground. This energy source maximized signal energy (4 1/2 tons of muzzle energy) as well as ensured safety and efficiency in the urban environment surveyed.

### 3.4 MICRO-GRAVITY SURVEY

The micro-gravity survey data are presented in Appendix D, Volume II. Two hundred and twenty gravity stations were surveyed during the month of March 1981, at the locations shown on Drawing 3 (in pocket). For the purposes of this report, a gravity station is a point where the acceleraticn due to gravity is precisely measured at the earth's surface. Several regional stations were surveyed to establish regional gravity gradients of the area and to provide insight into large-scale, three-dimensional trends in subsurface structures. Closely spaced stations were surveyed over specific local features (buried channels and faults) to provide two-dimensional profiles for evaluating local subsurface structures. The gravity survey was performed to supplement information from exploratory borings and to provide a source of geophysical information in urban areas along the alignment not suitable for seismic refraction surveying.

The location of each gravity station was marked and surveyed for precise determination of elevation and location by Mollenhauer, Higashi and Moore, Inc., Land Surveyors, Los Angeles, California. Gravity measurements at all base stations were tied into gravity measured at one main base station, the CDMG Los Angeles State Building gravity station, so that a "universal" simple Bouguer anomaly could be calculated for each station surveyed.

Gravity measurements were recorded using a La Coste and Romberg Model D Gravimeter. All data were reduced using a Bouguer density of 2.0 gm/cc. Inner-zone terrain corrections were computed to a distance of 1.76 kilometers from the stations. Because this correction constitutes only a part of total terrain correction affecting the stations in the study area (from features in excess of 1.76 kilometers from the stations), the data are considered to represent "simple" rather than "complete " Bouguer anomalies.

The discussion of general structural trends observed on the Simple Bouguer Gravity Map (Drawing 3) and the density models used for gravity profiles 1 through 6 (presented on Figures D-1 through D-6, Appendix D, Volume II) represent a reasonable interpretation of gravity survey data based on knowledge of existing geologic conditions and information obtained from exploratory borings and seismic refraction surveying. The reliability of gravity survey data for this survey was limited by nearby features such as large buildings, retaining walls, underground pipes and vehicles parked in the roadways.

## 3.5 DOWNHOLE AND CROSSHOLE SURVEYS

The downhole and crosshole survey data, figures and tables are presented in Appendix E, Volume II.

### 3.5.1 Downhole Surveys

A downhole shear wave velocity survey was performed at each of the 17 proposed station sites along the proposed alignment. Measurements were made at 5-foot intervals from the ground surface to depths of 200 feet in borings, except in CEG 5 and CEG 31, where the depths of measurement were 130 and 115 feet, respectively.

Boring CEG 31, near the Hollywood Bowl Station, was found to be collapsing below 115 feet so the survey was terminated at this depth. All other borings were surveyed to their maximum depths. An additional downhole survey was performed in Boring CEG 23A because of the poor results obtained at Boring CEG 23.

Shearing energy was generated by striking a wood timber with a sledge hammer. At each measurement elevation (5-foot intervals), reversed horizontal blows were made at the ends of the timber to generate shear waves, and a vertical blow was made on the surface to generate compressional waves. A 12-channel signal enhancement seismograph (Geometrics Model ES 1210) allowed the summing of several blows in one direction when necessary to increase the signal-to-noise ratio. Usually, two records were made at each elevation to ensure repeatability of signal arrivals. A sidewall clamping triaxial seismometer array was lowered into the borings to record wave arrivals at 5-foot intervals to the bottom of the hole. Shear waves were identified by recording wave arrivals with opposite first motions on adjacent channels of the seismograph.

### 3.5.2 Crosshole Surveys

The crosshole technique for determining shear wave velocities in in situ materials was utilized in a 3-borehole array at each of 10 selected station sites. Crosshole seismic wave measurements at 5-foot vertical intervals were performed in each of 10 arrays. Each array consisted of three borings spaced approximately 15 feet apart. All borings were drilled to a depth of 100 feet. An end boring was cased with 4-inch I.D. plastic pipe, while the other two borings were cased with 3-inch I.D. plastic pipe. The pipes were grouted in place with cement grout. The shear wave hammer was placed in the end hole of the array, and vertical geophones were placed in the remaining two borings. The shear wave generating hammer and the two geophones were lowered to the same depth in all borings. The hammer was coupled to the wall of the boring by means of hydraulic jacks, and the geophones were coupled by means of expanding heavy rubber balloons which protruded from one side of the geophone housing. The hammer was 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 was used as a signal storage device. Upon casing the borings, deviation measurements were performed to determine orientation and location of borings at depth. Deviation measurements were made in a grooved aluminum casing that was inserted inside the casing of each boring of the array.

## 3.6 GAS AND PETROLEUM ANALYSES

Gas and petroleum analyses are presented in Appendices F1 and F2, Volume II.

### 3.6.1 Gas Analyses

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 subconsultant Ryland-Cummings, Inc. The hydrocarbon gases tested were: methane, ethane, propane, n-butane, isobutane, n-pentane, isopentane, and $C_6$+, undifferentiated. The non-hydrocarbon gases tested were: nitrogen, oxygen, carbon monoxide, carbon dioxide, and hydrogen sulfide. Specific hydrocarbon and non-hydrocarbon gases were collected at shallow depths at nine locations (Borings CEG 1, 2, 10, 11, 19, 21, 22, 23 and 23A). Samples of air were analyzed at each location 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.

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 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 (see Volume II, Appendix F, Sheets 1-20, inclusive).

### 3.6.2 Petroleum Analyses

Petroleum analyses and results are presented in Appendix F, Volume II. Laboratory analyses of petroleum samples were done by subconsultant Strata-Analysts Group. Fourteen samples were tested. They consisted of hydrocarbon-bearing sandstones and siltstones which were collected from eight different borings. Laboratory analyses were performed to identify the concentrations of oil and water and the hydrocarbon content. Identification of hydrocarbons was done using two chromatographic methods: (1) the PTC method, which generally defines compounds in the $C_1$ to $C_8$ normal hydrocarbon paraffin series, and (2) the Scot method, which generally defines compounds in the $C_8$ to $C_{18}$ normal paraffin series. The PTC method could not differentiate the very heavy tar-like hydrocarbons that were present in the samples because the samples were altered. Both chromatographic methods and techniques are proprietary; therefore, a description of the methods and techniques cannot be made available. Similarly, the complete chromatograms cannot be made available.

## 3.7 WATER QUALITY ANALYSES

Thirty-six water quality analyses were performed by Jacobs Laboratory, formerly Pomeroy Johnston and Bailey (PJB). The results are presented in Appendix G, Volume II. The water was tested for basic cations, anions, conductivity, total dissolved solids (TDS), pH, turbidity and Boron. Cation/Anion balance was not achieved on many of the samples because of the presence of an unmeasured cation, probably aluminum or barium.

## 3.8 LABORATORY TESTING PROGRAM

Results of the laboratory tests are presented in graphic or tabulated form in Appendix H, Volume II. The test program was conducted in general accordance with the schedule of tests specified in Table 1 of the SCRTD RFP No. 88025 and our Proposal No. 80-1855 dated August 1980. The number of tests performed was determined by the need to obtain more complete data or to eliminate unnecessary testing. The following laboratory test results appear on the boring logs in the Engineering Geologic Profiles, Drawings 5 through 12:

- Moisture/Density
- Unconfined Compression
- Static Triaxial Compression
- Cyclic Triaxial
- Dynamic Triaxial
- Resonant Column.

### 3.8.1 Index and Identification Tests

Field classification was verified in the laboratory by visual examination in accordance with the Unified Soil Classification System and American Standard Testing Methods (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.

Grain-size distributions were performed to assist in the soils classification and to correlate test data. Grain-size distributions were determined mainly by sieve analysis performed in accordance with ASTM D-422-63 test method. Hydrometer analyses were also performed when the soil had a significant percentage of soil particles passing the No. 200 sieve.

Atterberg Limit tests were performed on selected soils samples to determine their classification and their plasticity. The testing procedure was in accordance with ASTM D-423-66 and D-424-59 test methods.

Moisture content determinations were performed, using a modified version of the ASTM D-2261 test method.

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

### 3.8.2 Engineering Properties Tests

Unconfined compression tests were performed on selected samples of cohesive soils for the purpose of evaluating undrained, unconfined shear strength. The test procedure was strain-controlled loading in accordance with ASTM D-2166 test method.

Consolidated-undrained triaxial compression tests were performed on selected undisturbed soil samples. Some of the tests were performed as progressive tests.

Undrained, quick triaxial tests were also performed on selected undisturbed samples.

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

Swell tests were performed on selected undisturbed samples of cohesive, potentially expansive soils.

Consolidation tests were performed on selected undisturbed soil samples placed in 1-inch high, 2.42-inch diameter brass rings, or 3-inch diameter Shelby tubes trimmed to a 2.42-inch diameter. The apparatus used for the consolidation test was designed by Converse to receive the 1-inch high brass rings directly from the field. The data obtained from these special observations were used to estimate the rates of consolidation and excess pore-pressure dissipation of these soils in the field during and after construction.

Permeability tests were generally performed in conjunction with the static and cyclic triaxial tests, using the same selected undisturbed samples of soil.

Porosity and void ratio of selected undisturbed samples were determined by measuring the dry unit weight and specific gravity, then calculating the void ratio, (e) and porosity, (n).

The cyclic triaxial compression test evaluates soil shear strength, liquefaction, and deformation characteristics under cyclic loading conditions. A cylindrical specimen of soil was 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.

The resonant column test determines the shear modulus and damping of soil specimens at shear strain values of approximately $10^{-6}$ to $10^{-4}$ inches per inch. A cylindrical soil specimen was encased in a thin membrane, placed in a pressure cell and subjected to the desired ambient stress conditions. The specimen was 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 was measured using an accelerometer coupled to the free end. Shear modulus and damping values were calculated from the response data.

## 3.9 TBM MANUFACTURERS ROCK TEST RESULTS

The results of tests performed by TBM manufacturers on selected "hard-rock" samples from Santa Monica Mountain Borings CEG 30, 32, 32A and 33 are presented in Appendix I, Volume II. A summary of test data is shown on Table 5-6 in Section 5.0. The TBM manufacturers did not test the "soft-rock" samples submitted by the Converse team.

## 3.10 PETROLOGY

Results of 26 petrographic analyses, performed on selected rock samples obtained from Santa Monica Mountain cores in Borings CEG 30, 31, 32 and 32A, are presented in Appendix J, Volume II. The objectives of the petrographic analyses were to:

- verify and supplement field identification of lithologies and rock affinities
- provide additional data on the mineralogical and micro-structural characteristics of selected rock samples recovered from cores
- compare and correlate lithologic characteristics of subsurface samples
- provide data to aid in interpretation of geologic relationships in the general vicinity of Borings CEG 30, 31, 32 and 32A.

Thin sections were prepared by Von Huene's Petrographic Thin Section Laboratory, Pasadena, California. Each rock sample was slabbed, trimmed, and the resulting rock chip was mounted on a standard 27 x 46 mm glass slide. The mounted rock chip was then ground to a uniform thickness of 0.03 mm, polished and covered with a thin glass. If the rock sample was poorly consolidated, highly fractured or porous, it was impregnated with resin prior to preparation. The thin sections were routinely stained with sodium cobaltinitrate to aid in the identification of potash feldspars. A petrographic polarizing microscope was used to examine the petrographic sections.

# Section 4.0: Project Geology

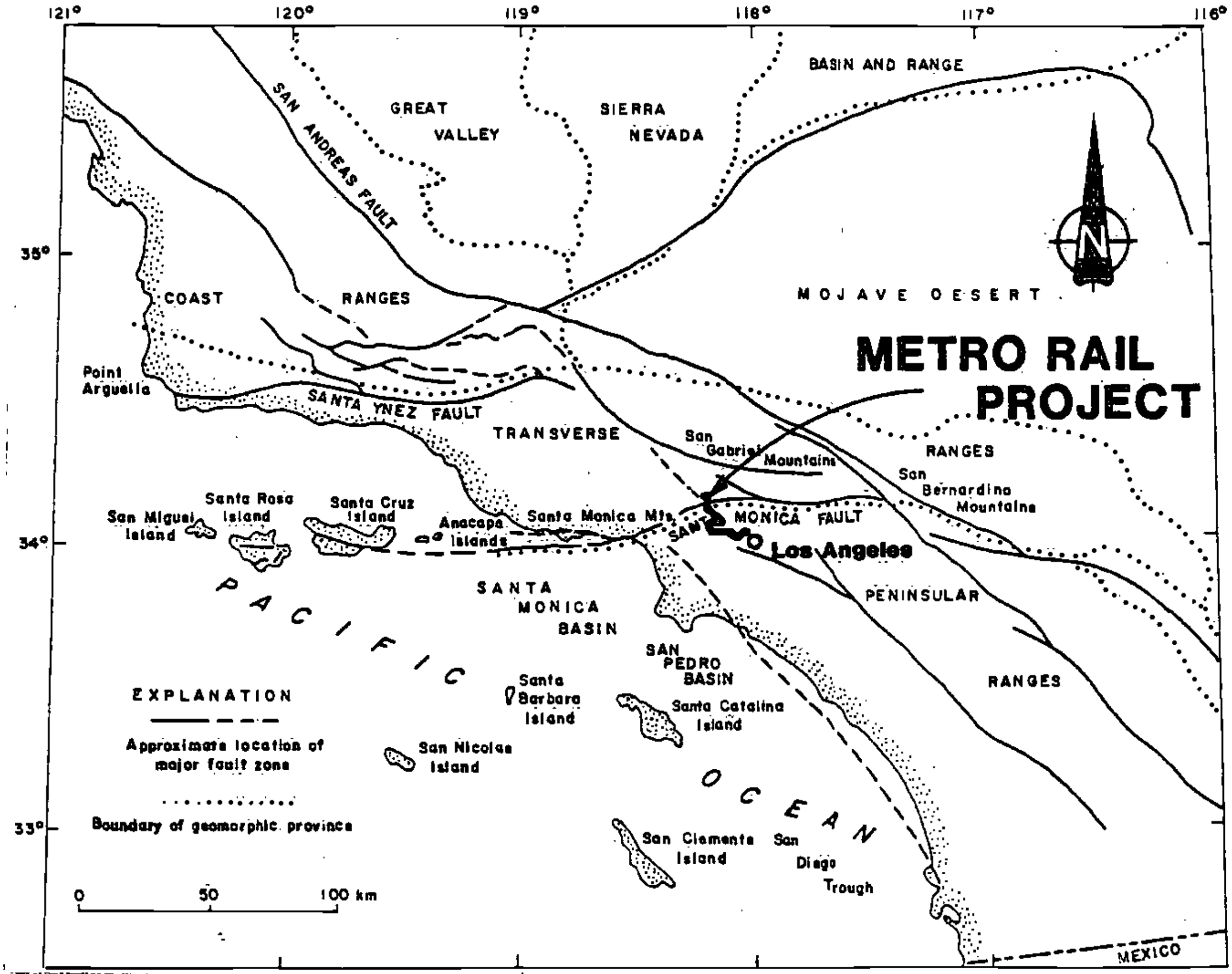
## 4.1 REGIONAL PHYSIOGRAPHIC FEATURES

The proposed Metro Rail alignment will be located at a junction of California's Transverse Ranges and Peninsular Ranges Physiographic Provinces in the Los Angeles area (Figure 4-1). Unlike most of the northwest trending structural features of coastal California, the Santa Monica Mountains in the Transverse Ranges Province trend east-west. The proposed alignment will cross three distinct geomorphic features: the Los Angeles Basin, Santa Monica Mountains and San Fernando Valley, as shown on the Generalized Geologic Map of the Los Angeles Region (Figure 4-2).

## 4.2 REGIONAL GEOLOGIC HISTORY

Geologic history in Miocene time includes deposition of marine sediments accompanied by intrusion and extrusion of as much as 4,000 feet of basaltic material. During this period and into early Pliocene time, the Santa Monica Mountains were uplifted and eroded. Resulting sediments were collected in the tectonically downwarped Los Angeles Basin on the south flank of the Santa Monica Mountains. By late Miocene or early Pliocene time, the depth of the ocean embayment into the Los Angeles Basin reached approximately 4,000 to 6,000 feet. Marine sediments approaching 5,000 feet in thickness were deposited during early Pliocene time in the Central Los Angeles Basin. During late Pliocene time, the downwarped Los Angeles Basin became shallower and more limited in extent as it filled with another 3,500 feet of sediments. The ocean shoreline during early Pleistocene time extended along the south side of the Santa Monica Mountains and the Elysian Hills (Drawing 1). These hills were offshore features in the process of emerging from the sea. Uplift, accompanied by erosion, was relatively rapid in early Pleistocene time, and the downdropped Los Angeles Basin continued filling with an additional 1,300 feet of unconsolidated San Pedro sand sediments. Although the Los Angeles Basin was below sea level much of the time, world-wide fluctuations of sea levels during Pleistocene time caused the ocean to withdraw several times.

Holocene time began with the last melting of the ice sheets (11,000 years ago), and resulted in coarse gravels and sands being deposited in stream channels extending into the Los Angeles Basin. In the Ballona Creek area (Figure 4-2), such material is encountered up to 60 feet below the present sea level. Events that have occurred during the period of recorded history help to reveal what has happened during the geologic past. For example, prior to 1825 and again during the flood of 1867-68, the Los Angeles River flowed westerly from the City of Los Angeles into Ballona Creek, instead of south through the Los Angeles Basin (Figure 4-2) to Long Beach, its present route. Therefore, it is reasonable to assume that, if rapid changes in the drainage patterns can occur in such a short time, they may have occurred many times in the geologic past along the proposed Metro Rail Project alignment.




REFERENCE:

Modified from Junger, 1977, Sheet 1, Figure 1

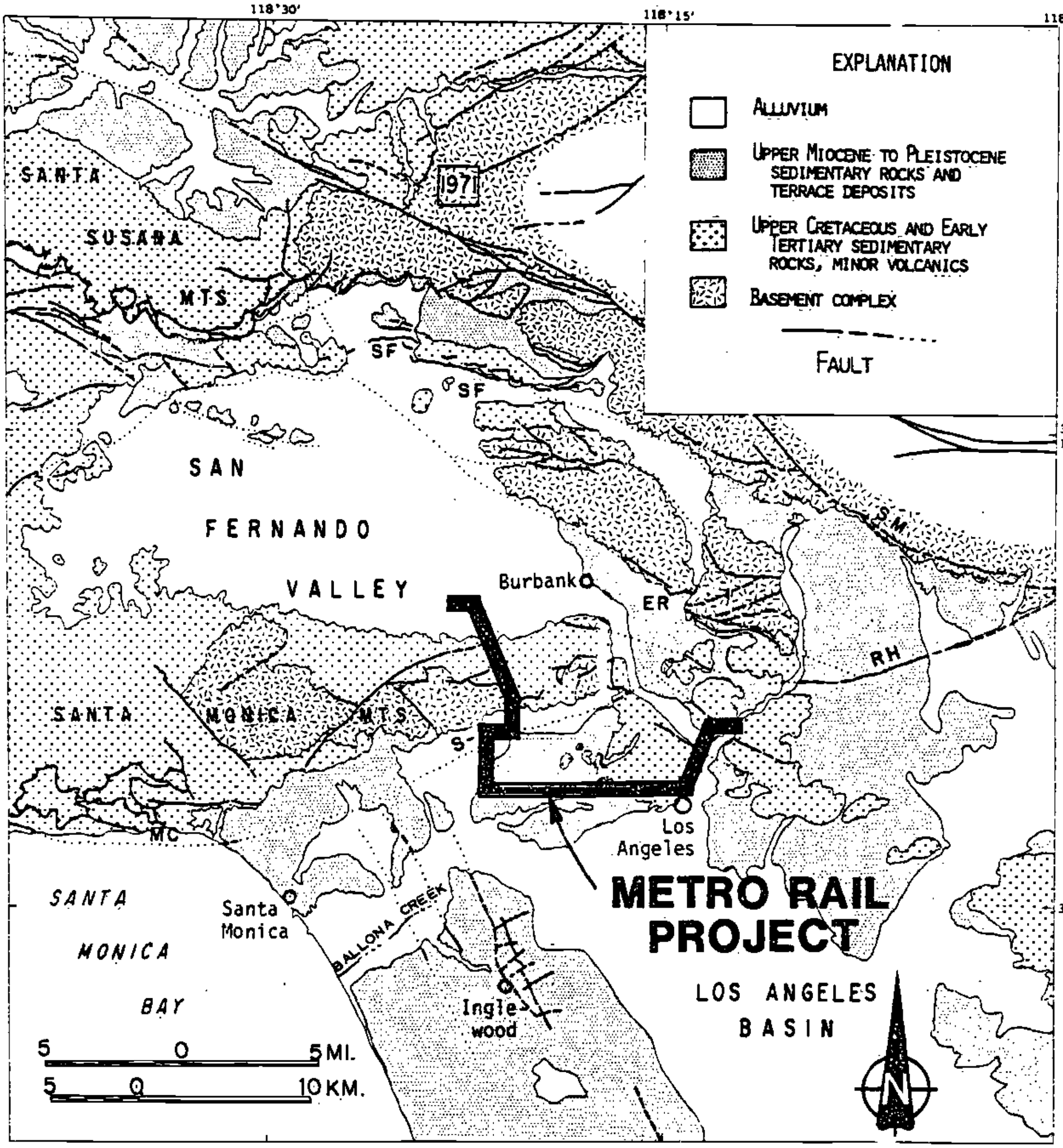
MAP OF TRANSVERSE RANGES AND PENINSULAR RANGES PHYSIOGRAPHIC PROVINCES IN THE LOS ANGELES AREA

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No.
4-1




Reference: Modified from Yerkes and others, 1977, Figure 1

GENERALIZED GEOLOGIC MAP OF THE LOS ANGELES REGION

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 4-2

## 4.3 PROJECT STRATIGRAPHY

### 4.3.1 Geologic Formations

The Metro Rail alignment will encounter several geologic formations from downtown Los Angeles to North Hollywood. The areal distribution of these materials is presented on Drawing 1, Geologic Map. These materials, with accompanying geologic map symbol, are listed below in order of increasing age:

| | | |
|---|---|---|
| Young Alluvium | (Qal) | Silt, sand, gravel, and boulders; chiefly unconsolidated (loose) and granular. |
| Old Alluvium | (Qalo) | Clay, silt, sand, and gravel; chiefly consolidated (stiff) and fine-grained. |
| San Pedro Formation | (SP) | Sand; clean, relatively cohesionless; locally impregnated with oil or tar (Formation not exposed at surface on geologic map). |
| Fernando Formation | (Tf) | Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone beds. |
| Puente Formation | (Tp) | Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone beds. |
| Topanga Formation | (Tt) | Siltstone, sandstone, conglomerate; chiefly hard, well cemented, massive sandstone; local soft, thin siltstone beds; includes some Cretaceous conglomerate and sandstone, undifferentiated. |
| Topanga Formation | (Tb) | Basalt; includes dolerite and amygdaloidal andesitic basalt; non-columnar flows and intrusives; deeply weathered, soft, crumbly at surface; hard, unweathered at depth. |

Other geologic formations are shown on the Geologic Map (Drawing 1) but will not be encountered during construction. These are:

| | | |
|---|---|---|
| Alluvial Fan | (Qf) | Silt, sand, gravel, and boulders; primarily semi-unconsolidated (dense) and granular. |
| Modelo Formation | (Tm) | Claystone, siltstone, sandstone; chiefly soft, diatomaceous stratified siltstone; local hard sandstone beds. |
| Granite | (Cg) | Chiefly granodiorites; deeply weathered, soft at surface; hard, unweathered at depth. |

### 4.3.2 Geologic Formations Subdivided into Geologic Units

The geologic formations have been subdivided into geologic units on Drawing 2, Geologic Profile, because there are different physical properties within the formations shown on the Geologic Map (Drawing 1).

More importantly, the geologic units (Drawing 2) are classified in terms of 'soft-ground tunneling' and 'rock tunneling' to assist designers.

#### 4.3.2.1 Soft-Ground Tunneling

| | |
|---|---|
| $A_1$ | YOUNG ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, sandy gravels, and locally (mainly Reaches 1, 2 and 8) contains cobbles and boulders. Primarily dense, but ranges from loose to very dense (Geologic Map symbol Qal, undifferentiated on Drawing 1). |
| $A_2$ | YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from soft to very stiff (geologic Map symbol Qal, undifferentiated on Drawing 1). |
| $A_3$ | OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium to very dense, containing more cohesive material than $A_1$ (Geologic Map symbol Qalo, undifferentiated on Drawing 1). |
| $A_4$ | OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from medium to hard; contains more cohesive material than $A_2$ (Geologic Map symbol Qalo, undifferentiated on Drawing 1). |
| SP | SAN PEDRO FORMATION: Predominantly clean, relatively cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium to very dense. Locally (mainly Reach 5) impregnated with oil or tar (Formation not exposed at surface on Geologic Map, Drawing 1). |
| C | FERNANDO AND PUENTE FORMATIONS: Claystone, siltstone, and sandstone; thinly to thickly bedded. Primarily low hardness, weak to moderately strong, but locally contains hard, thin sandstone beds (Geologic Map symbols Tf and Tp on Drawing 1). |

#### 4.3.2.2 Soils Density and Consistency Terms

The following correlation of density/consistency terms with standard penetration information is used to describe all soil materials (Peck and others, 1974):

| Sands | | Clays | |
|---|---|---|---|
| Number of Blows per ft, N | Relative Density | Number of Blows per ft, N | Consistency |
| | | Below 2 | Very Soft |
| 0 - 4 | Very loose | 2 - 4 | Soft |
| 4 - 10 | Loose | 4 - 8 | Medium (firm) |
| 10 - 30 | Medium | 8 - 15 | Stiff |
| 30 - 50 | Dense | 15 - 30 | Very stiff |
| Over 50 | Very dense | Over 30 | Hard |

#### 4.3.2.3 Rock Tunneling

| | |
|---|---|
| 1-5 | TOPANGA FORMATION: Conglomerate, sandstone and siltstone; thickly bedded; primarily hard (Geologic Map symbol Tt on Drawing 1). |
| 1-3 | TOPANGA FORMATION: Basalt; intrusive, primarily hard (Geologic Map symbol Tb on Drawing 1). |

### 4.3.3 Geotechnical Terms and Definitions

The following are definitions of geotechnical terms used in this report.

| | |
|---|---|
| FAULT | |
| A rock fracture along which there has been a displacement of the sides relative to one another parallel to the fracture. | |
| FAULT ZONE | |
| A fault that is expressed in relative terms of width. A fault zone may be tens of centimeters or several kilometers in width. A fault zone may consist of fault gouge, fault breccia, fault blocks, or many related faults together with fractured and crushed rock, or any combination of any of these. In common usage, many well known fault zones are simply referred to as faults, such as the San Andreas fault. | |
| ACTIVE FAULT | |
| A fault that can be shown to exhibit displacement at or near the ground surface at least once within the past 10,000 years; i.e., Holocene displacement. | |
| POTENTIALLY ACTIVE FAULT | |
| No known displacement within the past 10,000 years, but exhibits evidence of Quaternary fault displacement within the past 2,000,000 years. | |
| FAULT BRECCIA | |
| Cemented to uncemented angular and commonly slickensided rock fragments that may range from sand-sized to many meters in diameter, usually with a matrix of fault gouge. Fault breccia is often found in fault zones. | |
| SLICKENSIDES | |
| A polished, usually striated rock surface resulting from scraping of rock surfaces as differential movements occur within rock, along a fault plane, or within a fault zone. Certain types of broken or crushed rocks, as shales, may be filled with shiny slickensided fractures. | |
| MAXIMUM CREDIBLE EARTHQUAKE (MCE) | |
| The largest magnitude event which can be reasonably postulated to occur, based upon existing geologic and seismologic evidence independent of time. | |
| SHEAR | |
| A rock structure with the physical characteristics of a fault, such as slickensides, fracturing, or gouge, but where evidence of differential movement (displacement) is not directly observed, only inferred. | |
| SHEAR ZONE | |
| A shear zone may consist of gouge, breccia, large rock blocks, many related shears together with fractured and crushed rock, or any combination of these materials where evidence of differential movement (displacement) is not directly observed, only inferred. | |
| JOINTS AND FRACTURES | |
| The terms "joints" and "fractures", as applied to rock descriptions, are interchangeable; however, in the CEG boring logs included in this report, the term "joint" is used only for the more extensive and well developed rock fractures. | |
| PERCHED GROUND WATER | |
| Ground water occurring in a saturated zone, lying above and separated from the permanent ground water table by impervious material. | |
| PERMANENT GROUND WATER | |
| The upper surface of a body of free water which completely fills all openings; in fractured impervious rocks, it is the surface at the contact between the water body in the openings and the overlying ground or air. | |

| | |
|---|---|
| ARTESIAN WATER | Ground water that is under sufficient pressure to rise above the level encountered by a boring, but that does not necessarily rise to or above the surface of the ground. |
| WET (w) | Infers Units $A_1$, $A_3$ and SP are judged likely to be saturated. Infers Unit C and other units are judged likely to be saturated below permanent water levels, to extent of seeps and inflows. |
| BOULDERY (b) | Infers Unit $A_1$ is judged to contain some boulders up to 4 ft diameter. |
| SOIL PROPERTIES | The consistency and compactness of geologic Units $A_1$, $A_2$, $A_3$, $A_4$, SP and C are described in the boring logs and report in accordance with subsection 4.3.2.2. |
| ROCK PROPERTIES | The physical condition, hardness, strength and weathering characteristics of geologic Units Tt and Tb are described in the boring logs and report in accordance with Table A-3, Appendix A, Volume II. |
| N-VALUE | Number of blows per foot of penetration using a standard penetration sampler with a weight of 140 pounds falling 30 inches (ASTM D1586-67) |
| PETROLIFEROUS (p) | Infers some geologic units are judged to contain either oil, tar or gas. |
| SOFT-GROUND TUNNELING | Refers to geologic units $A_1$, $A_2$, $A_3$, $A_4$, SP and C as shown on Drawing 2 and described in subsection 4.3.2.1 |
| ROCK TUNNELING | Refers to geologic formations Tt and Tb as shown on Geologic Map, Drawing 1; the Geologic Profile, Drawing 2, classifies these geologic formations in terms of Terzaghi Rock Conditions Numbers 1 (hard and intact) through 9 (swelling rock). |
| SHEAR WAVE, TRANSVERSE WAVE OR S-WAVE | A body wave in which the particle motion is perpendicular to the direction of propagation. |
| COMPRESSION WAVE, LONGITUDINAL WAVE, OR P-WAVE | A wave in which an element of the medium changes volume without rotation. |
| ANTICLINE | Strata that dip in opposite directions (upfold) from a common ridge or axis, like the roof of a house. |
| SYNCLINE | Strata that dip inward (downfold) from both sides toward a common axis; like a bowl. |

### 4.3.4 $A_1$ - Young Alluvium (Granular)

Young Alluvium, designated $A_1$ on Drawing 2, of Holocene age is a relatively modern (in terms of geologic time), granular material deposited in swift streams. The areal distribution is shown as Qal (undifferentiated) on Drawing 1. These deposits occur along the Metro Rail alignment, primarily in Reaches 1, 2 and 8 (Drawing 2). Specific types of materials are described in logs of Borings CEG 3 through 9, and 35 through 38 (Appendix A, Volume II). Engineering properties are illustrated on Engineering Geology Profiles, Drawings 5 and 12. Some important characteristics are:

Material - Clean sands and gravels but includes silty sands, gravelly sands, sandy gravels, cobbles and boulders.

Compactness - Primarily dense, but ranges from loose to very dense; relatively cohesionless compared to Old Alluvium Unit $A_3$.

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.

### 4.3.5 $A_2$ - Young Alluvium (Fine-grained)

Young Alluvium, designated $A_2$ on Drawing 2, of Holocene age is a relatively modern stream deposit, but differs from $A_1$ by being predominantly fine-grained and deposited in relatively "quiet" water. These deposits occur near the surface of the proposed Metro Rail alignment, primarily in the northern half of Reach 5, Reach 6 and the north end of Reach 7 (Drawing 2). Irregular-shaped lenses of $A_2$ interfinger with $A_1$ in Reaches 2 and 8. Specific types of materials are described in logs of Borings CEG 22 through 28, 33 and 34 (Appendix A, Volume II). Engineering properties are illustrated on Engineering Geology Profiles, Drawings 8, 9, 11 and 12. Some important characteristics are:

Material - Clayey silts and sandy silts but includes clays, sandy clays and clayey sands.

Consistency - Primarily stiff, but ranges from soft to very stiff.

Non-Bouldery Ground - Boulders were not encountered in the borings. However, their absence cannot be completely assured because $A_2$ is associated with flood plain deposits that are judged to have had boulder-size carrying capacity during past floods.

### 4.3.6 $A_3$ - Old Alluvium (Granular)

Old Alluvium, designated $A_3$ on Drawing 2, of Pleistocene age is a granular material deposited in relatively swift water, but differs from $A_1$ in that it contains more cohesive material. The areal distribution is illustrated as Qalo (undifferentiated) on Drawing 1. These deposits occur as irregular-shaped lenses in the eastern portion of Reaches 1, 4 and 6 and as deep sediments under the Young Alluvium of Reach 8 in the San Fernando Valley (Drawing 2). Specific types of materials are described in the logs of Borings CEG 15, 16, 17 and 35 through through 38 (Appendix A, Volume II). Engineering properties are illustrated on Engineering Geology Profiles, Drawings 5, 7 and 12. Some important characteristics are:

Material - Silty sands, but includes clean sands, gravelly sands, and sandy gravels.

Compactness - Primarily dense, but ranges from medium to very dense.

Non-Bouldery Ground - Boulders were not encountered in the borings. However, these are relatively swift water deposits judged capable of carrying boulder-sized material. The quantity of boulders is believed to be less than in $A_1$.

### 4.3.7 $A_4$ - Old Alluvium (Fine-grained)

Old Alluvium, designated $A_4$ on Drawing 2, of Pleistocene age is a fine-grained material deposited in relatively "quiet" water, but differs from $A_2$ in that it contains more cohesive material. These deposits are widespread, occurring at the surface in Reaches 3, 4 and 5, as deep deposits beneath the surface in Reaches 5 and 6, and as irregular-shaped lenses in Reach 8 (Drawing 2). Specific types of materials are described in logs of Borings CEG 14 through 28, 35 and 36 (Appendix A, Volume II). Engineering properties are presented on Engineering Geology Profile, Drawings 6 through 9 and 12. Some important characteristics are:

Material - Clayey silts and sandy silts, but includes clays, sandy clays and clayey sands.

Consistency - Primarily stiff, but ranges from medium to hard.

Non-Bouldery Ground - Boulders were not encountered in the borings. Only a few scattered boulders are believed present because of the distant downstream location from the Santa Monica Mountains; i.e., reduced carrying capacity of streams and relatively "quiet water deposition".

### 4.3.8 SP - San Pedro Formation

The San Pedro Formation of Pleistocene age is a clean "beach type" sand, deposited in a shallow sea environment, unconformably overlying the Fernando/Puente formations and underlying the Old Alluvium. The sand is not exposed at the surface; therefore it does not show on the Geologic Map, Drawing 1. However, the proposed Metro Rail alignment will encounter the sand in Reaches 4 and 5 (Drawing 2) if track grade is 50 to 150 feet below the ground surface. Specific types of materials are described in logs of Borings CEG 15 through 23A (Appendix A, Volume II). Engineering properties are shown on Engineering Geology Profiles, Drawings 7 and 8. Some important characteristics are:

Material - Predominantly bedded and cross-bedded, clean, relatively cohesionless fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels; locally contains claystone/siltstone interbeds as in Boring CEG 23. A good surface exposure of the San Pedro sand occurs in Bent Springs Canyon, Palos Verdes Hills, about 15 miles south of the project (Figure 4-3). This photograph shows about 100 feet of sand, cross-bedded sand, and interbedded layers and lenses of fine gravel and siltstone.

Compactness - Dense but ranges from medium to very dense.

Petroliferous - Contains oil, tar and gas in Reach 5, Borings WC-6 through CEG 23A (Drawing 2), above the Salt Lake Oil Field.

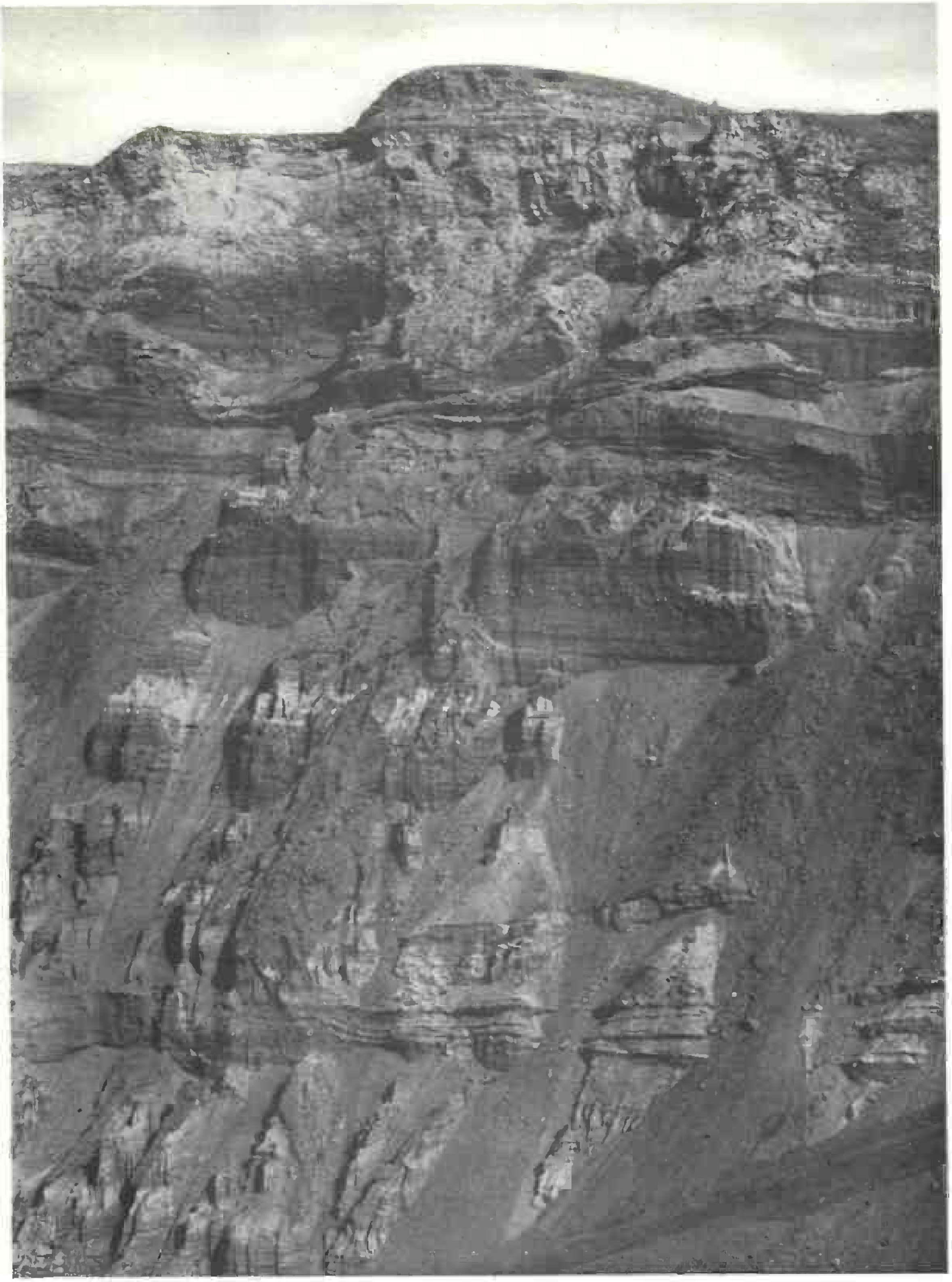


Figure 4-3

San Pedro Sand
Well bedded and cross-bedded sand with interbeds of siltstone.
Exposure in Bent Springs Canyon, Palos Verdes Hills, south of project area.

Ref: Woodring and others (1946), Plate 19.

### 4.3.9 C - Fernando Formation

The Fernando Formation of Pliocene age consists of well stratified claystone and siltstone with interbeds of sandstone. The areal distribution is shown as Tf on Drawing 1 and profile extent designated C on Drawing 2. The Fernando Formation conformably overlies the Puente Formation. The lithologic contact between the Fernando Formation and Puente Formation is gradational and difficult to locate accurately, whether in the subsurface or on the surface, because the composition of the materials is similar. The Fernando Formation wraps around the Puente Formation at the eastern end of the Los Angeles Anticline and rises to the surface in the downtown Los Angeles area (Drawing 1). The proposed Metro Rail alignment will encounter the Fernando Formation in Reach 2 if track grade is 10 to 150 feet deep, and in the western portion of Reach 4 and a portion of Reach 5 if the track grade is greater than 80 feet deep (Drawing 2). Specific types of materials are described in the logs of Borings CEG 7 through 10 and 17 through 22 (Appendix A, Volume II). Engineering properties are shown on Engineering Geologic Profiles, Drawings 5, 6, 7 and 8. Some important characteristics are:

Material - Claystone, siltstone and sandstone, mostly thinly bedded. Figure 4-4 is typical of the Fernando Formation.

Bedding Attitudes - Based on limited surface exposures, combined with the fact the Metro Rail alignment parallels the south flank of the Los Angeles Anticline, bedding is judged to trend northwesterly with attendant southwesterly dips ranging from 5° to 60°. Thus, the beds would dip unsupported into near-vertical, southerly-facing excavations. The trend of bedding would cross the alignment at about 45°, thus individual beds could follow the alignment several hundreds of feet.

Hardness - Low hardness, weak to moderately strong. Locally contains hard sandstone beds ranging from less than 1 inch to 3 feet in thickness; i.e., Borings CEG 9, 10, 18, 20, 21 and 22. Hard beds are estimated to comprise less than 1% of the Formation; estimated unconfined compressive strength may exceed 15,000 psi.

Slaking - The clayey beds air-slake (deteriorate 1 to 6 inches into the excavated surface) within a day or two when exposed in surface excavations. Therefore, clayey beds are judged to air-slake in either cut-and-cover line segment excavations or in tunnel excavations when subjected to high volume ventilation air flow (for gassy reaches). Additional deterioration of the tunnel invert is considered likely if the clayey beds are exposed to wetting and continuous construction traffic.

Petroliferous - Contains oil, tar and gas in Reach 5, Borings WC-6 through 22 (Drawing 2) in the Salt Lake Oil Field.

### 4.3.10 C - Puente Formation

The Puente Formation of upper Miocene age consists of well stratified claystone and siltstone with interbeds of sandstone. The areal distribution is shown as Tp on Drawing 1 and profile extent designated C on Drawing 2. The Puente Formation ranges from thinly to thickly bedded. Field observations indicate that the bedding has been contorted and deformed as the result of slumping and sliding, contemporaneous with deposition. These contorted beds, which are commonly associated with coarse-grained pebbly sandstone layers, suggest that the deposition of these coarser sediments may have initiated slumping in the previously deposited siltstone beds. The thin-bedded sequence

Figure 4-4

Fernando Formation
Thin bedded siltstone and sandstone

Ref: Lamar (1970), p. 35.

illustrated in Figure 4-5 occurs in Chavez Ravine (Dodger Stadium), one mile north of Boring CEG 6. Figure 4-6 is an illustration of the thicker sequence of sandstone and interbedded siltstone of the Puente Formation, as exposed on the north flank of the Los Angeles Anticline (Drawing 1) along Glendale Boulevard.

The Puente Formation will be the chief material encountered in Reach 1, if track grade is 5 to 120 feet deep, and in Reaches 3 and 4 (Drawing 2) if track grade is 15 to 90 feet deep. Specific types of materials are described in the logs of Borings CEG 1 through 6, and 11 through 16 (Appendix A, Volume II). Engineering properties are shown on Engineering Geology Profiles, Drawings 5, 6 and 7. Some important characteristics are:

Material - Thick bedded sequence of claystone, siltstone and sandstone. The major difference between the Puente Formation and Fernando Formation is that the Puente Formation:

a) exhibits consistently thicker beds,

b) contains a larger quantity of hard sandstone beds ranging from less than 1 inch to 3 feet in thickness, and

c) contains more clay-size particles.

Hardness - Low hardness and weak to moderately strong. Locally contains hard sandstone beds ranging in thickness from less than 1 inch to 3 feet; e.g., Borings CEG 1, 4, 12, 13, 14 and 16. Hard beds are estimated to comprise less than 2% of the formation; the estimated unconfined compressive strength may exceed 15,000 psi.

Slaking - Tends to air-slake in surface excavations, slightly more than the Fernando Formation because of more clay content. The formation is judged to air-slake in either cut-and-cover line segment excavations or in tunnel segments when subjected to high volume ventilation air flow (for gassy reaches).

Petroliferous - Contains oil, tar and gas in Reach 1, Borings CEG 1 through 4, inclusive; Boring CEG 2 filled with oil to within 15 feet of the ground surface two days after drilling was completed. Reach 3 can also expect oil and gas.

### 4.3.11 Topanga Formation (Sedimentary Rocks)

The Topanga Formation of Middle Miocene age and designated Terzaghi Rock Condition Numbers 1 through 5, inclusive, on Drawing 2, consists of a hard, well-cemented massive to thickly bedded sandstone with interbeds of siltstone and conglomerate. The areal distribution is shown as Tt on Drawing No. 1. The thin siltstone interbeds are low hardness. Figure 4-7 is an illustration of alternating thick- and thin-bedded conglomerate, sandstone and siltstone similar to material believed to be encountered in the Metro Rail alignment. Figure 4-8 is an illustration of the hard, massively bedded sandstone at Cahuenga Pass, north of Boring CEG 31. These hard to very hard, massive, well-cemented conglomerate and sandstone beds are anticipated in the core of the Santa Monica Mountains adjacent to the intruded basalt in Reach 7 (Drawing 2). Sandstones and siltstones encountered at the base of the Santa Monica Mountains in Borings CEG 28A, 29, 34 and 35, are of low to moderate hardness.

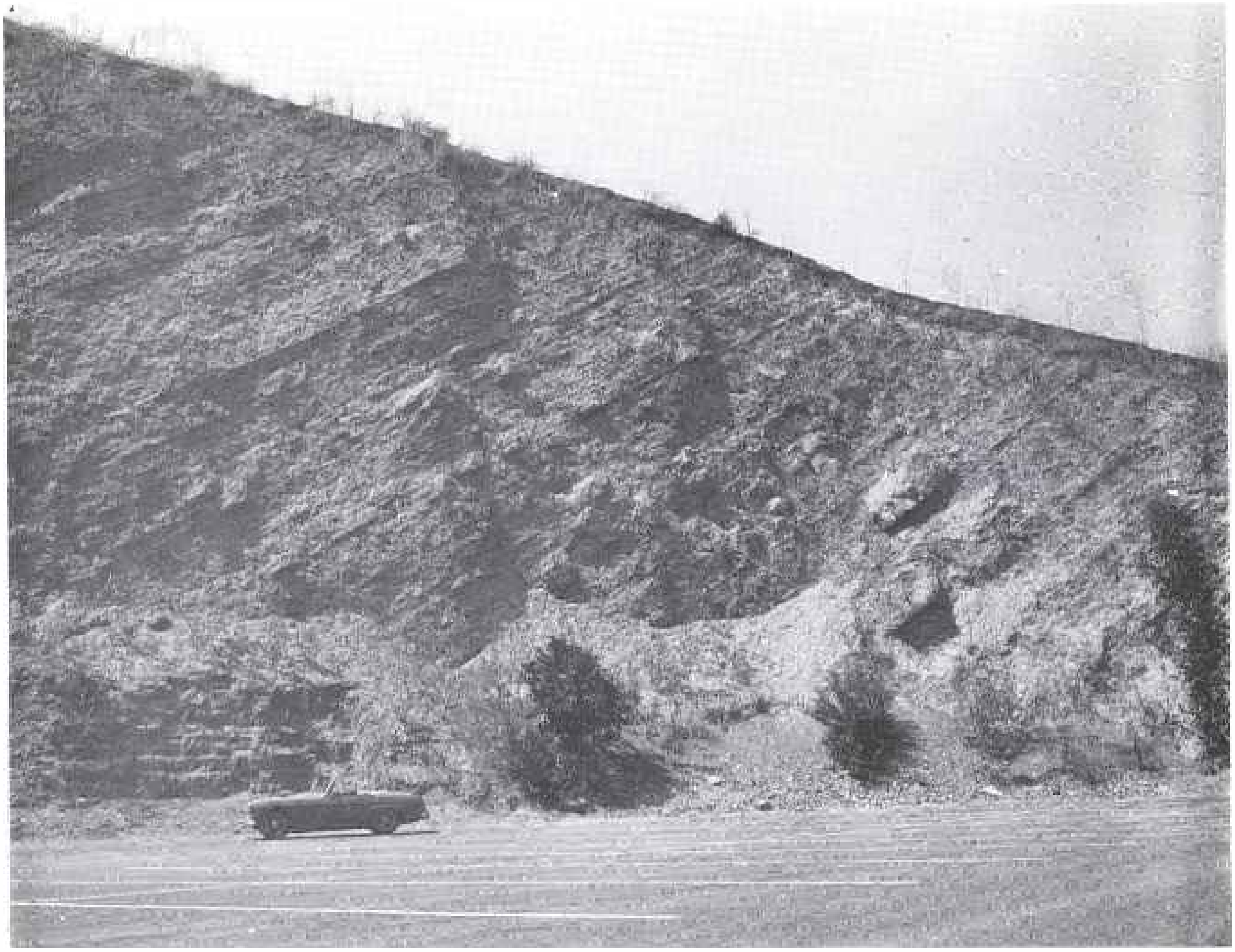


Figure 4-5

Puente Formation
Thin bedded claystone, siltstone and sandstone

Ref: Lamar (1970), p. 25.

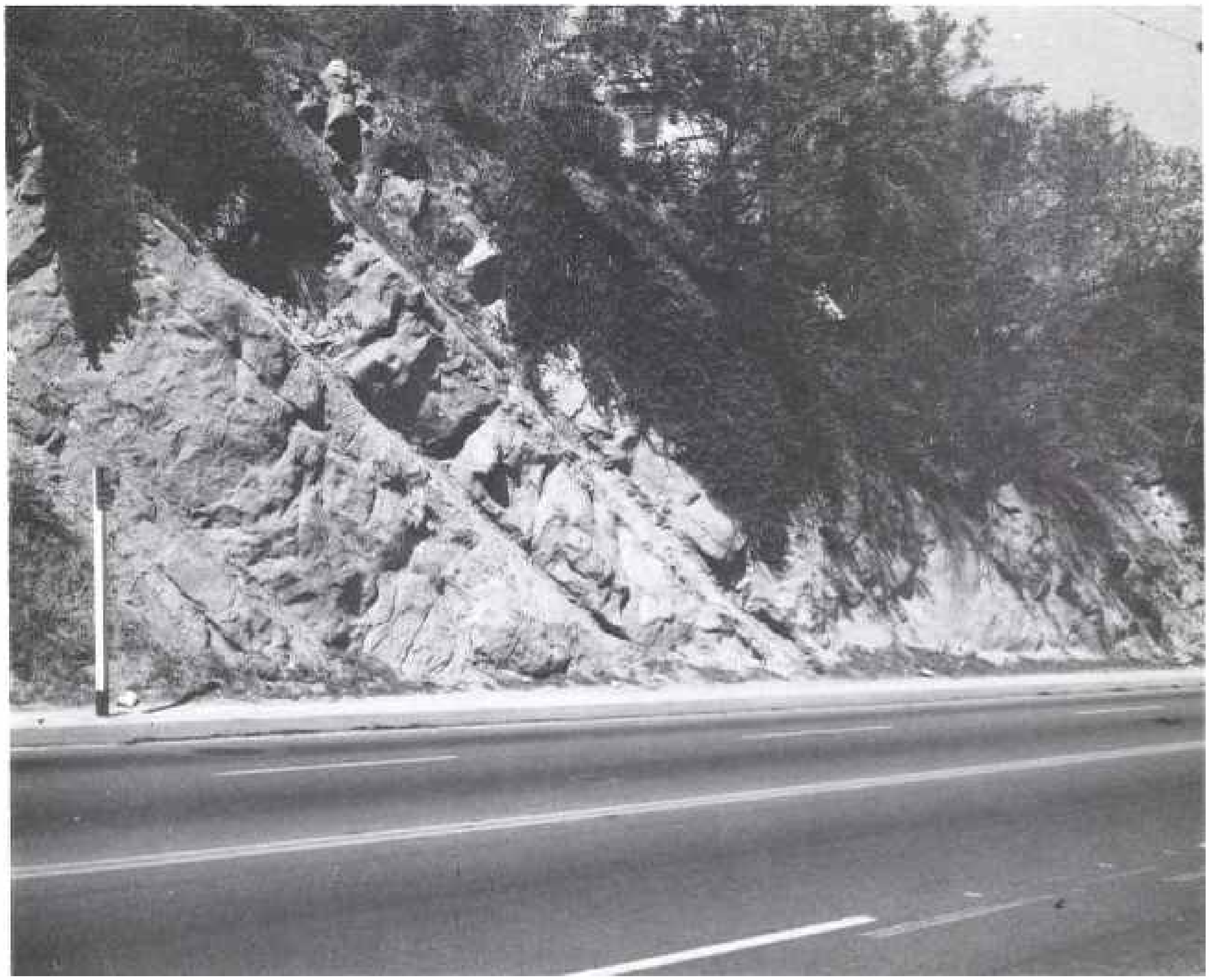


Figure 4-6

Puente Formation
Thickly bedded sandstone and interbedded siltstone

Ref: Lamar (1970), p. 26.

Figure 4-7

Topanga Formation
Alternating thick and thin bedded conglomerate, sandstone and siltstone

Ref: Lamar (1970), p. 17.

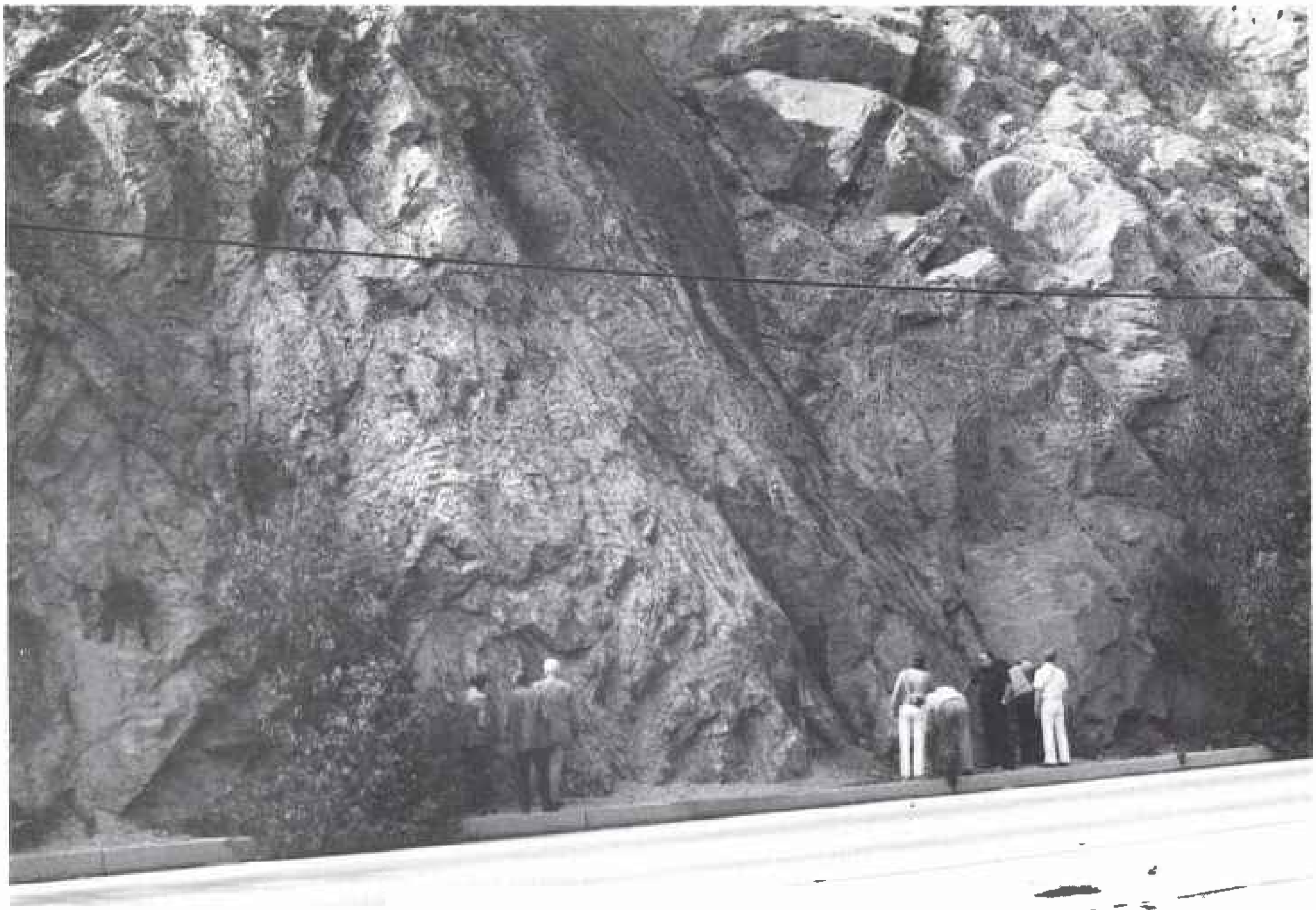


Figure 4-8

Topanga Formation
Hard, massively bedded sandstone.
Cahuenga Pass, north of Boring CEG 31.
Note the basalt intrusion between sandstone beds.

#### 4.3.12 Topanga Formation (Basalt)

The basalt rocks of the Topanga Formation of Middle Miocene age, designated Terzaghi Rock Condition Numbers 1 - 5 inclusive on Drawing 2, have intruded the sedimentary sequence in Reach 7 (Drawing 2). About 7,000 feet of this hard and intact, to massive moderately jointed rock, will be encountered along the alignment. The areal distribution is shown as Tb on Drawing 1. Figure 4-8 is an illustration of the gray basalt intruded along hard sandstone beds at Cahuenga Pass. The intrusion has baked the conglomerates and sandstone. Petrographic analyses of cores obtained from Borings CEG 31 and 32 indicate that basalt is the principal rock. However, less than 5 percent of the cores analyzed contain of rocks classified as dolerite and amygdalodial andesitic-basalt. Fractures in the basalt are known to store ground water in sufficient quantity to permit temporary inflows into any tunnel construction. Inflows may exceed 600 gpm over a period of a few days, judging from experience during construction of the near-by existing Los Angeles City Sewer Tunnel (Section 6.3 and Drawing 1).

## 4.4 GEOLOGIC STRUCTURAL FEATURES

Most of the major geologic structural features along the proposed alignment area trend east-west.

### 4.4.1 Major Folds

The proposed Metro Rail alignment traverses near or over several major folds (Drawing 1). In sequence, from downtown Los Angeles to North Hollywood, these folds are:

- Los Angeles Anticline
- Three Salt Lake Oil Field Folds
- Hollywood Syncline
- Santa Monica Mountain Anticline
- San Fernando Valley Synclinorium.

#### 4.4.1.1 Los Angeles Anticline

The Los Angeles anticline (upfold), a major structure trending about N70W, influences the dip of strata for the first 5 miles of the proposed alignment (Drawing 1). The Los Angeles anticline also coincides with the Los Angeles Oil Field. The major portion of the oil field is located north of the proposed alignment.

#### 4.4.1.2 Salt Lake Oil Field Folds

Three other N70W folds occur in the vicinity of the Salt Lake Oil Field near the intersection of Wilshire Boulevard and Fairfax Avenue (Drawing 1). The La Brea Tar Pits occur in the alluvium overlying the more southerly of these folds.

#### 4.4.1.3 Hollywood Syncline

The axis of the Hollywood syncline (downfold) trends east-west and roughly parallels the alignment from the intersection of Fairfax and Fountain avenues to the intersection of Sunset and Cahuenga boulevards (Drawing 1). The syncline defines the Hollywood ground water basin.

#### 4.4.1.4 Santa Monica Mountain Anticline

The Santa Monica Mountains are structurally an anticline with an east-west axis plunging to the west. Throughout late Pleistocene to recent time, this rapidly rising anticline has produced large amounts of debris which have pushed the Pacific Ocean shoreline west and south and formed interfingering marine and nonmarine deposits.

#### 4.4.1.5 San Fernando Valley Basin

The San Fernando Valley is an asymmetric synclinorium (basin) developed chiefly in Miocene and younger rocks that have been deformed by late Quaternary folding and faulting, especially at the northern margin, and by thrusting along the Santa Susana and Sierra Madre faults.

### 4.4.2 Faults

The proposed Metro Rail alignment crosses several faults (Drawing 1). These faults are listed below in the sequence they occur along the alignment from downtown Los Angeles to North Hollywood.

- MacArthur Park fault
- 6th Street fault
- 3rd Street fault
- San Vicente fault
- Santa Monica fault (zone)
- Hollywood fault
- Hollywood Bowl fault
- Unnamed fault (north of Boring CEG 32)
- Unnamed fault (north of Boring CEG 32A)
- Benedict Canyon fault
- Unnamed fault (north of Boring CEG 36)
- Unnamed fault (north of Boring CEG 38).

#### 4.4.2.1 MacArthur Park Fault

The MacArthur Park fault, east side down (Drawing 2) relative to the west side (near-vertical fault), is inferred in the Puente Formation (Lamar, 1970). This fault is not known to be active or potentially active. Neither the physical condition nor the width of the fault is known. Since the fault trace crosses the alignment at right angles, it would not follow any excavation (Drawing 1). Artesian flow from Boring CEG 11 may indicate the fault is a barrier to ground water, as well as a trap for gas and oil. The highly saline water contains 19,670 total dissolved solids (sea water is about 35,000 ppm), suggesting an origin deep in an oil-bearing formation.

#### 4.4.2.2 6th Street Fault

This fault is near-vertical with north side up relative to the south side and is in the Fernando Formation (Drawing 2). The fault location (Drawing 1) is based on Salt Lake Oil Field data (Crowder, 1961). It is not known to be active or potentially active, but it is probably a trap for gas and oil migration. During our seismic profiling, two anomalies commonly associated with faulting were observed in the area underlying Lines S-38 and S-39 (see Appendix C, Volume II: Figure C-25 for location, and Figures C-6 and C-7 for results). These anomalies probably represent the 6th Street fault from 60 to 80 feet below the ground surface. The fault is crossed twice by the alignment, but if the track grade is no more than about 80 feet deep, the fault should not be penetrated. The fault is judged not to penetrate the San Pedro sand or Old Alluvium overlying the Fernando Formation (Drawing 2). The physical properties in and adjoining the fault are not known.

#### 4.4.2.3 3rd Street Fault

Displacement on this fault is north side up relative to the south side and is in the Fernando Formation (Drawing 2). This fault is also located based on Salt Lake Oil Field data (Crowder, 1961) and is not known to be active or potentially active. Neither the physical condition nor the width of the fault is known, but the fault is likely a trap for gas and oil. The fault trace crosses the alignment at nearly right angles by Boring CEG 22, thus would not follow any excavation for more than a few tens of feet (Drawing 1). The fault should not be encountered if track grade is less than about 140 feet deep.

#### 4.4.2.4 San Vicente Fault

This fault is also north side up relative to the south side (Drawing 2). The fault location is based on Salt Lake Oil Field data (Crowder, 1961) and is in the Fernando Formation (Drawing 1). This fault is not known to be active or potentially active. Neither the physical condition nor the width of the fault is known, but the fault is likely a trap for gas and oil. The fault trace crosses the alignment at about a 45° angle near Boring CEG 23 (Drawing 1). The fault should not be encountered if track grade is less than about 200 feet below the ground surface.

#### 4.4.2.5 Santa Monica Fault (zone)

The near-surface location of the Santa Monica fault zone is not well defined. The location shown on Drawing 1 is based on oil well and water well data, CEG borings, seismic profiles, gravity data and ground water information. Interpretation of gravity survey (Figure D-3, Appendix D, Volume II) suggests a location at Fairfax High School near Boring CEG 23A but could not conclusively confirm location of this fault. However, Gravity Profile 4 (Figure D-4, Appendix D, Volume II) near Boring CEG 27 appears to have located the Santa Monica fault in the Hollywood area; i.e., about 150 feet of vertical offset along a 50° north-dipping reverse fault (north side up) with bedrock thrust over Old Alluvium. The fault is judged to be potentially active, and the fault trace crosses the proposed alignment at an angle of about 35° near the intersection of Melrose and Fairfax avenues (Drawing 1). Based on our interpretation, the Santa Monica fault displaces Old Alluvium ($A_4$), as illustrated on Drawing 2. A summary of information and opinions reported by other investigators not associated with the Metro Rail Project is as follows:

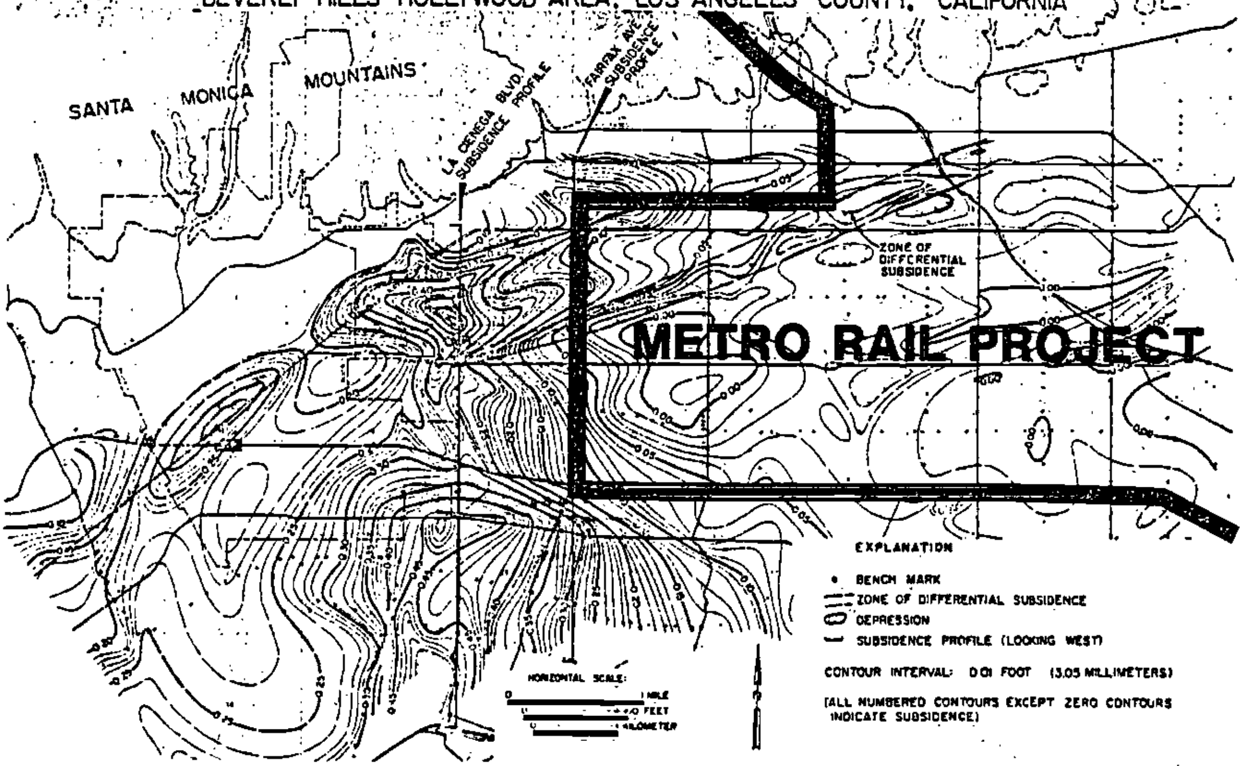
- The near-surface location of the Santa Monica fault in the Beverly Hills-Hollywood area is defined by a zone of differential subsidence (Figure 4-9), coincident with a ground water barrier in the Pleistocene sediments (Figure 4-10). This interpretation implies that movement along the Santa Monica fault extended into part of the Pleistocene. Holocene movement [11,000 ybp (years before present)] cannot be precluded on the basis of current knowledge, and based on micro-earthquake activity, the Santa Monica fault appears to be actively undergoing strain accumulation and release (Hill, 1979, pp. A-3, B-4 and B-11).
- The Santa Monica fault is a distinctly separate structural feature from the Hollywood fault; i.e., the Hollywood fault lies at the base of the Santa Monica Mountains and is separated from the Santa Monica fault by the Hollywood syncline (Drawing 1). The Santa Monica fault is also distinctly separate from the Raymond fault, although it is on the same trend (Converse, 1972).
- The basement surface is upthrown on the north more than 7,500 feet. The base of the upper Miocene is upthrown about 6,500 feet. The base of the lower Pliocene is upthrown about 3,000 feet. Left-lateral offset is also suggested (Yerkes, 1965, p. A51).

#### 4.4.2.6 Hollywood Fault

The Hollywood fault is located at the base of the Santa Monica Mountains (Drawings 1 and 2). The proposed alignment will penetrate this zone, which apparently has two branches in the Cahuenga Pass: the Hollywood fault between Borings CEG 28 and 28A and Gravity Profile, Figure D-5, Appendix D, Volume II; and the Hollywood Bowl fault at Borings CEG 30 and 31 and Gravity Profile Figure D-6, Appendix D, Volume II (see Hollywood Bowl fault description in subsection 4.4.2.7). The Hollywood fault is judged to be active based on interpretation of Borings CEG 28 and 28A (Drawings 1 and 2), Seismic Refraction Survey Area 4 (Drawing 3) and micro-gravity profile 5 (Figure D-5, Appendix D, Volume II), the principal reason being the apparent 270 feet of vertical displacement (north side up) of Young and Old Alluvium (Drawing 2). The fault and/or fault zone should have minor influence relative to broken rock and/or ground water inflows on the anticipated mixed-face tunneling conditions. Opinions by other investigators, not associated with the Metro Rail Project, are:

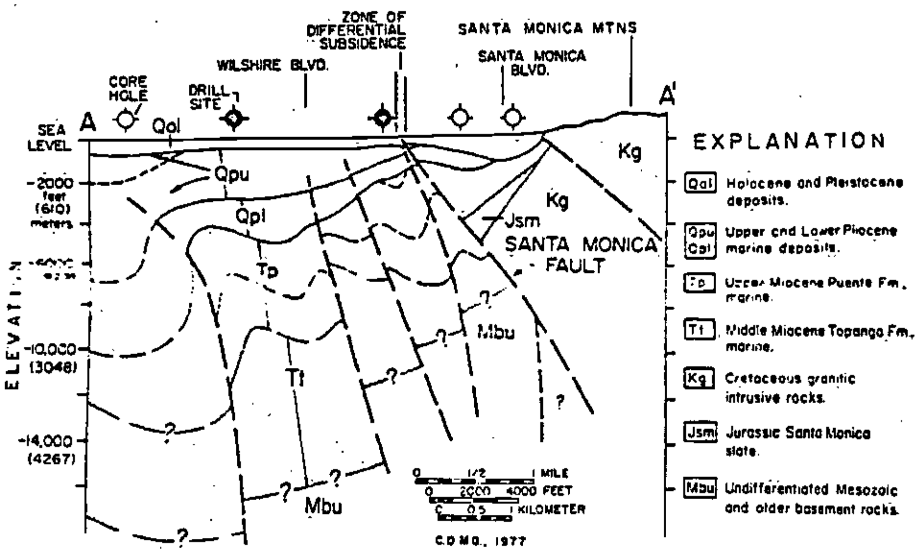
- Based on geomorphic evidence, a fault is present along the south edge of the Santa Monica Mountains and caused post-Pleistocene uplift of the Santa Monica Mountains causing tilting of the Santa Monica Plain (Hoots, 1930).

TOTAL ELEVATION CHANGE (IN FEET) BETWEEN 1955 AND 1970 LEVELING SURVEYS
BEVERLY HILLS-HOLLYWOOD AREA, LOS ANGELES COUNTY, CALIFORNIA




REFERENCE:

Hill and others, 1979, p. B-25




GENERALIZED CROSS SECTION ALONG LA CIENEGA BLVD. (LOOKING WEST)

REFERENCE:

Hill and others, 1979, p. B-32

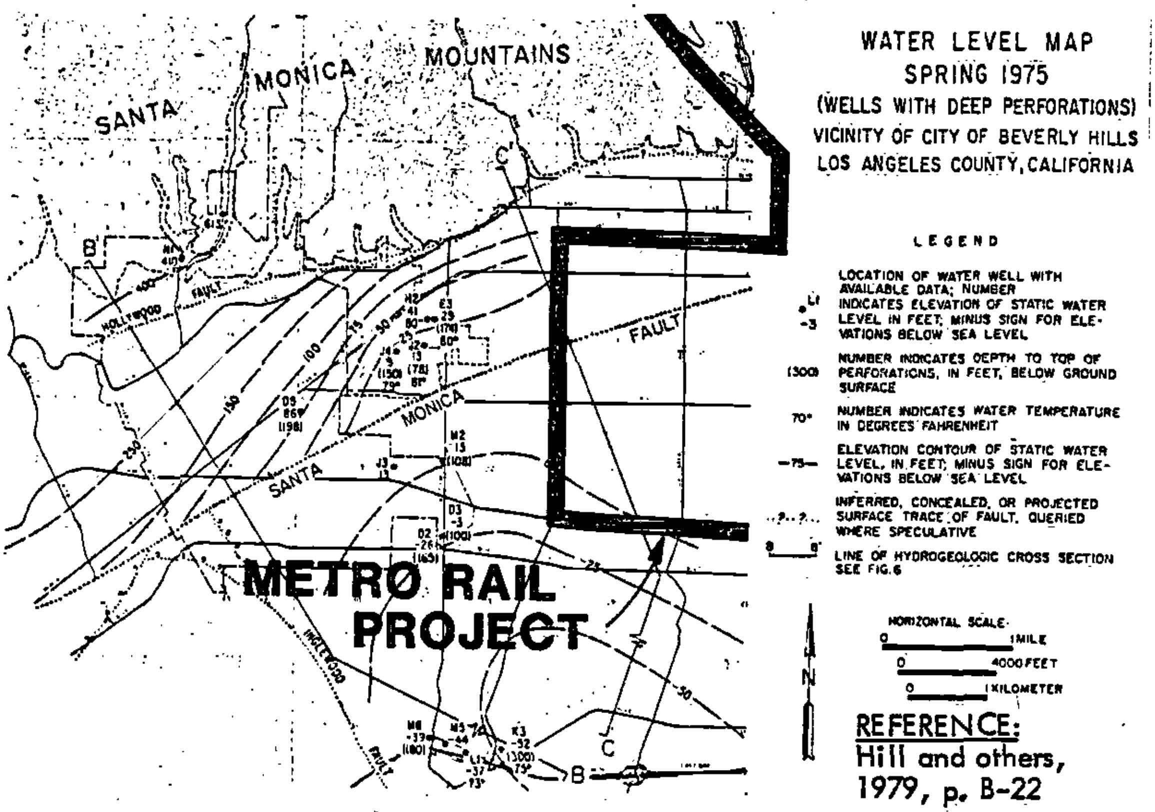
PLAN AND SECTION OF ZONE OF DIFFERENTIAL SUBSIDENCE ALONG SANTA MONICA FAULT

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 4-9




SANTA MONICA FAULT AS A GROUND WATER BARRIER

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 4-10

- Based on offset alluvial sediments and other geologic evidence, the Hollywood fault is judged to have been active during very late Quaternary (including Holocene) time (Weber, 1980, p. A-3).
- East of the Los Angeles River, in the Atwater area of Los Angeles, a series of gentle south-facing breaks in slope 2 to 3 m in height apparently represent scarps along the principal, most recently active trace of the zone. The Hollywood fault zone is primarily expressed at the ground surface by scarp-like features in older and younger alluvial deposits. These sediments, lying at a depth of about 35 meters on the north (upthrown) side, are displaced downward about 35 meters on the south side (Weber, and others, 1980, p. B-58).
- There is no subsurface evidence that the Hollywood fault crosses the Los Angeles River alluvium in the Atwater area, judging from a 1-mile diameter, continuous undisplaced clay layer located about 100 feet below the ground surface (defined by 60 borings). The clay layer is believed to be ±30,000 years old, according to paleoclimatic age-dating of redwood tree remnants obtained in the clay layer (Converse Davis Associates, 1972).
- The Hollywood fault is classified as potentially active (no recognized historic activity, but may move again in the near future). (Yerkes, and others, 1977, p. 7).
- Data from wells drilled north of Beverly Hills indicate the existence of a number of a north-dipping fault zones with a minimum of 1,500 feet of vertical separation at the base of the Modelo Formation (Lamar, 1970, p. 38).
- The Hollywood fault extends along the southern edge of the Santa Monica Mountains and at depth serves as the northerly edge of the Hollywood Basin, one of the ground water basins included in the Coastal Plain. The Hollywood fault truncates the northern flank of the Hollywood syncline and is a complete barrier to ground water movement to the north and east (California Department of Water Resources Bulletin 104, 1961, p. 88 and 95).

4.4.2.7 <u>Hollywood Bowl Fault</u>

The Hollywood Bowl fault was encountered in Borings CEG 31 and 32 and is interpreted to be present in Gravity Profile 6 (Figure D-6, Appendix D, Volume II) and seismic line S-51 (Appendix C, Volume II: Figure C-16 for interpretation and Figure C-28 for location). This fault does not appear to have offset alluvial deposits and is steeply dipping (±80°), with the north side displaced upward relative to the south side (see Drawing 1 for location and Drawing 2 for interpretation). The amount of displacement and the age of last displacement is unknown. However, the fault is not known to be an active or potentially active fault. The fault is judged to represent a zone of discontinuities several hundreds of feet wide and be a likely source of ground water inflow to any tunnel construction.

4.4.2.8 <u>Unnamed Fault North of Boring CEG 32</u>

An unnamed fault occurs at the contact of Topanga Formation sandstone and basalt at the location shown on Drawing 1. This fault, mapped by Hoots (1930), is nearly vertical, with the north side down relative to the south side (Drawing 2). The fault is not known to be active or potentially active. The fault trace crosses the alignment at nearly right angles and should not follow any excavation for more than a few tens of feet, but it is likely to be encountered. MWD's 1940 Hollywood Tunnel (Section 6.4), encountered a moderately blocky and seamy area about 80 feet wide and a two-day inflow of 600 gpm at this fault location.

#### 4.4.2.9 Unnamed Fault North of Boring CEG 32A

An unnamed fault in the near-vertical dipping Topanga sandstone and conglomerate is inferred from interpretation of aerial photographs, faulting encountered in the Los Angeles City Sewer Tunnel (Section 6.3), Hoots' 1930 Geologic Map, and surface geomorphic expression (Drawing 1). This fault may be encountered in the alignment. The Los Angeles City Sewer Tunnel, during construction, encountered a maximum inflow of about 200 gallons per minute (for a few days) from this fault contact area. "Heavy" ground pressures were reported in the Los Angeles City Sewer Tunnel in this area. The fault is not known to be active or potentially active. The fault trace crosses the alignment at nearly right angles and should not follow the project line for more than a few tens of feet. The north side is down relative to the south side in this near-vertical fault (Drawing 2).

#### 4.4.2.10 Benedict Canyon Fault

The proposed alignment could penetrate the Benedict Canyon fault if track grade is deeper than 40 feet below the ground surface (Drawing 2). The fault is not known to be active or potentially active. The location of the fault is based on topographic expression on the north flank of the Santa Monica Mountains and confirmed by our seismic profiling (Appendix C, Volume II: Figure C-18, seismic line 28 for interpretation, and Figure C-29 for location). The Benedict Canyon fault location, as mapped by Hoots (1930), cuts diagonally across the Santa Monica Mountains northwest of Beverly Hills. However, according to Hoots, the fault terminates west of the proposed alignment. The fault has been projected northeastward across the alignment (Drawing 1), based on works of others (Los Angeles City Geologic Map Sheet No. 94, 1970; Hill, 1979; and Weber, 1980). Tertiary movement along this fault zone appears to have resulted in a horizontal offset of several stratigraphic units for a distance of approximately 1.5 miles (Hoots, 1930). The fault shows nearly 2.5 km of left-lateral slip separation at the contact between upper Cretaceous rocks and Paleocene rocks (Weber, 1980). Gravity gradients suggest a zone of steep north-facing gradients that possibly express two faults, the more southerly being the Benedict Canyon (Weber, 1980), p. B-52). Gravity data indicates that rocks along both faults are down relatively on the north, which is compatible with geologic evidence to the west in the mountains (Weber, 1980, p. B-52). If projected eastward near the abrupt bend from east to south of the Los Angeles River, water-well data suggests that the bottom of the alluvial basin is displaced downward 170 meters on the north side (California Water Rights Board, 1960 Cross-section M-M' on Plate 5E, and p. 111-7 to 111-8).

#### 4.4.2.11 Unnamed Fault North of Boring CEG 36

The location (Drawing 1) of this postulated fault is based on surveyed elevation change data along the south edge of the San Fernando Valley, as interpreted by J.H. Bennett (Weber, 1980, p. B-99, and Plate I), suggesting an east-trending fault in the vicinity of the Ventura Freeway. The fault is not known to be active or potentially active, nor act as a ground water barrier. This postulated fault is expected to have little or no effect on the Metro Rail Project.

#### 4.4.2.12 Unnamed Fault North of Boring CEG 38

The location (Drawing 1) of this postulated fault is based on an apparent east-northeast trending, south-facing, linear break in topography discernible on USGS quadrangle maps published in 1901 and 1926, suggesting a possible fault. In addition, elevation change data, by J.H. Bennett (Weber, 1980, p. B-99, and Plate I) suggest a zone of subsidence to the south. These relationships suggest that youthful deposits of Tujunga Wash may be offset downward relatively to the south in recent time (Weber, 1980). The fault is not known to be active or potentially active, nor to act as a ground water barrier. This postulated fault is expected to have little or no effect on the Metro Rail Project.

## 4.5 GROUND WATER BASINS

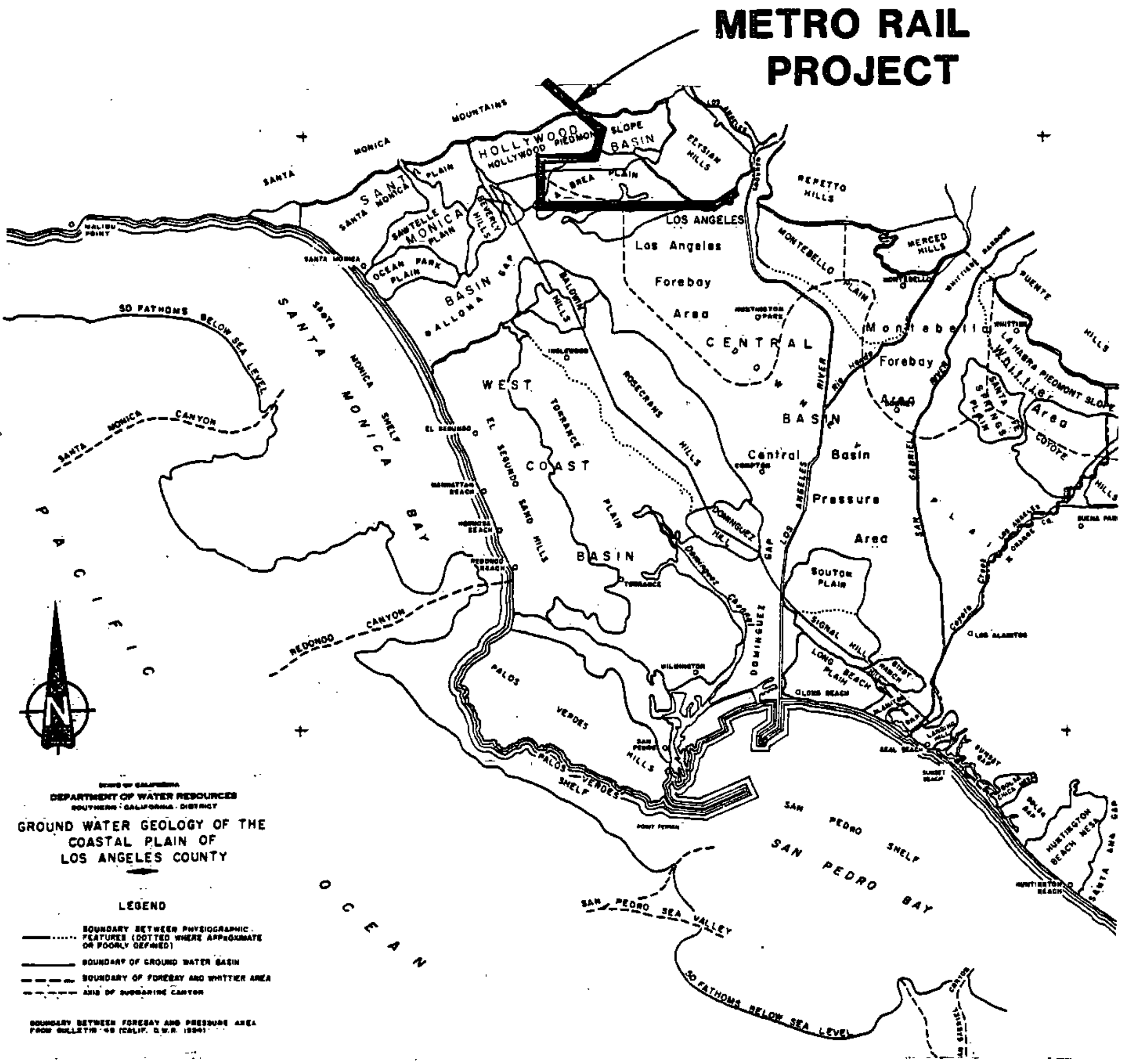
### 4.5.1 General

The proposed alignment will traverse four hydrologic units, each having distinct characteristics with respect to storage and transmission of ground water. Three of these units are considered ground water basins. The fourth is the Santa Monica Mountain mass (Figure 4-11). These units, starting from downtown Los Angeles to North Hollywood, are:

- Los Angeles Forebay Area (Central Basin)
- Hollywood Basin
- Santa Monica Mountains
- San Fernando Valley Basin (Drawing 1).

### 4.5.2 Los Angeles Forebay Area

The Los Angeles Forebay area is in the Central Ground Water Basin, extending southerly and westerly in an irregular semi-circular fashion from the mouth of the Los Angeles Narrows near downtown Los Angeles (Figure 4-11). The Forebay area includes the area traversed by the proposed alignment from downtown Los Angeles to the Hollywood Basin. The term "forebay" refers to an intake area where substantial infiltration of surface water into the basin can occur. This concept is a gross simplification. Several aquicludes of sufficiently low transmissivity occur locally, permitting perched ground water conditions similar to those shown on Drawing 2 along the alignment. Where the aquiclude




REFERENCE:

California Department of Water Resources, 1961, Plate 2

MAP OF COASTAL GROUND WATER BASINS

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Figure No. 4-11

is missing, the aquifers are in direct hydraulic continuity with the surface. Ground water occurs in Young Alluvium and Old Alluvium and other underlying pervious Pleistocene sediments. The known water-bearing sediments extend to depths of 1,600 feet below the ground surface in the southern parts of the Forebay. The Tertiary sedimentary rocks beneath the basin are essentially nonwater-bearing. Coastal Plain ground water contours for shallow aquifers, November 1973, are shown on Figure 4-12.

### 4.5.3 Hollywood Basin

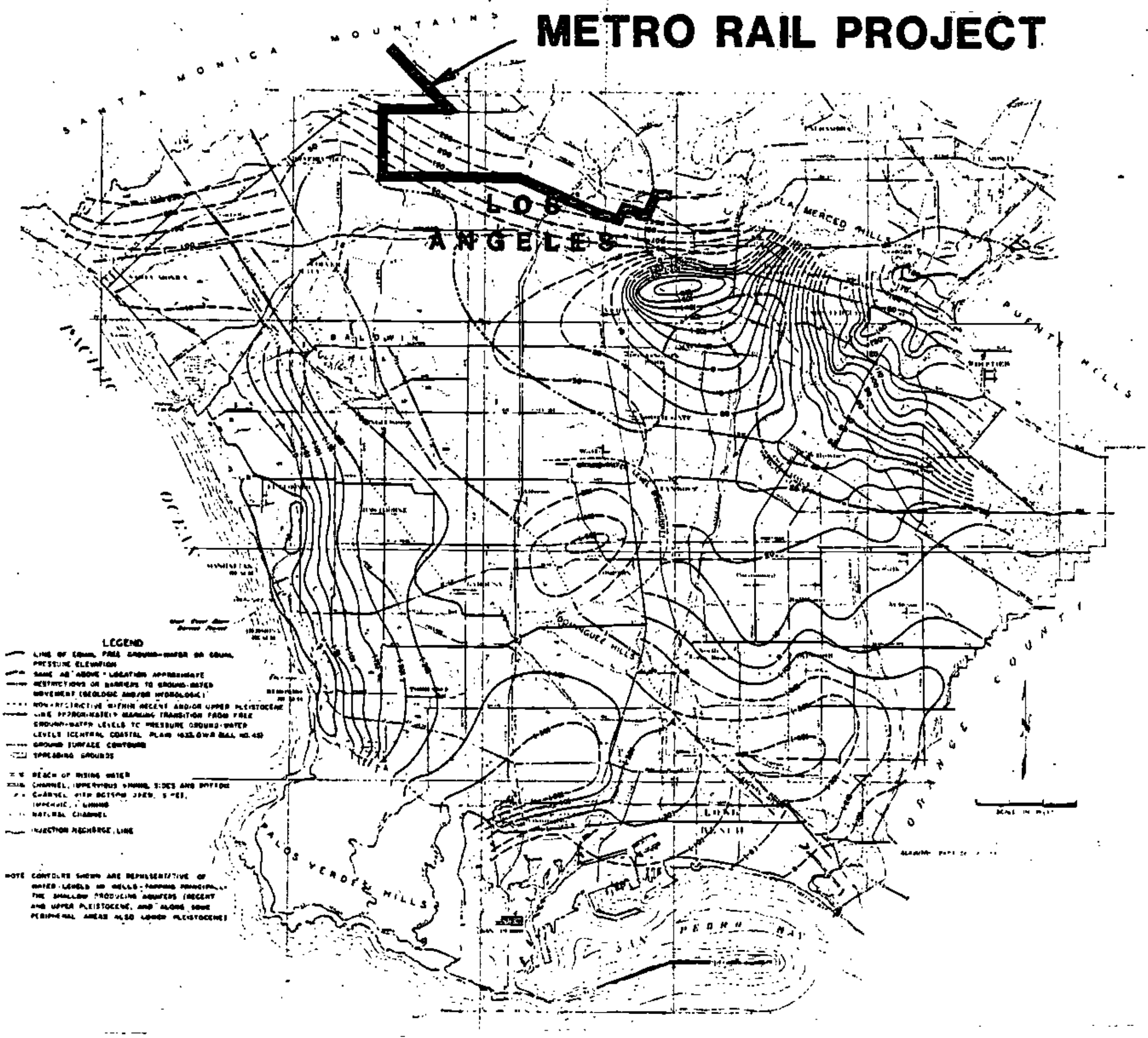
This basin extends from the southern margin of the Santa Monica Mountains southerly to the Santa Monica fault (Figure 4-10). Many water wells were present in the Hollywood Basin around the turn of the century, but most of these have since been destroyed as land use has changed. Most of the water wells were located in the deeper portions of the basin corresponding to the Hollywood synclinal axis near the Santa Monica Mountains (Drawing 1). Sediments containing known aquifers extend to a maximum depth of 650 feet and include alluvium and Pleistocene sediments. In general, aquifers in the Hollywood Basin possess relatively low transmissivity rates. A zone of differential subsidence, coincident with the Santa Monica fault, on the south side of the Basin (Figure 4-9) is attributed, in large part, to ground water withdrawals. This subsidence is judged not to impact the Metro Rail Project, provided there is no more heavy pumping and attendant water level declines. Coastal Plain ground water contours, November 1973, are illustrated on Figure 4-12.

### 4.5.4 Santa Monica Mountains

This mountain range does not constitute a ground water basin, but rather a mass of Tertiary sedimentary (Tt) and volcanic (Tb) rocks (Drawing 1) and other older crystalline rocks with a limited capacity for transmission of water. The term "nonwater-bearing" has been used by others but is meant to imply that these materials yield relatively limited quantities of water to wells, not that the materials contain no water. Wells that intersect extensive joint and fracture systems can produce ground water in fairly sizable quantities for short periods of time. Such joint and fracture systems are significant in tunneling. A case in point occurred, during construction of MWD's Hollywood Tunnel in 1941 (Section 6.4). At that time, flash flows of up to approximately 600 gpm were encountered, lasting for a few hours (MWD, 1942). Water pressure tests in Borings CEG 30 and 31 indicated that the bedrock in these locations is highly permeable. Pressure tests completed in borings that encountered geologic Unit C show the bedrock to be relatively impermeable (see Table A-4, Appendix A, Volume II).

### 4.5.5 San Fernando Valley Basin

This basin lies on the north side of the Santa Monica Mountains. In this basin, ground water occurs chiefly in the Young and Old Alluvium that, in places, reach depths of 1,000 feet. In this area the water-bearing sediments




REFERENCE:

Los Angeles County Flood Control District
1973-74 HYDROLOGIC REPORT, 1975, p. 275

COASTAL PLAIN GROUND WATER CONTOURS - SHALLOW AQUIFERS - NOVEMBER 1973

| | |
|---|---|
| Southern California Rapid Transit District<br>METRO RAIL PROJECT | Project No.<br>80-1280 |
| CWDD/ESA/GRC Geotechnical Consultants | Figure No.<br>4-12 |

are about 600 feet thick. These sediments are permeable and freely yield water to wells. In general, water levels in the San Fernando Basin have declined markedly, in some cases 100 feet or more, since the mid-forties in response to heavy pumping. Efforts by both the City of Los Angeles and the Los Angeles County Flood Control District to replenish the basin with imported water seem to have arrested this decline. San Fernando Valley ground water contours, April 1974, are presented on Figure 4-13 (LACFCD, 1975). As shown on Figure 4-13, there is a relatively deep 150-foot ground water depression about 4 1/2 miles east of the Metro Rail Project near the "bend" in the Los Angeles River. A rapid rise in water levels at this depression, due to wet winters or supplementary recharge, is judged to be capable of raising water levels at the project area several tens of feet in a year or two.

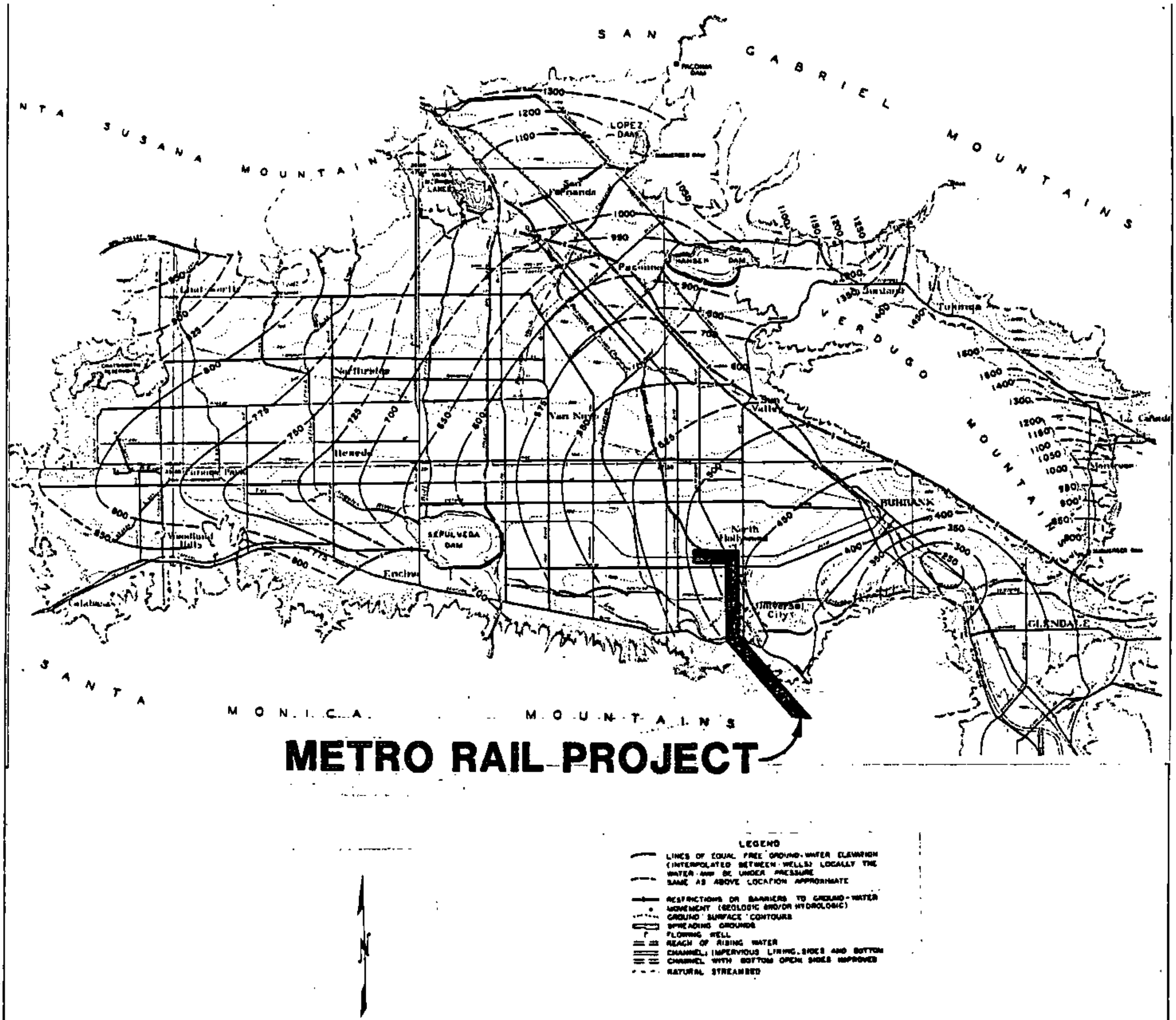
## 4.6 GROUND WATER QUALITY

With very few exceptions, water quality along the alignment is poor (Table G-1, Appendix A, Volume II); i.e., exceeds 500 parts per million (ppm) total dissolved solids (TDS), which is the U.S. Environmental Protection Agency drinking water standard (Todd, 1980). Chloride, sulfate and total dissolved solids contents are very high, as is conductivity. The TDS of the artesian water from Boring CEG 11 is extremely high, i.e. 19,670 ppm, as were waters from Borings CEG 6 (20,230 ppm) and 19 (15,425 ppm). Mineral springs were common in the Hollywood area at the turn of the century. Above-normal concentrations of certain ions are to be expected where ground water is associated with oil and gas. However, high total dissolved solids (TDS) were also encountered in the sulfate-type water in the San Fernando Valley, ranging from a low of 732 ppm in Boring CEG 36 to a high of 2605 ppm in Boring CEG 35; averaging about 1,000 ppm from seven boring samples. More specific discussion and results of ground water quality analyses are presented in Sections 5.1.6 and 10.6, and Appendix G, Volume II.

## 4.7 OIL AND GAS

### 4.7.1 General

Oil was first discovered in the Los Angeles Basin in 1880, and the Los Angeles City Oil Field was discovered in 1892, based on oil seeps at the surface. Oil is produced chiefly from thick deposits of lower Pliocene and upper Miocene strata. About 58% of recovered oil has come from the lower Pliocene rocks and about 42% from upper Miocene rocks. In relation to area, the Los Angeles Basin is the most prolific of California's oil producing districts and is one of the most prolific in the world. A unique combination of factors and timing of events accounts for the productivity of this basin. The petroliferous sediment accumulated rapidly in stagnant cool water more than 1,600 feet deep during the advancing and maximum phases of the last marine transgression. The initially high organic content of the sediment was preserved because of poor




SAN FERNANDO VALLEY
GROUNDWATER CONTOURS
APRIL 1974

REFERENCE:

Los Angeles County Flood Control District, 1973-74 HYDROLOGIC REPORT, 1975, p. 281

SAN FERNANDO VALLEY GROUND WATER CONTOURS - APRIL 1974

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Figure No.
4-13

circulation in the constricted basin and because of the rapid filling of the basin. Great thicknesses of intercalated source and reservoir rocks include numerous permeable conduits, through which the fluid hydrocarbons were expelled by load compression toward pre-existing and developing structural traps (faults).

The proposed Metro Rail alignment will pass over or near several oil fields enroute from downtown Los Angeles to North Hollywood. The areal distribution of these oil fields is shown on Drawing 1 and they are:

- Union Station Oil Field (1st Street at Alameda)
- Los Angeles Downtown Oil Field (Pico Boulevard at Broadway)
- Los Angeles City Oil Field (Vermont Avenue to Dodger Stadium)
- Western Avenue Oil Field (Beverly Boulevard at Western Avenue)
- Salt Lake Oil Field (La Brea Tar Pits area)
- South Salt Lake Oil Field (Wilshire Boulevard at Fairfax Avenue).

### 4.7.2 Union Station Oil Field

Little is known of this oil field, but it does produce from the Puente Formation at very shallow depths. Oil was observed in Boring CEG 2 less than 15 feet below the ground surface.

### 4.7.3 Los Angeles Downtown Oil Field

This oil field was discovered in 1964 by Standard Oil Company of California, and produces from the Puente Formation. The field is located south of the proposed alignment (south of Borings CEG 8, 9 and 10). Oil was not encountered in these borings, although traces of gas were encountered.

### 4.7.4 Los Angeles City Oil Field

This field was discovered in 1892 and produced more than a million barrels of oil per year for a few years. The northern limit of production is probably controlled by the Los Angeles anticlinal structure (Drawing 1). Shallow production from the Puente Formation is low-gravity oil (12-20° A.P.I), and production parallels the strike of bedding (east-west-trending). The oil field contains shallow accumulations of petroleum, surface seeps, and more than 1,250 wells, only 54 of which were active in 1974 (Yerkes, 1977). Most of the wells, drilled before 1900, were not surveyed or otherwise accurately located, and the ground surface has since been developed for cultural uses. Consequently, no accurate record exists on the location of many of these wells. The east-west trending structure dips southward about 30°, extending to depths of 500 to 1,000 feet below the surface. Gas and seeping oil were encountered in 1976 during construction of the Los Angeles County Flood Control District's Sacatella Tunnel (Section 6.5) near Hoover Street (Drawing 1). Gas was encountered in Borings CEG 10, 11 and 12.

### 4.7.5 Western Avenue Oil Field

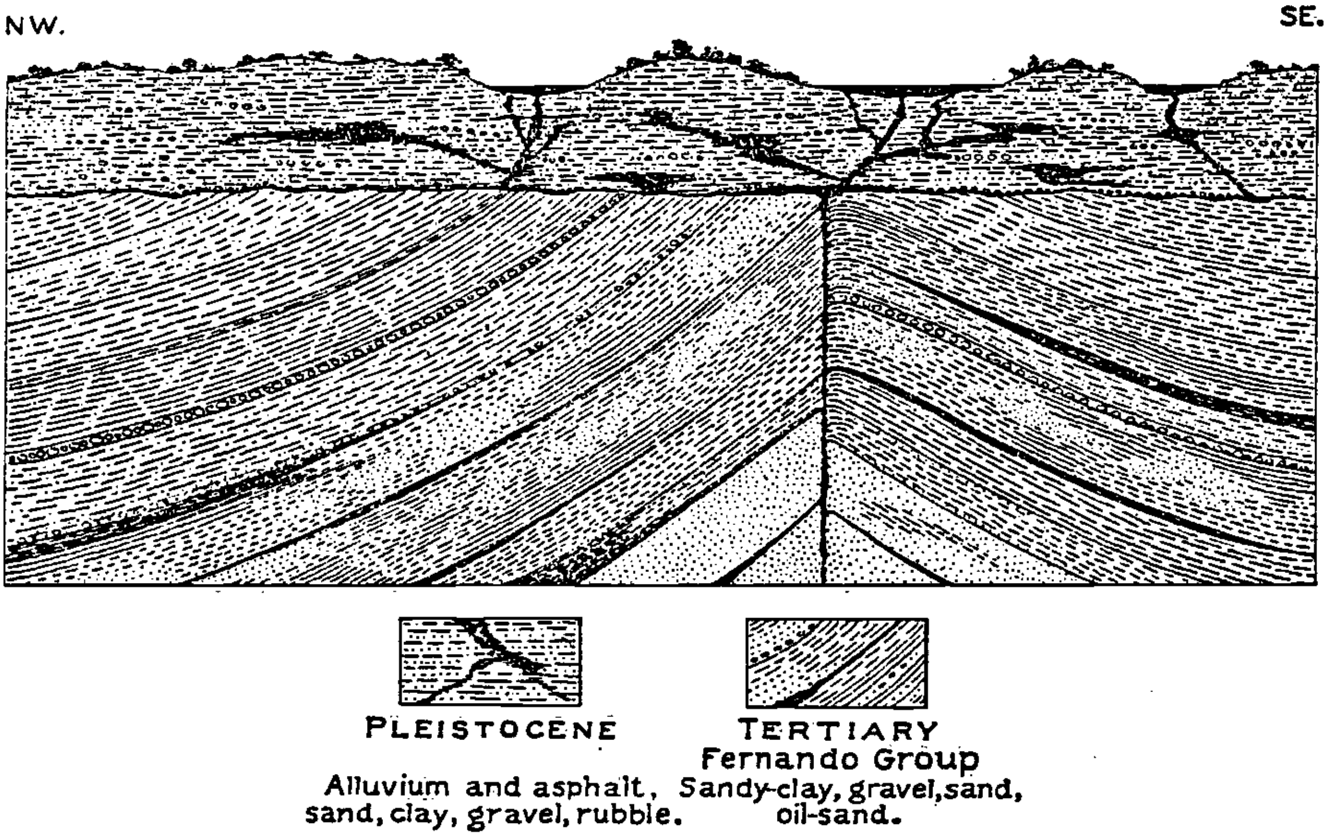
This oil field is apparently a westward extension of the Los Angeles City Oil Field on the south flank of the Los Angeles anticline (Drawing 1). Little is known of this oil field, but its distance from the proposed Metro Rail alignment (approximately one mile north of Boring CEG 15), does not preclude the likelihood that gas can migrate southward beneath the water table (Appendix F, Volume II).

### 4.7.6 Salt Lake Oil Field

This field was first developed in 1903, but was long known by large seeps of heavy oil on the north side of Wilshire Boulevard. The tar around the seeps was mined in pits in the early days and yielded the famous Rancho La Brea fauna of Pleistocene vertebrates. This field was created by anticline and synclinal structures (Drawing 1) in concert with several minor faults--6th Street, 3rd Street, and San Vicente (Crowder, 1961). Tar, oil and gas are present in the underlying Fernando Formation as well as the overlying San Pedro Formation and alluvial deposits, as testified by Borings CEG 19, 20, 21, 22, 23 and 23A. Gas was not encountered north of Boring CEG 23A. The possibility exists that the project excavations could encounter abandoned oil well casings.

An early diagrammatic cross section (Stock, 1946) shows geologic structure and relationships of formations at La Brea Tar Pits during late Pleistocene time (Figure 4-14). A more recent study (Woodward and Marcus, 1973) concludes that animal and plant remains at La Brea Tar Pits are chiefly confined to the uppermost of three well-defined sedimentary units. All appear to be of late Pleistocene age, with fossil bone deposits corresponding to Wisconsin age or younger. The outpourings of tar from vents or fissures occurred during successive stages of alluviation accompanying build-up of the Santa Monica Plain. Location of petroliferous and gassy alluvium, San Pedro Formation and Fernando Formation, relative to the Metro Rail alignment, is presented on Drawing 2 at location of Borings CEG 19, 20, 21, 22 and 23.

The source rocks from which the petroleum comes are the oil sands interstratified with the older shales and sandstones of the Fernando Formation that underlie the Pleistocene beds in the region of Rancho La Brea. As determined by the geologic structure (Figure 4-14) in the Salt Lake Oil Field, these older marine strata are deformed and folded. Immediately north of Rancho La Brea, an upward flexure of the older rocks, whose crest has been broken, extends apparently in the NW-SE direction and, without much question, facilitates the upward movement of gas and oil through vents in the vicinity of the asphalt beds. Subsidiary structures, e.g., small faults, local fractures or a minor fold developed in the Fernando Formation, may account for the apparent localization of tar. Presumably, the exudation of the petroleum and penetration of the sedimentary strata, forming the tar pools and asphaltic material, occurred concomitantly with deposition of the Pleistocene alluvial accumulation. At the present time, oil and gas reach the surface through small fissures, pipes or chimneys, the oil forming small and generally shallow pools




Cross-section showing geologic structure and relationships of formations at Rancho La Brea during period of miring of Pleistocene animals and plants. Character and structure of the sediments containing the oil sands taken from section in Salt Lake Oil Field, after Arnold (U.S. Geol. Surv. Bull. 309, p. 189, 1907).

REFERENCE:

Stock, 1946, p. 18

LA BREA TAR PITS - DIAGRAMMATIC CROSS SECTION

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Figure No.
4-14

about the vents. Bubbles of gas rise constantly to the surface of the artificial lake, and the outpours of oil spread over and through the soil of the adjacent ground. Occasionally a downward movement of the oil or tar can be discerned at the vents. A temporary release of gas pressure below permits the heavy oil to recede again into the pipe or chimney whence it has exuded. During the Pleistocene period, the exudation of the petroleum was much more extensive than at the present time. The pools of oil thus formed occupied the natural depressions of an irregular land surface and were, on occasion, many square feet in area.

### 4.7.7 South Salt Lake Oil Field

Little is known except that commercial oil production began in this field in 1903. Since 1908, the oil field has produced less than 10 barrels per day from about 100 wells. This decrease is attributed to the need for deeper drilling and the growth of the residential district. This field is considered part of the overall Salt Lake Oil Field and will contribute to oil and gas problems for the proposed Metro Rail alignment.

## 4.8 PROJECT TUNNEL GAS CLASSIFICATIONS

Individual intervals of the alignment have been classified in terms of the relative likelihood of encountering gas, using the classification system contained in the Tunnel Safety Orders issued by the California Division of Industrial Safety. These classes, adopted from California Administrative Code, Title 8, p. 684.18, are as follows:

Nongassy - Applied to intervals where there is little likelihood of encountering gas during the construction of the tunnel.

Potentially gassy - Applied to intervals where there is a possibility of encountering flammable gas or hydrocarbons.

Gassy - Applied to intervals where it is likely gas will be encountered.

Extrahazardous - Applied to intervals if the Division finds that there is a serious danger to the safety of employees.

Based on these criteria and data available at this time, specific intervals of the proposed Metro Rail alignment have been classified. These specific intervals are shown on Drawing 2 and listed below:

- Borings CEG 1 to 4 - gassy
- Borings CEG 4 to 9 - potentially gassy
- Borings CEG 9 to 12 - gassy
- Borings CEG 12 to 18 - potentially gassy
- Borings CEG 18 to WC-8 - gassy
- Borings CEG WC-8 to 38 - nongassy.

These preliminary classifications are considered approximate and are presented to alert the designer to the need to make provisions in the current engineering studies to cope with these areas. Actual conditions are expected to vary from those encountered in borings.

Although not shown on Drawing 2, there is judged to be a potentially "extra-hazardous" area along the proposed Metro Rail alignment in the eastern half of Reach 1 and the entire length of Reach 5.

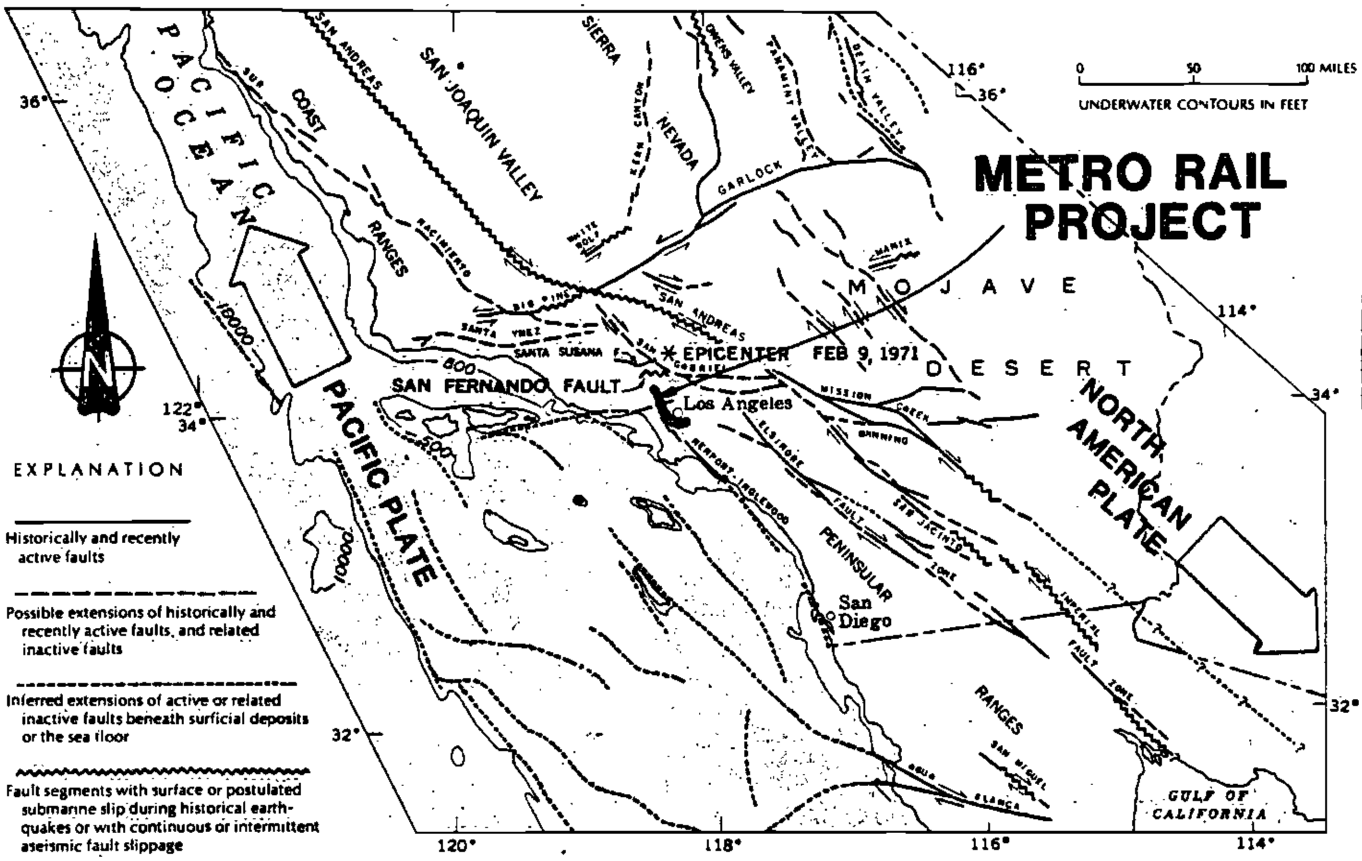
## 4.9 REGIONAL SEISMICITY

The following is a limited discussion on regional seismicity. A detailed discussion is beyond the scope of this report. A detailed Seismological Investigation is being performed and the report will be submitted separately.

California is known as earthquake country, and the Los Angeles area, through which the Metro Rail alignment will traverse, is no exception. California is located within the Circum-Pacific Seismic Belt, along which 80% of the world's earthquakes take place. For at least the past 30 million years, the tectonics of Southern California have been dominated by relative motion between the Pacific and North American tectonic plates (Figure 4-15). The San Andreas fault zone is the boundary between these tectonic plates. The Pacific Plate continues to shift northwesterly with respect to the North American Plate at a rate of about 5.5 cm (2.2 inches) per year. South of the Transverse Ranges, this motion is manifested as right-lateral slip on several mapped, northwest-trending faults. However, the northwesterly plate motion is impeded within the Transverse Ranges and along their north and south flanks, resulting in north-south-oriented tectonic compression, which is manifested as mountain-building and reverse faulting (Figure 4-15). Nearly all tectonic movement in Southern California occurs as seismic fault slippage; i.e., permanent crustal deformation is accompanied by earthquakes which suddenly release gradually accumulated elastic strains. No permanent deformation is known to result from non-seismic (creep) displacements in the Metro Rail alignment area.

Important faults in the project area were identified in Section 4.4.2. Active and potentially active faults in the region are shown on Figure 4-16. Epicentral plots, from 1933 to 1980, for local Richter magnitudes ranging from 3.5 to 6.4 are also included on Figure 4-16.

Some of these are historically active faults; i.e., displaced the ground surface as in the 1971 San Fernando earthquake (M 6.4) which occurred on the east-west-trending Sierra Madre fault zone. Others are seismically active faults that have not displaced the ground surface in historic times; e.g., the 1933 Long Beach earthquake (M 6.3), which occurred on the Newport-Inglewood fault zone.




REFERENCE:

Modified from Wentworth and Yerkes, 1971, USGS Professional Paper No. 733, p. 7.

PACIFIC AND NORTH AMERICAN TECTONIC PLATES AND FAULTS

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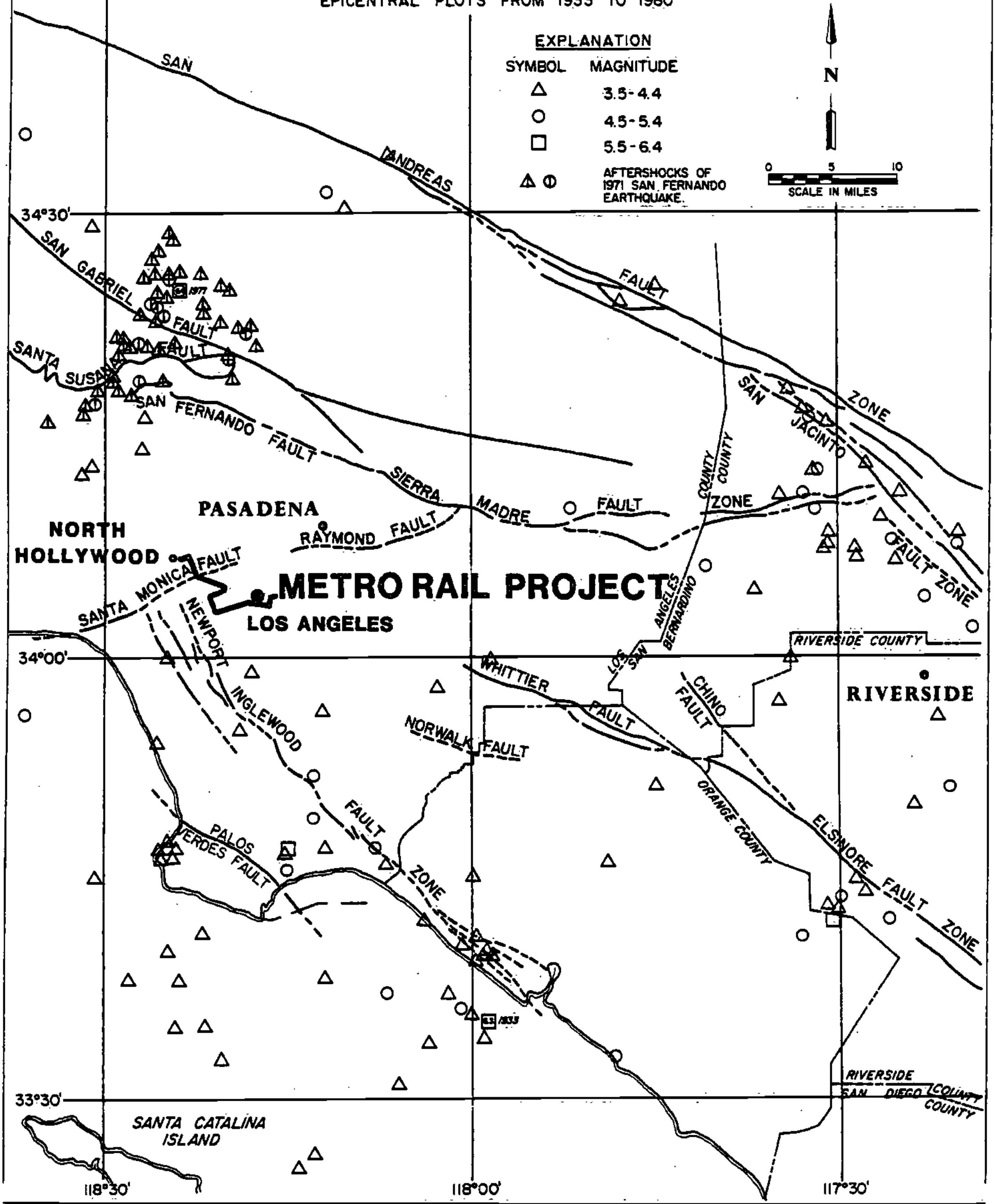
Figure No. 4-15

# EARTHQUAKE EPICENTER AND FAULT MAP

EPICENTRAL PLOTS FROM 1933 TO 1980



# Section 5.0
# Geologic Features of Engineering Significance

# Section 5.0: Geologic Features of Engineering Significance

## 5.1 INTRODUCTION

Engineering geologic features of lithologic units encountered in the investigation program are presented for each formation in specific reaches of the alignment. The proposed Metro Rail Project alignment is divided into eight reaches based on an assessment of the combined geologic, engineering, ground water, gas/oil and surface cultural features. It must be understood that the boundaries of each of the eight reaches are not sharp and must be considered transitional because they are based, in part, on interpretation of subsurface data and, in part, on surface features. These reaches, shown on Drawing 2 are listed below:

- Reach 1 - Downtown, Los Angeles River
  East Portal to the Hollywood Freeway (includes Borings CEG 1 through 6)

- Reach 2 - Broadway to Harbor Freeway
  Hollywood Freeway to the Harbor Freeway (includes Borings CEG 7, 8 and 9)

- Reach 3 - East Wilshire Boulevard
  Harbor Freeway to Normandie Avenue (includes Borings CEG 10 through 14)

- Reach 4 - Central Wilshire Boulevard
  Normandie Avenue to La Brea Avenue (includes Borings CEG 15 through 18)

- Reach 5 - La Brea Area
  La Brea Avenue to Melrose Avenue (includes Borings CEG 19 through 23A)

- Reach 6 - Hollywood Area
  Melrose Avenue to Yucca Street (includes Borings CEG 24 through 28)

- Reach 7 - Santa Monica Mountains
  Yucca Street to Universal City Station (includes Borings CEG 28A through 34)

- Reach 8 - San Fernando Valley
  Universal City Station to the north Portal (includes Borings CEG 35 to 38).

The following is a discussion of geologic engineering properties of "soft-ground" and "rock" units expected along the alignment. Specific characteristics of individual reaches are presented in Section 5.2.

For the purpose of this report, the various formations previously identified in the geologic literature have been grouped into geologic units with similar engineering properties, which are expected to exhibit similar behavior in construction. Two general categories are considered: "soft-ground" and "rock".

### 5.1.1 "Soft-Ground"

This category contains the Young and Old Alluvium, the San Pedro Formation (sand), and the Fernando and Puente formations. The Fernando and Puente formations are often referred to as "bedrock" or "rock" in various other papers and reports and at places within this report, but they have the engineering properties of hard, dense soils with significant cohesive strength. Hence they are grouped here in "soft-ground". Six different geologic units are identified within the soft-ground category:

- $A_1$ - Young Alluvium: granular, loose to very dense
- $A_2$ - Young Alluvium: fine-grained, soft to very stiff
- $A_3$ - Old Alluvium: granular, medium to very dense
- $A_4$ - Old Alluvium: fine-grained, medium to hard
- SP - San Pedro Formation: relatively cohesionless sand, medium to very dense
- C - Claystone, siltstone, sandstone: fine-grained, low hardness, represents the Fernando and Puente formations. Although classified as soft-ground, this material could be termed "soil-like" bedrock.

The above six geologic units are further identified, where appropriate, as: bouldery (b), petroliferous (p), and wet or saturated (w). These terms are defined in Section 4.3.3 and on Drawing 2.

### 5.1.2 "Rock"

This category includes sedimentary rock of the Topanga Formation (conglomerate, sandstone and siltstone), and volcanic rock of the Topanga Formation (basalt). These materials are much harder and stronger than the geologic units grouped above in the "soft-ground" category and are expected to require different construction procedures, and exhibit a different type of behavior.

About three miles of tunnel in the Santa Monica Mountains are expected to encounter these rock units. These rock units are identified on Drawing 2 using the Terzaghi (1946) Rock Condition Numbers modified where applicable for apparent wetness or saturation (w). The Terzaghi Rock Condition Numbers used herein are:

- No. 1 - hard and intact
- No. 2 - hard and stratified
- No. 3 - massive, moderately jointed (spacing average > 6 ft apart)
- No. 4 - moderately blocky and seamy (spacing 2 to 6 ft apart)
- No. 5 - very blocky and seamy (joint spacing 1 in to 2 ft apart)
- Nos. 6 through 9 - crushed but chemically intact rock (6) through swelling and squeezing rock (9) conditions. These rock conditions are not expected to be encountered in significant amounts, based on records of previous tunnels near here (see Section 6.0).

In practice, there are no sharp boundaries between the Terzaghi Rock Condition Numbers; therefore, a range of several Terzaghi Numbers is used for describing some rock conditions.

### 5.1.3 Geologic and Engineering Properties of Soft-Ground Units

The quantitative geologic and engineering properties of the six different geologic soft-ground units are discussed in this section.

The qualitative engineering geology differences, and similarities (description, composition, consistency, compactness, bedding) of these geologic units were discussed in Sections 4.3.4 through 4.3.10.

A summary of laboratory test results for geologic units $A_1$, $A_2$, $A_3$, $A_4$, SP and C is presented in Table 5-2. The summary includes high, low, mean and standard deviation of engineering properties for selected tests performed on each of these geologic units. Soils engineering properties from laboratory tests are presented in Tables H-1 and H-2 Appendix H, Volume II.

Engineering properties, shown graphically on CEG Boring Logs in Drawings 5 through 12 "Engineering Geology Profile", include:

- ° Standard Penetration Tests (N-Values)
- ° Unconfined Compression Strength
- ° Static Triaxial Compression
- ° Cyclic Triaxial
- ° Dynamic Triaxial
- ° Resonant Column
- ° Moisture Content
- ° Dry Density
- ° Petroleum Sample
- ° Dip of Bedding
- ° Downhole Compressive Wave Velocity
- ° Downhole Shear Wave Velocity.

The mean unconfined compressive strength (psi) and/or direct shear (ø, C) for the six soft-ground units are presented in Table 5-1.

**Table 5-1 Mean Strengths of Soft-Ground Units**

| Unit | Unconfined Strength (psi) | Direct Shear ø(deg.)/C(ksf) |
|---|---|---|
| $A_1$ | - | 36.5/0.54 |
| $A_2$ | 71.8* | 26.8/0.75 |
| $A_3$ | - | 35.2/0.69 |
| $A_4$ | 43.5 | 27.5/1.17 |
| SP | - | 32.3/0.32 |
| C | 79.4 | 26.8/1.21 |

*One test.

TABLE 5-2 Summary of Laboratory Test Results

| Geologic Unit* | | Dry Density Pcf | Water Content % | Unconfined Strength, psi | Direct Shear φ, degrees | Direct Shear C, ksf | Liquid Limit | Plasticity Index | Coefficient of Permeability Kv, cm/sec |
|---|---|---|---|---|---|---|---|---|---|
| $A_1$ | Number of Tests | 21 | 21 | -0- | 11 | 11 | -0- | -0- | 11 |
| | High | 131 | 31 | - | 44 | 0.81 | - | - | $4.3 \times 10^{-4}$ |
| | Low | 91 | 7 | - | 26 | 0.14 | - | - | $1.0 \times 10^{-7}$ |
| | † Mean | 110.7 | 15.7 | - | 36.5 | 0.54 | - | - | $3.0 \times 10^{-5}$ |
| | Std. Deviation | ±10.3 | ±5.9 | - | ±7.8 | ±0.27 | - | - | |
| $A_2$ | Number of Tests | 14 | 14 | 1 | 8 | 8 | 4 | 4 | 2 |
| | High | 108 | 28 | 71.8 | 38 | 1.30 | 53 | 28 | $2.9 \times 10^{-5}$ |
| | Low | 90 | 13 | - | 15 | - | 29 | 9 | $4.8 \times 10^{-7}$ |
| | † Mean | 102.0 | 22.2 | - | 26.8 | 0.75 | 37 | 15 | $3.7 \times 10^{-6}$ |
| | Std. Deviation | ±5.5 | ±4.4 | - | ±9.8 | ±0.47 | +10.8 | ±8.8 | |
| $A_3$ | Number of Tests | 44 | 44 | -0- | 25 | 25 | 7 | 7 | 11 |
| | High | 135 | 28 | - | 40.5 | 2.58 | 47 | 22 | $7.0 \times 10^{-4}$ |
| | Low | 94 | 6 | - | 23.5 | 0.16 | 32 | 7 | $2.0 \times 10^{-7}$ |
| | † Mean | 110.7 | 17.1 | - | 35.2 | 0.69 | 36 | 15 | $3.8 \times 10^{-6}$ |
| | Std. Deviation | ±8.2 | ±4.9 | - | +6.3 | ±0.56 | ±5.1 | ±5.1 | |
| $A_4$ | Number of Tests | 60 | 60 | 12 | 18 | 18 | 15 | 15 | 12 |
| | High | 121 | 48 | 93.8 | 40 | 2.64 | 80 | 51 | $1.7 \times 10^{-5}$ |
| | Low | 79 | 11 | 9.3 | 5 | 0.22 | 32 | 6 | $1.5 \times 10^{-8}$ |
| | † Mean | 103.6 | 22.9 | 43.5 | 27.5 | 1.17 | 47 | 23 | $3.0 \times 10^{-7}$ |
| | Std. Deviation | ±9.6 | ±6.7 | ±20.9 | ±13.0 | ±0.59 | ±12.2 | ±12.3 | |
| SP | Number of Tests | 15 | 15 | -0- | 9 | 9 | -0- | -0- | 4 |
| | High | 114 | 24 | - | 35 | 0.56 | - | - | $3.4 \times 10^{-4}$ |
| | Low | 93 | 10 | - | 29.5 | 0.07 | - | - | $3.1 \times 10^{-5}$ |
| | † Mean | 104.1 | 18.9 | - | 32.3 | 0.32 | - | - | $1.1 \times 10^{-4}$ |
| | Std. Deviation | ±6.5 | ±4.6 | - | ±2.6 | ±0.15 | - | - | |
| C | Number of Tests | 173 | 173 | 113 | 5 | 5 | 45 | 45 | 12 |
| | High | 116 | 48 | 183.0 | 41.0 | 2.01 | 75 | 38 | $9.7 \times 10^{-7}$ |
| | Low | 73 | 18 | 3.5 | - | 0.62 | 31 | 3 | $3.1 \times 10^{-8}$ |
| | † Mean | 92.5 | 29.0 | 79.4 | 26.8 | 1.21 | 48 | 15 | $9.9 \times 10^{8}$ |
| | Std. Deviation | ±7.7 | ±5.8 | - | - | - | ±10.5 | ±7.0 | |

*As described on Drawing 2 and in report

†The arithmetic average, or mean, was used for each test, except for the permeability tests. The geometric mean was used for permeability tests defined as:

$$\text{geometric mean ms} = (a_1 \times a_2 \times \ldots \times a_n)^{1/n}$$

### 5.1.4 Geologic and Engineering Properties of Rock Units

The quantitative geologic and engineering properties of the two rock units are discussed here. The qualitative engineering geology differences were presented in Sections 4.3.11 and 4.3.12. Limited tests were performed on rock samples by tunnel boring machine (TBM) manufacturers. The results are presented in Table 5-4. As shown, no tests were performed on soft-ground samples identified as Tp (Puente Formation) or Tf (Fernando Formation), and results of requested tests from one of the manufacturers was never received. The range of results of tests performed to date is shown in Table 5-3.

Table 5-3 Rock Unit Unconfined Strength

| Formation | Rock Description | Range of Unconfined Compressive Strength (psi) |
|---|---|---|
| Topanga | Massive sandstone | 3,622 to 9,072 |
| Topanga | Weak sandstone | 326 to 775 |
| Topanga | Fractured basalt, well healed | 1,811 to 5,040 |
| Topanga | Basalt, massive, strong | 181 to 9,300 |

Special note must be made of the very low, unconfined compressive strength of 181 psi (Table 5-4) submitted by Jarva. We suspect this is an error. As noted in Jarva's June 17, 1981 letter (Appendix I, Volume II), the tensile strength of 101 psi (Brazilian Tensile test) on the same sample was highest of any sample tested. Based on these two parameters, the unconfined compressive strength of relatively defect-free basalt is judged to exceed 9,000 psi.

### 5.1.5 Gas and Oil

The location of oil fields and occurrence of gas in relation to the alignment are described in Section 4.7. Anticipated "tunnel gas classifications" along the alignment are defined in Section 4.8, and the locations expected to be encountered are illustrated on Drawing 2.

Tests for gas and oil were performed at selected CEG borings along the proposed Metro Rail alignment. A summary of test results and interpretations, from gas chromatograms, is presented in Appendices F1 and F11, Volume II.

#### 5.1.5.1 Gas

Natural gases that formed at the depth related to known oil fields are likely to collect beneath perched water levels. Additionally, gases that collect beneath perched water levels are likely to migrate laterally. Consequently, gases may be present beneath the perched water levels in Metro Rail Project Reaches 3, 4 and 5 (Drawing 2) even though these reaches are, in some cases, quite far from the known oil fields shown on Drawing 1. Additionally, gases which migrate laterally are expected to collect in "pockets" at impermeable

TABLE 5-4 Tunnel Boring Machine (TBM) Manufacturers Tests

| TBM Company | CEG Hole No. | Sample I.D. or *Depth* | Unconfined Compression (psi) | Silica $SiO_2$ (%) | Hardness | Unconfined Tensile (psi) | Density ($g/cm^3$) | Remarks |
|---|---|---|---|---|---|---|---|---|
| AEC Incorporated (Alpine) 5-18-81 | 11 | S-6 | not tested | - | - | - | - | Tp - weak siltstone |
| | 12 | S-12 | not tested | - | - | - | - | Tp - weak claystone |
| | 9 | S-2 | not tested | - | - | - | - | Tf - weak claystone |
| | 22 | S-5 | not tested | - | - | - | - | Tf - weak siltstone |
| | 19 | S-5 | not tested | - | - | - | - | Tf - oily siltstone |
| | 32A | 382.0 | 9,072 | - | - | - | - | Tt - massive sandstone |
| | 33 | 216.0 | 652 | - | - | - | - | Tt - weak sandstone |
| Atlas Copco Jarva 5/27/81 | 32A | 388.0 | 3,441 | - | 1.4* | - | - | Tt - massive sandstone |
| | 33 | 201.5 | 326 | - | 0.5* | 75 | - | Tt - weak sandstone |
| | 32 | 199.0 | 1,811 | - | 0.5* | - | - | Basalt - fractured, well-healed |
| | 32 | 310.5 | 181 | - | 1.7* | 388 | - | Basalt - massive, strong |
| The Robbins Company 5/27/81 | 32A | 382.5 | 8,270** | 10 to 20 | 2 to 7† | - | 2.44 | Tt - massive sandstone |
| | 33 | 195.0 | 775 | 20 to 40 | 2 to 7† | - | 2.11 | Tt - weak sandstone |
| | 32 | 198.0 | 5,040 | 0 | 3 to 6† | - | 2.66 | Basalt - fractured, well-healed |
| | 32 | 309.5 | 9,300** | 0 | 3 to 6† | - | 2.49 | Basalt - massive, strong |
| Zokor | 11 | S-6 | not tested | | | | | Tp - weak siltstone |
| | 12 | S-12 | not tested | | | | | Tp - weak claystone |
| | 9 | S-2 | not tested | | | | | Tf - weak claystone |
| | 22 | S-5 | not tested | | | | | Tf - weak siltstone |
| | 19 | S-5 | not tested | | | | | Tf - oily siltstone |
| | 32A | 383.0 | not tested | | | | | Tt - massive sandstone |
| | 30 | 116.0 | not tested | | | | | Tt - weak sandstone |
| | 32 | 197.0 | not tested | | | | | Basalt - fractured, well-healed |
| | 32 | 311.5 | not tested | | | | | Basalt - massive, strong |

*Punch test hardness

**Slow RPM, hi torque TBM in 22-ft diameter tunnel = 21-ft/hr advance if all is working well

†Mohs hardness range

structural traps (often faults). The known faults that could be traps along the alignment are the MacArthur, 6th Street, 3rd Street, San Vicente and Santa Monica faults.

Determination of gas volume and pressure were beyond the scope of this study; however, the following observations were made:

- Expansion of gas, in the sample tube, pushed the material 2 to 3 inches beyond each end of the tube during the drilling program in Boring CEG 1 (108 ft, Puente Formation claystone) and Boring CEG 23 (150 ft and 172 ft, siltstone interbeds in San Pedro Formation.
- Expansion of gaseous material is a very significant factor in reducing strength of the material, as noted by Smith and Byrne, 1980, pp. 1, 2:

> The behaviour of oilsand is unusual, however, primarily because of the presence of dissolved gas in the pore fluids. Under in situ stresses, the oilsand is very dense and has a relatively high strength. However, as it is unloaded the gas comes out of solution, causing the oilsand to expand, with an associated significant drop in strength. This behaviour is a major concern with respect to the design of shafts and tunnels in the oilsands. The empirical approach is of limited value for design of shafts or tunnels in the oilsands because of its unusual properties and because there is so little previous tunneling experience in oilsand on which to draw. Alternative approaches for the design of shafts or tunnels are available, which are based on continuum analyses of deformation behaviour of the material in which the excavation is to be made. One such approach, which is described in this paper, is termed the convergence-confinement method.

- Gas was observed bubbling in Borings CEG 1, 2, 11, 19, 20 and 23.
- Boring CEG 23 encountered the largest gas flow, and this fact is believed the result of gas being trapped by the Santa Monica fault; e.g., apparently gas could not migrate northerly into nongassy Reach 6 (Drawing 2). This fault area may be comparable to MWD's San Fernando Tunnel at the Santa Susana faults where a fatal gas explosion occurred (Section 6.1.7)
- Tonner Tunnel was classified as "gassy" in the Puente Formation, but there was no problem because ventilation was adequate (Section 6.6.5).

5.1.5.2 Oil

The location of oil is described in Section 4.7 and shown on Drawing 1. Analyses of petroliferous (p) samples encountered in borings is presented on Drawing 2. The mean oil and water content of each of the geologic units sampled is presented in Table 5-5:

Table 5-5 Oil & Water Content

| Geologic Unit | Oil & Water Content (percent by weight) Oil | Water | Reach |
|---|---|---|---|
| $A_4$ | 11 | 8 | 5 |
| SP | 14 | 8 | 5 |
| C* | 7 | 17 | 1 |

*Puente Formation

Under ordinary circumstances, unconfined compressive strength cannot be obtained from granular soils, but the test results presented in Table 5-6 were possible because of the presence of tar in the granular sand.

Table 5-6 "Tar Sand" Unconfined Strength

| Boring CEG No. | Depth (ft) | Geologic Unit | Unconfined Compression (psi) |
|---|---|---|---|
| 19 | 20 | $A_4$ | 8.1 |
| 19 | 41 | SP | Deformed under own weight |
| 23 | 105 | SP | 1.2 |

A special controlled temperature test was performed on two samples of "tar sand" using a pocket penetrometer to obtain an estimate of their unconfined compressive strength. The results shown in Table 5-7 suggest the strength of "tar sands" is related to temperature and, in open excavation, temperature may have an effect perhaps 2 to 6 inches into the excavated surface.

Table 5-7 "Tar Sand" Unconfined Strength w/Temperature

| Boring CEG No. | Sample Depth (ft) | Geologic Unit | Temperature (°F) | Unconfined Compression (TSF) |
|---|---|---|---|---|
| 19 | 20 | $A_4$ | 46 | 3.4 |
| | | | 70 | 2.5 |
| | | | 104 | 1.4 |
| | | | 158 | 1.1 |
| 23 | 105 | SP | 41 | >4.5 |
| | | | 72 | 2.0 |
| | | | 100 | 0.7 |
| | | | 140 | 0 |

A very small amount of oil was obtained from a fresh, hard, massive siltstone of the Topanga Formation from Boring CEG 31 (Hollywood Bowl fault) at a depth of 87 to 89 feet. The oil content was only 0.06 percent by weight, and water content 2.26 percent by weight. This finding is significant because Boring CEG 31 is in "non-gassy" Reach 7. The origin of the oil is believed to be a deeper petroleum source that migrated toward the surface (Appendix FII.4, Volume II).

No gas was detected in Boring CEG 31. Since this occurrence of oil may be fault-related, future testing for gas near this location would be prudent.

### 5.1.6 Water Quality

Chemical analyses and selected water parameters of sampled water obtained in CEG borings is presented in Appendix G, Volume II. Results of the water analyses, from perched or permament, shallow or deep, ground water along the proposed Metro Rail Project alignment, indicate significant Reaches containing very poor to poor water quality; e.g., high in Total Dissolved Solids (TDS). For example, Boring CEG 6 (Reach 1), Boring CEG 11, (Reach 3), Boring CEG 19 (Reach 5) contain TDS exceeding 15,000 parts per million (ppm). By comparison, the U.S. Environmental Protection Agency standard for potable domestic drinking water is 500 ppm.

#### 5.1.6.1 Sulfate ($SO_4$) Content

Ground water along the alignment is high in sulfate content. The sulfate content in 19 of the 31 CEG borings sampled exceeded 150 ppm (see Appendix G, Volume II). Sulfate content above 150 ppm is generally regarded to be deleterious to concrete lining, requiring sulfate-tolerant concrete. For example, Type II cement is appropriate for sulfate concentrations from 150 to 1,000 ppm. Therefore, the Reaches in Table 5-8 would appear to require Type II cement based on samples removed from borings.

Table 5-8 Ground Water Sulfate Content

| Boring CEG No. | Reach | Ground Water Sulfate Content (ppm) |
|---|---|---|
| 1 | 1 | 475 |
| 3 | 1 | 152 |
| 16 | 4 | 231 |
| 19 | 5 | 240 |
| 21 | 5 | 263 |
| 22 | 5 | 149 |
| 23A | 5 | 154 |
| 26 | 6 | 161 |
| 27 | 6 | 245 |
| 28A | 6 | 272 |
| 30 | 7 | 202 |
| 31 | 7 | 161 |
| 32A | 7 | 434 |
| 33 | 7 | 693 |
| 36 | 8 | 253 |
| 37 | 8 | 418 |
| 38 | 8 | 463 |

Borings and Reaches where sulfate content of ground water exceeds 1,000 ppm, possibly requiring special cement:

| | | |
|---|---|---|
| 10 | 2 | 2,200 |
| 29 | 7 | 2,600 |

5.1.6.2 Sodium Chloride (NaCl) Content

Several chemical analyses of ground water samples exhibit very high sodium chloride, or salt, content as presented in Table 5-9. These "salty", sodium chloride-type waters are judged to originate from oil field brines. About 10 miles of the alignment are located near existing oil fields. The sodium chloride content of these waters is considered high when exceeding 1,000 ppm. Such water easily corrodes metals used in construction.

Table 5-9 Ground Water Sodium Chloride Content

| Boring CEG No. | Reach | Ground Water Sodium Chloride Content (ppm) |
|---|---|---|
| 3 | 1 | 3,000 |
| 4 | 1 | 4,600 |
| 6 | 1 | 19,000 |
| 10 | 3 | 1,400 |
| 11 | 3 | 18,000 |
| 12 | 3 | 5,340 |
| 16 | 4 | 5,700 |
| 19 | 5 | 13,600 |
| 29 | 7 | 3,090 |
| 35 | 8 | 2,218 |

The poor quality ground water, however, is not limited to the reaches near oil fields, as testimony from high TDS, in Borings CEG 29, 33, 35 and 38 (Cahuenga Pass to north Portal).

Based on the above, and knowing that much of the ground water is highly mineralized, additional corrosion studies are warranted to assist the designer in selecting proper materials for construction.

5.1.6.3 Water Chemistry Average and Exceptional Deviations

Table 5-10 indicates the average water chemistry for some important characteristics and the exceptional deviations from the average.

TABLE 5-10 "Average" and "Exceptions" - Water Quality Analyses

| | Boring CEG No. | Location | pH | TDS (ppm) | Boron (ppm) | Fluoride (ppm) | Comments |
|---|---|---|---|---|---|---|---|
| "Average" | - | - | 7.6 | 700 | 2.0 | 0.7 | All borings |
| "Exceptions" | 6 | Downtown L.A. | - | 20,230 | 38 | - | Oil field brine? |
| | 11 | MacArthur Park | - | 19,670 | 37 | - | Artesian, oil field brine |
| | 19 | La Brea Tar Pits | - | 15,425 | 11 | - | Oil field brine |
| | 22 | La Brea Area | 8.0 | - | - | - | Shallow (16') alluvium |
| | 29 | Franklin Avenue | 8.0 | 5,996 | - | - | Hollywood fault area |
| | 31 | Hollywood Bowl | 8.6 | - | - | - | Shallow (28') alluvium |
| | 32 | Santa Monica Mts. | 9.8 | - | - | 1.3 | Topanga conglomerate |
| | 32A | Santa Monica Mts. | 8.0 | - | - | - | Basalt |

## 5.1.7 Seismic Refraction Surveys

Seismic refraction surveys were performed at six locations along the proposed Metro Rail Project alignment (Drawing 3) to locate and evaluate faults and buried river channels. The results are presented in Appendix C, Volume II. A summary of seismic refraction interpretations is presented in Table 5-11.

TABLE 5-11 Seismic Refraction Interpretation

| Area | Reach | Purpose | Results | Ground Water Depths | Comments |
|---|---|---|---|---|---|
| 1-Macy Street | 1 | Delineate buried alluvial channel(s); verify fault[a] | 130' deep channel 700' west of existing LA River channel; no fault | 20' to 44' beneath S-6 | Ancestral Los Angeles River channel |
| 2-Hancock Park | 5 | Evaluate alluvial depth & substantiate Salt Lake Oil Field faults[b] | Alluvium 60' to 100' deep; two possible fault-related anomalies beneath S-38 & S-39 | No distinct saturated zone. Perched water likely | La Brea tar pit area |
| 3-Fairfax High | 5 | Determine alluvial depth & locate Santa Monica fault[c] | Alluvium 105' to 190' deep; one slight step anomaly (fault?) noted in the alluvial/bedrock interface | 18' to 42' deep | Bedrock surface slopes gently west 105' to 190' below surface |
| 4-Hollywood Fault | 7 | Locate Hollywood fault[d] | Fault not detected to depth range of information (±100') | 13' to 52' below ground surface | Hollywood fault likely south of seismic lines & Boring 28A |
| 5-Hollywood Bowl | 7 | Evaluate Hollywood Bowl fault offset/location[e] | Anomaly indicative of possible fault offset beneath northwest end of S-51 | In alluvium, 8' to 34' deep; in bedrock, 24' to 40' deep | Possible northerly splay of Hollywood fault |
| 6-North Hollywood | 7 | Locate Benedict Canyon fault[d] | Substantial anomaly (fault) beneath line S-28; slight step anomaly beneath S-33 | 5' to 10' beneath S-28 to S-34 & 120' beneath S-21 & S-22 | Confirmed location of main Benedict Canyon fault |

[a] Yerkes, and others, 1977

[b] Crowder, 1961

[c] Crowder, 1961, and Weber, 1980

[d] Hoots, 1930, and Weber, 1980

[e] Hoots, 1930, and Los Angeles City, 1970

## 5.1.8 Micro-Gravity Surveys

Micro-gravity and regional gravity surveys were performed along the proposed alignment at the locations shown on Drawing 3. The purpose was to supplement boring and seismic refraction surveys relative to locating faults and buried river channels along the alignment. The interpretive results are shown on Figures D-1 through D-6, Appendix D, Volume II, and summarized on Table 5-12.

The most important result is the location and sense of displacement of the Santa Monica fault beneath Profile 4 (Figure D-4, Appendix D, Volume II). However, the proposed Metro Rail Project alignment is about 800 feet north of the Santa Monica fault at this location. The location of the Santa Monica fault, where the alignment does cross the Santa Monica fault trace, is not confirmed by Profile 3 (Figure D-3, Appendix D, Volume II) because the sense of displacement is south side up, and this does not correspond to the reported known displacement of north side up. Additionally, the Hollywood fault appears to have been accurately located by Profile 5 (Figure D-5, Appendix D, Volume II).

TABLE 5-12 Micro- and Regional Gravity Interpretation

| Profile No. | Reach | Area | Purpose | Results |
|---|---|---|---|---|
| 1 | 1 | Macy Street | Define L.A. River channel | Confirms 130' depths between Borings CEG 5 & 6 |
| 2 | 2 | 7th & Harlem | Define alluvial channeling | Data compares with borings |
| 3 | 5 | Fairfax High School | Evaluate Santa Monica fault | Apparent fault (south side up) at alluvium/San Pedro sand 180' to 500' below surface north of Boring CEG 23A |
| 4 | 6 | Fountain & Cahuenga | Evaluate Santa Monica fault | Apparent 50° north-dipping, 150' thrust offset with Fernando over alluvium (north side up); surface projection lies south of Boring CEG 27; not on tunnel alignment |
| 5 | 6 | Hollywood & Cahuenga | Evaluate Hollywood fault | Apparent 50° north-dipping, 400' bedrock offset thrust (north side up); surface projection lies south of Boring CEG 28A |
| 6 | 7 | Highland & Hollywood Freeways | Evaluate postulated fault south of Hollywood Bowl | Presence & location not conclusive; anomalous gravity offset |

Interpretation of regional gravity contour gradient (Drawing 3) on the north side of the Santa Monica Mountains does not change significantly, suggesting that the alluvial/bedrock contact probably follows a relatively constant slope out into the San Fernando Valley, as shown on Drawing 2.

### 5.1.9 Downhole and Crosshole Surveys

Downhole surveys were performed in 17 of the CEG Borings; i.e, 5, 7, 8, 9, 11, 13, 14, 15, 18, 20, 23, 23A, 24, 28, 31, 34 and 38. Results are presented in Appendix EI, Volume II. Crosshole surveys were conducted at 10 selected station sites; i.e., Borings CEG 5, 7, 11, 13, 15, 18, 20, 24, 28 and 34. The results are shown in Appendix EII, Volume II, and the velocities are listed on Table EII-1. The "average" downhole and crosshole shear wave velocities (Vs) 0 to 50 feet and 50 to 200 feet below the surface are listed in Table 5-13.

TABLE 5-13 "Average" Shear Wave Velocities (Vs)

| | Near-Surface Vs (0 -50') | Deep Vs (50 - 200') | Remarks |
|---|---|---|---|
| Downhole | 800 - 1,200 fps | 1,700 fps | Exception: Boring CEG 31 = 4,800 fps in Basalt & Conglomerate |
| Crosshole | 900 (±200) fps | 1,400 (±200) fps* | Exception: Boring CEG 28 = 1,700 (±200) fps in Topanga Sandstone |

*at 100-foot depth

#### 5.1.9.1 Downhole Shear Wave Velocities

The mean shear wave velocity ($\bar{V}s$) for geologic units are:

Table 5-14 Mean Shear Wave Velocities ($\bar{V}_s$) by Geologic Unit

| Geologic Unit | Shear Velocity $\bar{V}_s$ (fps) |
|---|---|
| $A_1$ | 1,115 |
| $A_2$ | 1,138* |
| $A_3$ | 1,470 |
| $A_4$ | 1,501 |
| SP | 1,128 |
| C - Fernando Formation | 1,392 |
| C - Puente Formation | 1,507 |
| Tt - Topanga Formation† | 2,509 |

* Crosshole

†Terzaghi Rock Condition Nos. 1-5

Based on Table 5-14 grouping by similarity in the densities of the geologic units are:

- $A_1$, $A_2$, SP
- $A_3$, $A_4$, C Fernando Formation
- Parts of the Puente and Topanga sedimentary units are denser than Units $A_1$, $A_2$, $A_3$, $A_4$ and the Fernando Formation.
- Old Alluvium appears generally to be denser than the Fernando Formation and about as dense as the Puente Formation.

### 5.1.9.2 <u>Crosshole Shear Wave Velocities</u>

The mean shear wave velocity (Vs) measured from the crosshole surveys for each geologic unit are so similar to that listed in Table 5-14 that they are not re-tabulated.

## 5.2 SPECIFIC CHARACTERISTICS OF REACHES

This section addresses characteristics of materials and conditions specific to each individual reach of the proposed Metro Rail Project alignment. Drawing 2 and Drawings 5 through 12, inclusive, should be referenced for a graphic representation of the materials discussed herein. The anticipated behavior in underground construction, and surface excavations, is described in Sections 7.0 and 8.0, respectively. A general description of "soft-ground" units, "rock" units, faulting, ground water, oil and gas, project tunnel gas classification, and regional seismicity was presented in Section 4.0.

### 5.2.1 <u>Reach 1 - Downtown, Los Angeles River (Borings CEG 1 through 6)</u>

Reach 1 begins at the east Portal (near Gallardo Street) and extends westerly about 1.2 miles to the Hollywood Freeway. This reach crosses alluvial deposits of the Los Angeles River. Claystone outcrops near the surface on each end of this reach (Drawing 2). Geologic units $A_1$ and C occur along this reach. Important known properties and conditions of these units are:

$A_1$ (Young Alluvium)

- Undrained, Quick, Direct Shear (two tests) - $\phi$ - Range = 26° to 41°; average 34°; C - Range = 0.14 to 1.2 ksf; average 0.7 ksf
- N-Value - Range 9 to >100; average 90 bpf
- Passing No. 200 Sieve (two tests) - Range 3 to 10; average 6.5%
- Coef. Perm (two tests) - Range $5.4 \times 10^{-6}$ to $4 \times 10^{-4}$ cm/sec; average $2 \times 10^{-4}$ cm/sec
- Estimated Transmissivity - To 200,000 gpd/ft
- Modern L.A. River Channel Deposits - To 130 feet deep
- Bouldery Ground - Believed up to 4 ft diam; quantity unknown, but not abundant
- Permanent Water Level - 20 to 30 ft below ground surface
- Water Quality - Very poor, up to 20,000 ppm TDS (Boring CEG6)

C (Puente Formation)

- Unconfined Compressive Strength (Qu) (20 tests) - Range 10 to 175 psi; average 79 psi
- Passing No. 200 sieve (two tests) - 68%
- Coef. Perm. (one test) - $7.7 \times 10^{-8}$ cm/sec.
- Swell (one test) - 0.64%
- Faults - None known
- Occasional Hard Sandstone Beds - 1 in. to 3 ft thick (quantity unknown)
- Fracture Spacing - Little (1 ft to 3 ft) to moderate (0.5 ft to 1 ft)
- Dip of Bedding - 10° to 60° from horizontal; probably southward
- Ground Water Condition - Probably seeps; formation tight, took less than 1 gpm in water pressure tests
- Gas Encountered - Boring CEG 1, 2, 3 and 4
- Oil Encountered - Boring CEG 2 at 37 ft rose to within 14 ft of surface in two days

5.2.2 Reach 2 - Broadway to Harbor Freeway (Borings CEG 7, 8 and 9)

In Reach 2, the proposed alignment traverses about 1.6 miles from the Hollywood Freeway southwest through the downtown area of Los Angeles and then northwest to the Harbor Freeway. Geologic units $A_1$, $A_2$ and C occur along this reach. Important known properties and conditions of these units are:

$A_1$ (Young Alluvium)

- Passing No. 4 Sieve (one test) - 32%
- Passing No. 200 Sieve (one test) - 4%
- Estimated Transmissivity - To 200,000 gpd/ft
- Ancestral L.A. River Channel - To 150 ft deep
- Permanent Water Level - 100 ft below ground surface
- Bouldery Ground - Judged up to 4 ft diam; quantity unknown, but less than Reach 1, based on boring data
- Water Quality - Unkown, not sampled this Reach

$A_2$ (Young Alluvium)

- Occurrence - As lenses in Unit $A_1$
- Undrained, Quick, Direct Shear (two tests) - $\phi$ Range = 28° to 38°; average 33°
  C - Range = 0 to 1.0 ksf; average 0.5 ksf
- N-Value - 15 to > 100 bpf; average 85 bpf
- Non-bouldery Ground

C (Fernando Formation)

- Unconfined compressive strength (Qu) (nine tests) - Range = 36 to 107 psi; average 70 psi
- Undrained, Quick, Direct Shear (three tests) - $\phi$ - Range = 28° to 38°; average 32°
  - C - Range = 0° to 2.4 ksf; average 1.1 ksf
- Passing No. 200 Sieve - 52%
- Coef. Perm - Range = $3.8 \times 10^{-8}$ to $5.0 \times 10^{-8}$ cm/sec; average $4.5 \times 10^{-8}$ cm/sec.
- Swell (two tests) - 2.0%
- Faults know - None known
- Occasional Hard Sandstone Beds - 1 in to 3 ft thick; quantity unknown but are less prevalent than Puente Formation
- Fracture Spacing - Little (1 ft to 3 ft) to moderate (0.5 ft to 1 ft)
- Dip of Bedding - 5° to 60° from horizontal; probably southerly
- Ground Water Condition - Probably seeps, tight formation, took less than 1 gpm in water pressure tests
- Gas Encountered - Borings CEG 7, 8 and 9
- Oil Encountered - Boring CEG 8 "tar"

### 5.2.3 Reach 3 - East Wilshire Boulevard (Borings CEG 10 through 14)

This portion of the alignment trends approximately 2.5 miles along Wilshire Boulevard to Normandie Avenue across a thin layer of Old Alluvium ($A_4$) which is underlain mostly by the Puente Formation (C). Important known properties of these units are:

$A_4$ (Old Alluvium)

- Undrained, Quick, Direct Shear (one test) - $\phi$ - 5°
  C - 0.98 ksf
- N-Value - 6 to 70 bpf; average 50 bpf
- Coef. Perm - Range = $1.5 \times 10^{-8}$ to $1.9 \times 10^{-7}$ cm/sec; average $1.0 \times 10^{-7}$ cm/sec
- Perched Water Level - Mostly 20 to 30 ft below ground surface; follows base of $A_4$
- Non-bouldery Ground - None encountered in borings

C (Puente Formation)

- Undrained, Quick, Direct Shear (one test) - $\phi$ - 44°
  C - 0.81 ksf
- Unconfined Compressive Strength (Qu) (39 tests) - Range 7 to 157 psi; average 66 psi
- Passing No. 200 Sieve - 76%
- Coef. Perm. - Range $3.9 \times 10^{-8}$ to $4.6 \times 10^{-7}$ cm/sec; average $2.1 \times 10^{-7}$ cm/sec
- Faults - MacArthur fault near Boring CEG 11; probably trap for gas; closely spaced fractures 10 ft each side of fault
- Occasional Hard Sandstone Bed - 1 in to 3 ft thick; quantity unknown
- Fracture Spacing - Little (1 ft to 3 ft) to moderate (0.5 ft to 1 ft)

- Dip of Bedding - 20° to 55° from horizontal; probably southerly
- Artesian Water - Boring CEG 11
- Permanent Ground Water Level - 120 to 150 ft below ground surface
- Ground Water Condition - Seeps; water pressure tests took less than 2 gpm
- Water Quality - Very poor, up to 19,670 ppm TDS in Boring CEG 11
- Gas Encountered - Borings CEG 10, 11 and 12
- Oil Not Encountered

### 5.2.4 Reach 4 - Central Wilshire Boulevard (Borings CEG 15 through 18)

This reach is about 2.5 miles along Wilshire Boulevard between Normandie Avenue and La Brea Avenue. Four geologic units are present: $A_3$, $A_4$, SP and C. Important features are:

$A_3$ (Old Alluvium)

- Unconfined Compressive Strength (Qu) (one test) - 40 psi
- Undrained, Quick, Direct Shear (one test) - $\phi$ - 24°; C - 1.2 ksf
- Passing No. 200 Sieve - 81% average
- Swell - 3.3%
- Coef. Perm. - Range = $2.1 \times 10^{-7}$ to $1.0 \times 10^{-6}$ cm/sec; average $6.1 \times 10^{-7}$ cm/sec
- Non-bouldery - None encountered in borings
- Perched Water Level - 20 to 40 ft below ground surface

$A_4$ (Old Alluvium)

- Unconfined Compressive Strength (Qu) - Range 37 to 40 psi; average 39 psi
- Undrained, Quick, Direct Shear (two tests) - $\phi$ - 32°; C - 0.78 ksf
- Passing No. 200 Sieve (two tests) - 78% average
- Swell (one test) - 9%
- Coef. Perm (four tests) - Range $2.1 \times 10^{-8}$ to $4.1 \times 10^{-6}$ cm/sec; average $1.5 \times 10^{-6}$ cm/sec
- Saturated - Below perched water level
- Non-bouldery - None encountered in borings

SP (San Pedro Formation, Sand)

- Undrained, Quick, Direct Shear (three tests) - $\phi$ - Range = 32° to 33°; average 29°; C - Range = 0.07 to 0.29 ksf; average 0.18 ksf
- Passing No. 200 Sieve (two tests) - 37%
- Coef. Perm (two tests) - Range $1.4 \times 10^{-4}$ to $3.4 \times 10^{-4}$ cm/sec; average $2.4 \times 10^{-4}$ cm/sec
- Non-petroliferous

C (Fernando Formation)

| | |
|---|---|
| • Unconfined Compressive Strength (Qu) (20 tests) | - Range = 9.5 to 182 psi; average 124 psi |
| • Passing No. 200 Sieve (one test) | - 78% |
| • Coef. Perm. (two tests) | - Range $6.1 \times 10^{-8}$ to $1.1 \times 10^{-7}$ cm/sec; average $8.8 \times 10^{-8}$ cm/sec |
| • Faults | - None known |
| • Bedding | - Dip from horizontal 10° to 50°; probably southward |
| • Occasional Hard Sandstone Beds | - 1 in to 3 ft thick; quantity unknown but are less prevalent than in Puente Formation |
| • Permanent Water Level | - About 150 ft below ground surface; formation tight; expect seeps |
| • Water Quality | - Poor, up to 6,926 ppm TDS in Boring CEG 16 |
| • Gas Encountered | - Minor in Boring CEG 18 |
| • Oil | - Not encountered |

### 5.2.5 Reach 5 - La Brea Area (Borings CEG 19 through 23A)

This reach of the alignment trends westward along Wilshire Boulevard and curves northward along Fairfax Avenue to Melrose Avenue. The reach crosses several notable geologic features including anticlines and synclines, tar sands, the Salt Lake Oil Field and numerous faults, particularly the Santa Monica fault zone. Geologic units $A_2$, $A_4$, SP and C are encountered. Important characteristics are:

$A_2$ (Young Alluvium) Borings CEG 23 and 23A only

| | |
|---|---|
| • Undrained, Quick, Direct Shear (one test) | - $\phi$ = 21°<br>C = 1.05 ksf |
| • No Petroleum | |
| • Perched Water Level | - 10 to 20 ft below ground surface (north portion of reach only) |

$A_4$ (Old Alluvium)

| | |
|---|---|
| • Undrained, Quick, Direct Shear (ten tests) | - $\phi$ - Range = 4.5° to 39.5°; average 32°<br>C - Range = 0.26 to 2.64 ksf; average 1.2 ksf |
| • N-Value | - Range = 14 to 92 bpf; average 50 bpf |
| • Passing No. 200 Sieve (three tests) | - Range = 9% to 53%; average 31% |
| • Coef. Perm (two tests) | - Range = $2.0 \times 10^{-7}$ to $5.3 \times 10^{-7}$ cm/sec; average $3.6 \times 10^{-7}$ cm/sec |
| • Swell (four tests) | - Range = 0 to 4%; average 2.0% |
| • Non bouldery | - None encountered in borings |
| • Petroliferous | - Borings CEG 19 and 20 |
| • Faults | - Judged to offset $A_4$ by Santa Monica fault; act as gas traps |
| • Perched Water Level | - 15 to 35 ft below ground surface (south portion of reach only) |

SP (San Pedro Formation, Sand) - La Brea Tar Pit Area

- Unconfined Compressive Strength (Qu) (one test) - 20 psi
- Undrained, Quick Direct Shear (seven tests) - $\phi$ - Range = 29° to 35°; average 32°
  C - Range = 0.25 to 0.56 ksf; average 0.39 ksf
- N-Value - Range 8 to >100 bpf; average 50 bpf
- Coef. Perm. (one test) - $8.8 \times 10^{-5}$ cm/sec
- Petroliferous - In all borings
- Gas - In all borings
- Depth Encountered - 40 to >200 ft below ground surface
- Faults - Judged to offset San Pedro Formation by Santa Monica fault vicinity Borings CEG 23 and 23A; width of faulted material judged to be 10 to 20 ft thick and "soft" tar sand not likely to be fractured but fault a likely gas trap

C (Fernando Formation)

- Unconfined Compressive Strength (Qu) (18 tests) - Range = 3.5 to 99 psi; average 56 psi
- Undrained, Quick, Direct Shear (three tests) - $\phi$ - Range = 30° to 41°; average 36°
  C - Range = 0.62 to 1.33 ksf; average 0.94 ksf
- Passing No. 200 Sieve - 97%
- Swell - 1%
- Coef. Perm. (two tests) - $4.1 \times 10^{-8}$ cm/sec
- Hard Sandstone Beds - Borings CEG 20, 21 and NRC-5900; quantity unknown
- Faults - 6th Street, 3rd Street, San Vicente; will be encountered if track grade deeper than about 150 ft; Santa Monica fault not likely to be avoided if track grade deeper than 40 to 50 ft vicinity Borings CEG 23, 23A; zone of intensely fractured formation could be 2,000ft and act as traps for gas
- Permanent Ground Water Level - About 150 ft below ground surface between Borings CEG 19 and 21
- Water Quality - Poor to very poor; TOS 15,425 ppm Boring CEG 19
- Gas - In all borings
- Oil - In all borings.

### 5.2.6 Reach 6 - Hollywood Area (Borings CEG 24 through 28)

This portion of the alignment is bounded by the Santa Monica fault zone on the south (near Melrose Avenue) and the Hollywood fault on the north (near Yucca Street), at the base of the Santa Monica Mountains. This reach is mantled by thick deposits of geologic units $A_2$, $A_3$ and $A_4$. Important characteristics are:

## $A_2$ (Young Alluvium)

- Unconfined Compressive Strength (Qu) (one test) - 72 psi
- Undrained, Quick, Direct Shear (six tests) - ø - Range = 20° to 39.5°; average 30°; C - Range = 0.18 to 1.12 ksf; average 0.70 ksf
- Passing No. 200 Sieve (two tests) - 53%
- Coef. Perm. (one test) - $2.2 \times 10^{-5}$ cm/sec
- Transmissivity - Estimated to be up to 50,000 gpd/ft
- Non-bouldery
- Water Content - May store sizeable quantities of water (e.g. inflows of 1,400 gpm); similar to water encountered in MWD's San Fernando Tunnel (Section 6.1.3)
- Perched Water Level - 20 to 30 ft below ground surface

## $A_4$ (Old Alluvium)

- Unconfined Compressive Strength (Qu) (14 tests) - Range = 28 to 94 psi; average 50 psi
- Undrained, Quick, Direct Shear (10 tests) - ø - Range = 5° to 40°; average 29°; C - Range = 0.64 to 2.0 ksf; average 1.35 ksf
- N-Value - Range = 3 to 86 bpf; average 40 bpf
- Passing No. 200 Sieve (seven tests) - 52%
- Coef. Perm. (two tests) - $1.0 \times 10^{-7}$ cm/sec
- Non-bouldery
- Fault - $A_4$ displaced by Hollywood fault and abruptly abuts Topanga Formation, creating a possible saturated and certainly a mixed face condition; potentially 1,000 ft wide
- Gas - None encountered in borings
- Oil - None encountered in borings

### 5.2.7 Reach 7 - Santa Monica Mountains (Borings CEG 28A through 34)

Reach 7 is about 3.3 miles long and contains the following hard rocks, to which Terzaghi Rock Condition Numbers 1 through 5 are applicable (see Drawing 2). This reach can be subdivided into three rock units. From south to north, they are shown in Table 5-15.

Table 5-15 Rock Condition No. - Reach 7

| Miles | Rock Formation | Estimated Terzaghi Rock Condition No. |
|---|---|---|
| 0.8 | Topanga sandstone/conglomerate | $4_w$ & $5_w$ |
| 1.1 | Topanga basalt | $1_w$ & $3_w$ |
| 1.4 | Topanga sandstone/conglomerate | $2_w$ & $4_w$ |

Within this 3.3-mile reach, the tunnel should pass through five known faults (Drawing 2) and possibly a few unknown faults. Several other small faults are shown on the Los Angeles City geologic map (1970), Sheet Nos. 136 and 159. These are not confirmed.

Appendix J, Volume II, Petrology, presents the petrographic analyses of mineral constituents which may be helpful to TBM cutter design; i.e., percentage of quartz and abrasive minerals. Robbins, Inc. test indicated Topanga sandstone in Boring CEG 32A contained 10 percent to 20 percent silica ($SiO_2$), and weak Topanga sandstone in Boring CEG 33 contained 20 percent to 40 percent silica (unconfined compressive strength 9,000 psi), whereas there is no silica in the basalt from Boring CEG 32.

Shearing, chaotic bedding, brecciated sandstone and slickensided surfaces were encountered in Boring CEG 29. Cores in Boring CEG 30 and 31 were intensely fractured and locally sheared. This is interpreted to mean the Hollywood Bowl fault has broken these rocks and locally produced a 3-foot thick green clay gouge at a depth of 160 to 163 feet.

The Los Angeles City Sewer Tunnel was driven (conventionally) with relative ease through these two unnamed faults and the Benedict Canyon fault without squeezing ground, with only minor "heavy" ground intervals and a maximum ground water inflow of 600 gpm (for a few hours to two days ) from these faults (Section 6.3 and Figure 6-2). Similar conditions were encountered in the MWD Hollywood Tunnel (Section 6.4). Judging from this, the two unnamed faults and the Benedict Canyon fault may not pose much of a problem; however, the physical description of rock core encountered in the Hollywood Bowl and Hollywood fault (zone) suggests there is a high potential for very blocky and seamy rocks. These very blocky rocks undoubtedly store water, and inflows of 500 gpm might well be expected for a few days.

Conglomerates containing hard gravel and cobbles were encountered in Borings CEG 33 and 34. Additionally, hard boulders in the conglomerate up to 3 feet in diameter were commonly observed in outcrops along Cahuenga Boulevard.

No oil or gas was encountered, but the presence of 0.06 percent oil by weight in the core from Boring CEG 31 may be a slight indication of gas in the sedimentary rocks south of the Hollywood Bowl.

### 5.2.8 Reach 8 - San Fernando Valley (Borings CEG 35 through 38)

Reach 8 is about 2.2 miles long and extends northward into the San Fernando Valley with thick deposits of geologic units $A_1$, $A_2$ and $A_3$. The units overlie bedrock which dips northward under the Valley. The Los Angeles River channel is traversed also. Important characteristics are:

## $A_1$ (Young Alluvium)

- Undrained, Quick, Direct Shear (Qu) (two tests) – $\phi = 38°$; C – 0.43 ksf
- N-Value – Range = 9 to >100 bpf; average 50 bpf
- Passing No. 200 Sieve (four tests) – Range = 8% to 86%; average 26%
- Coef. Perm. (two tests) – $2.8 \times 10^{-4}$ cm/sec
- Estimated Transmissivity – Up to 200,000 gpd/ft
- Modern L.A. River Channel Deposits – Up to 90 ft deep
- Bouldery Ground – Should be expected, but none was encountered in borings
- Permanent Water Level – 30 ft below ground surface in Los Angeles River area deepening to 120 ft from Boring CEG 35 to 38
- Water Quality – Poor, average TDS exceeds 1,000 ppm

## $A_2$ (Young Alluvium)

- Undrained, Quick, Direct Shear (five tests) – $\phi$ = Range = 32.5° to 39°; average 36°; C = Range = 0.19 to 0.56 ksf; average 0.41 ksf
- Occurrence – As scattered lenses in $A_1$

## $A_3$ (Old Alluvium)

- Unconfined Compressive Strength (eight tests) – Range = 9.3 to 46.2 psi; average 28 psi
- Undrained, Quick, Direct Shear (Qu) (16 tests) – $\phi$ – Range = 6.5° to 40.5°; average 34°; C – Range = 0.16 to 1.40 ksf; average 0.48 ksf
- Passing No. 200 Sieve (three tests) – 13%
- Coef. Perm. (one test) – $6.4 \times 10^{-5}$ cm/sec
- Non-bouldery – None encountered in borings
- Permanent Water Level – 120 ft below ground surface
- Faults – Two unnamed faults postulated in $A_3$ by others; physical properties are unknown; not likely to be encountered if track grade is 100 ft below ground surface

# Section 6.0
# Previous Tunneling Experience in Area

# Section 6.0: Previous Tunneling Experience in Area

Los Angeles is one of the world's largest cities in area, and with a metropolitan population of over seven million people. More than 60 tunnels, with total length greater than 50 miles, have been bored within the city limits (Proctor, 1973). This history of local tunneling experience was reviewed, with particular attention to case histories that involved geologic formations or settings similar to those anticipated along the Metro Rail alignment. The tunnels selected for detailed study and presented herein are:

- Metropolitan Water District of Southern California (MWD) San Fernando Tunnel
- MWD Newhall Tunnel (LAC)
- City of Los Angeles Sewer Tunnel
- MWD Hollywood Tunnel
- Los Angeles County Flood Control District (LACFCD) Sacatella Tunnel
- MWD Tonner Tunnel

The use of case histories can provide general information on the response of the materials to excavation, methods of excavation and support, construction problems, rates of advance and costs. However, overall success or failure of each tunnel project was dependent on the site-specific geology, methods of excavation and support employed, and organization of the contractor.

## 6.1 MWD SAN FERNANDO TUNNEL

### 6.1.1 Facts and Figures

| | |
|---|---|
| Tunnel Length | 29,100 ft (5.5 miles) |
| Tunnel Diameter | 22 ft O.D. excavated; 18 ft I.D. concrete lined |
| Initial Support | Precast concrete segmented ring<br>4 segments/ring; segments 8 in. thick & 4 ft wide |
| Excavation Method | Digger-type Robbins excavator mounted in a shield<br>(the last 1,000 ft were conventional drill & blast) |
| Geology | Soft sandstone & siltstone<br>(Pico Formation) & Old Alluvium |
| Eventual Use | MWD water supply tunnel |
| Contractor | Lockheed Shipbuilding & Construction Company<br>Seattle, Washington |
| Bid Price | $19,346,800 on February 20, 1969 |
| Estimated Bid Completion Time | 1,360 working days |
| Tunneling Period | January 1970 to November 1975; gas explosion stopped work for 27 months |

6.1.2 Major Progress, Prior to Ground Water Problem

During the latter period of the first 17,000 feet of tunneling, record advances were made using a Robbins-built shield, swinging and sliding boom with a rotatable digging head:

104 feet in one 8-hour shift
277 feet in one 3-shift day
1,077 feet for a 5-day week
3,500 feet for one month.

A special tunnel-liner erector installed 4-foot-long concrete segments at a rate of 12 per hour over an 80-hour week to support the shield's 144-hour weekly operation (Construction Methods and Equipment, January 1971, p. 1).

Excavated material consisted of dry and water-bearing Old Alluvium sand, with minor gravel and cobbles, lightly cemented to the extent that slight ripping was required.

6.1.3 Ground Water in Old Alluvium

Just west of the North Olive View fault (see Figure 6-1) substantial amounts

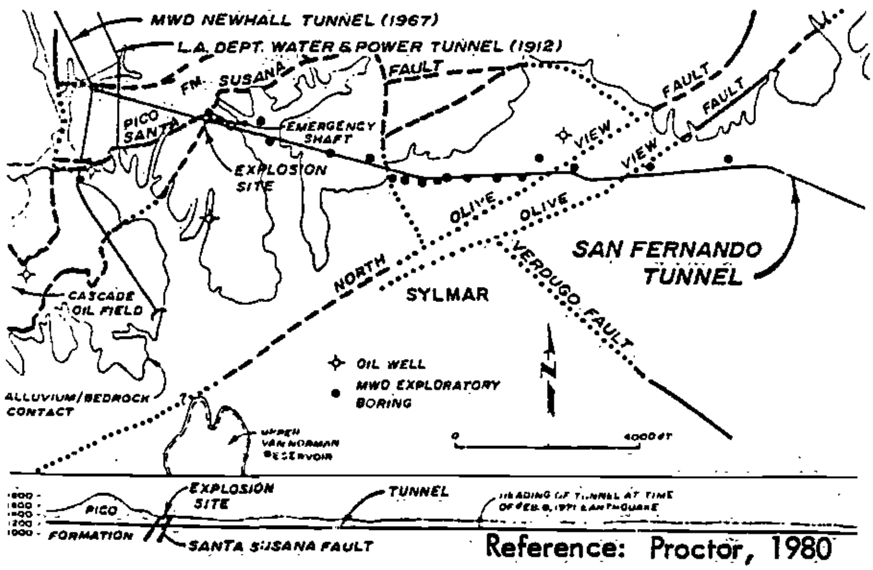



Fig.6-1 *Map and section along part of the San Fernando Tunnel.*

of water (1,400 gpm peak inflow) were forecast and encountered in the Old Alluvium. In this area, tunnel cover was about 140 feet. Prior to encountering this water, rate of advance was 150 to 200 feet per 3-shift day; in the water zone, the advance rate was still a respectable 60 feet per 8-hour shift; the other two shifts drilled horizontal dewatering holes ahead of the shield.

The following are excerpts from <u>California Builder and Engineer</u>, March 26, 1971:

- In late 1970, the excavation encountered perched ground water that sprayed the miners with water under slight pressure.
- Water pressure was relieved by three-inch borings drilled horizontally ahead of the excavator 150 to 250 feet.
- Two-inch perforated plastic pipe was inserted to drain ahead of the heading. This reduced the digging to a single shift per day and slowed progress considerably.

### 6.1.4 Solution to Ground Water Problem

(The following are excerpts from B.P. Boisen of A.A. Mathews, Inc., Memorandum, March 22, 1971, p. 5).

There are several perched water tables in the tunnel area, and test holes by the owner had pinpointed them before Lockheed Shipbuilding & Construction Co. (LSCC) started work. The solution was horizontal dewatering because an impervious stratum just beneath the tunnel grade would have made dewatering by wells ineffective. Drilling ahead of the shield brought the water around it and into the tunnel where it could be pumped out.

- Horizontal holes were drilled from 150 to 250 ft ahead of the shield, i.e., starting about 30 ft back of the face, running as close to the tunnel edge as possible. The drill stem cut a 4-in-diameter hole and 10-ft-long, 1.5-in-diameter hollow slotted plastic pipe sections were driven inside the drill stem.
- Water drained into the pipe slots and into the tunnel. There, a pair of pumps discharged the water into a surface flood control channel and through holes drilled down from a street above.

### 6.1.5 Effects of San Fernando Earthquake

On February 9, 1971, a 6.4 Richter Magnitude earthquake occurred, with an epicenter only a few miles from the San Fernando Tunnel. The following was noted from excerpts of <u>Engineering News Record</u>, June 24, 1971, p. 25:

- The tunnel did not suffer any major damage, however, the shield was squeezed laterally and tightly bound in place. It took two weeks to free the shield by jetting along the skin at three o'clock and nine o'clock.
- Tunnel work continued, with all workmen on hand, after a three-day shutdown to assess damage.
- When level coordinates were shot from undisturbed benchmarks, it showed that the east portal elevation was 7.2 ft higher than its original position. The gate structure shaft, about midway through the tunnel, was 2.5 ft higher. And the west portal, where the bore ties into Magazine Canyon shaft and Newhall Tunnel, was up 1.25 ft.

- There were no visible changes in the tunnel, largely due to the articulated precast tunnel supports. The segments are independent of each other, and there was opportunity for movement.
- Nearly 3.5 miles of tunnel were finished when the quake struck and the 7-ft uplift can be absorbed in moving individual segments such a slight amount that it would not be detected by the naked eye.

### 6.1.6 Caving Problems

(The following are excerpts from B.P. Boisen of A.A. Mathews, Inc., Memorandum, March 22, 1971, p. 4).

In the Old Alluvium west of the North Olive View fault, where the ground water was encountered, the contractor was plagued with problems. Approximately nine times there were runs at the face of several thousand gallons of water, sand and gravel. A number of times a cave would develop up to 40 feet above the shield. When the shield had advanced beyond these runs, the contractor would drill a hole from the surface and drop sand into the void. Grout was then pumped into the sand. On one occasion, a 10-ft-diameter cave worked its way to the surface of Foothill Boulevard. The contractor poured sand into the void and added a surcharge of sand about 8 feet above the street. The next day the sand had settled to around 12 feet below the street.

The shield was equipped with six breast doors that were used to support the upper half of the face. Generally the contractor advanced the shield through these troublesome zones with the breast doors closed and the apron full of muck. This then imposed a greater thrust on the precast segments during shoves. This was when the contractor started using 10 inch-thick segments (instead of original 8 inch-thick segments) to develop the thrust necessary to shove the shield.

### 6.1.7 Fatal Gas Explosion

(The following are excerpts from R.J. Proctor, 1980, "San Fernando Tunnel Explosion", Underground Space, v. 4, no. 4, p. 217-219).

At 12:30 a.m. on the morning of June 24, 1971, a fatal gas explosion occurred in a Los Angeles area tunnel. Of the heading crew, 17 workers died and one survived. The explosion halted work for 2 1/2 years due to settlement discussions and new contractual agreements. Only 2,500 feet remained to "hole-through" the 5.5-mile tunnel. The explosion occurred near the Santa Susana fault (see Figure 6-1). Several events occurred that provided evidence of a possible gas hazard. The MWD geologic report, given with the specifications to all bidders, warned of the possibility of encountering oil and/or gas in the western part of the tunnel route. This warning was based on:

- producing oil fields in the region, one within 1.7 miles
- oil and tar seeps in the area
- the presence of Pico Formation sandstone, a known producer of oil in the western part of the tunnel route

- the presence of oil and gas in two nearby tunnels - The L.A. Dept. of Water and Power's Newhall Tunnel in 1912, and the MWD's Balboa Tunnel in 1967
- the location of the Santa Susana fault, which acts as an oil trap for the nearby Cascade oil field
- several months prior to the explosion, the contractor posted a notice that stated "expect explosive gas ahead"
- one day before the explosion, a core with "kerosene or diesel smell" was extracted from the face
- one day before the incident, a minor gas explosion occurred that sent four miners to the hospital.

Work resumed 27 months after the disaster, and was completed in November 1975. Most of the interval was spent working out an agreement between the owner and the contractor on procedures and costs, but without the admission of liability by either side.

To complete the tunnel, a board of tunnel consultants (J. Barry Cooke, Lyman D. Wilbur and J. Donavan Jacobs) was convened. Their recommendations, all of which were complied with, included:

- increasing the ventilation system from the rated 35,000 cfm to 70,000 cfm [This can move air down the tunnel at the rate of 200 ft per min.]
- building a remote hydraulic system to power the repaired boring machine
- installing a multiple-head constant monitoring system for gas, plus two full-time sniffer men
- drilling at all times during tunnel excavation at a minumum of 20 ft into the face [Contractor elected to drill 80-ft holes and use up 60 ft before redrilling.] Four holes should be drilled into the face while in the Santa Susana fault zone. [Clay in the fault planes may accumulate gas behind them.]
- limiting daily advances to 20 ft [After this restriction and the TBM were removed, maximum progress was no more than 25 ft daily.]
- drilling a ventilation shaft 150 ft deep, and incorporating a rescue chamber and emergency ladder 600 ft back of the heading.

The following are excerpts from B.P. Boisen of A.A. Mathews, Inc., Memorandum, March 27, 1974, p. 2-3:

After the explosion, and upon resumption of tunneling, with increased ventilation gas monitors did not detect concentrations greater than 7 percent LEL, and gas was not encountered when the heading was beyond the Santa Susana fault zone and into the siltstones, sandstones and conglomerates of the Pico Formation.

In the opinion of B.P. Boisen of A.A. Mathews, successful tunneling in gassy conditions is possible and can be attributed to:

| | |
|---|---|
| Adequate Ventilation | 90% |
| Constant Testing and Monitoring | 9% |
| Use of Permissible Equipment | 1% |

The 1% of success due to using permissible equipment is the major cost factor and is looked upon with the most dislike.

## 6.2 MWD NEWHALL TUNNEL

### 6.2.1 Facts and Figures

| | |
|---|---|
| Tunnel Length | 3.5 miles (Magazine Canyon to City of Newhall) |
| Tunnel Diameter | 26 ft O.D. excavated; 20.5 ft I.D. |
| Initial Support | Steel ring beams & wood lagging; also precast concrete lining 3 segments/ring, 12 in. thick & 4 ft wide |
| Excavation Method | By Kiewit: a Calweld oscillating cutter head TBM (8000 ft)<br>By Dixon: a Calweld rotary cutter head TBM (10,000 ft) |
| Geology | Hard to soft sandstone (Pico Formation) & gravelly, cobbly sandstone (Saugus Formation) with oil seeps |
| Eventual Use | MWD water supply tunnel |
| Contractor | L.E. Dixon Co., Arundel Corp., MacDonald & Kruse, Inc., & Peter Kiewit Sons' Co. (joint venture) |
| Bid Price | $35,000,000 awarded June 1966 |
| Tunneling Period | 1966-1970 |

### 6.2.2 Excavation Progress

(The following are excerpts from Maynard M. Anderson, Civil Engineering, Sept 1970, p. 69).

- After rotary boring 10,000 feet, the contractor chose to abandon the "mole", since the weakly consolidated sedimentary rock would frequently slough ahead and above the cutting wheel, stalling the machine and necessitating extensive hand-cribbing of the overbreak. Despite this set-back, the average driving rate was probably better than what it would have been with conventional tunneling. Through ideal ground for the "mole" [CWDD note: in dry, moderately consolidated siltstone and sandstone], it bored over 100 ft in a single 3-shift day.
- Later [September 1969] the contractor started a second heading using a Calweld oscillating mining machine. This machine is mounted in a 26-ft O.D. full-circle shield that thrusts against a tunnel support system almost identical to the Tabor (MEMCO) tunnel lining. Excellent progress was made ... about 100 ft per working day ... through soft but well-standing sedimentary rock.

(The following are excerpts from LeVitt, R.R. of A.A. Mathews, Inc., Memorandum, June 2, 1970, p. 3-4).

- Some sandstone at the face was so hard that it was broken down only by the shearing action of the drag cutters on the [oscillating] cutter head ... lenses of cobbles were noted in the face and these were being sheared off or dislodged with some difficulty ... the contractor has not experienced any severe ground water conditions ... the ground generally has been a fine-grained dense sandstone with a very low percolation rate.
- Pushing the Calweld rotary shield through the earth required a very heavy thrust [nearly 4,000 tons] against a firm base. The contactor felt the walls of the bore were too soft to serve as such a base, therefore, it was decided to axially thrust against the tunnel support system. This necessitated moving the wooden lagging from outside to between the circular steel ribs, where the "tight" lagging would provide support for the forward thrust of the shield.

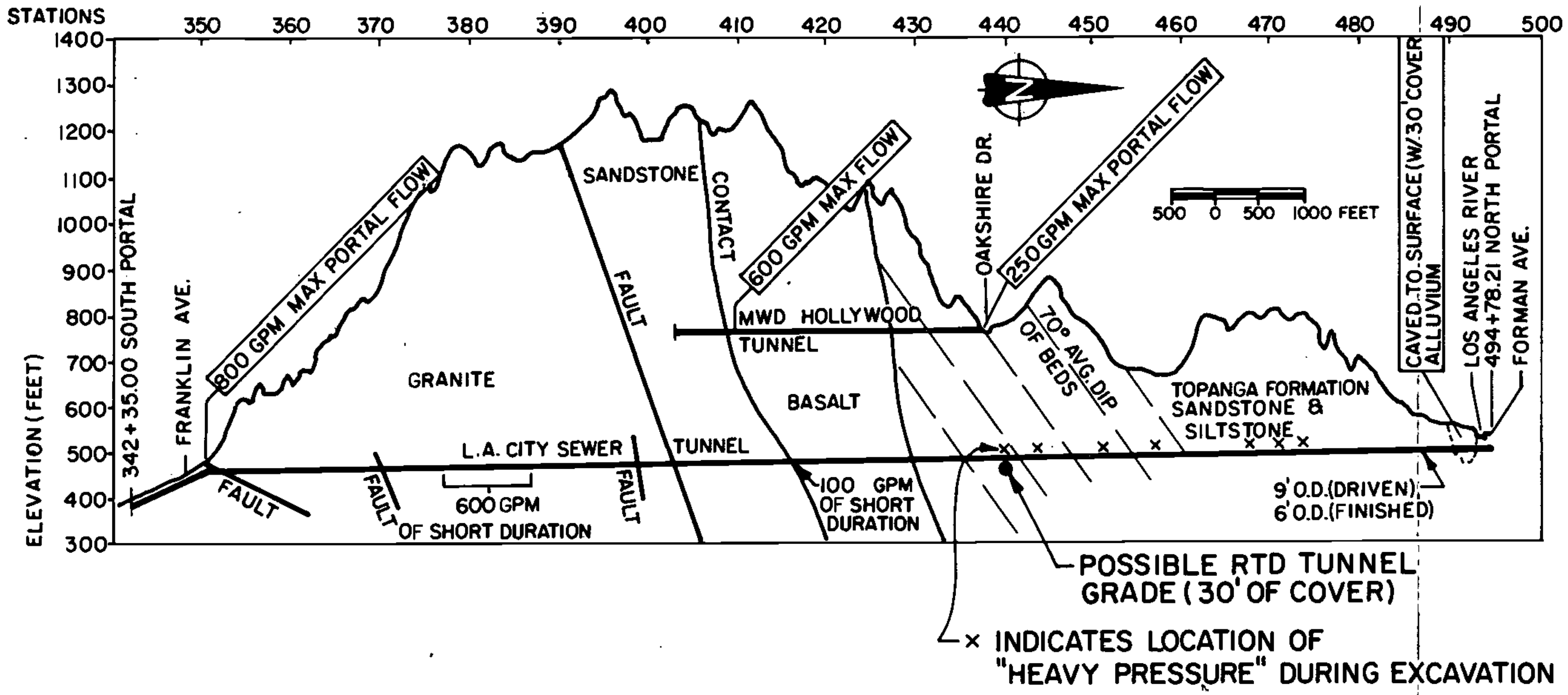
## 6.3 LAC (LA CIENEGA - SAN FERNANDO VALLEY) SEWER TUNNEL

### 6.3.1 Facts and Figures

| | |
|---|---|
| Tunnel Length | 2.8 miles |
| Tunnel Diameter | 9.0 ft O.D. excavated |
| Initial Support | 4 in. steel ribs & wood lagging; when soft ground encountered, steel liner plates |
| Excavation Method | Conventional jumbo drill & blast |
| Geology | Conglomerate, Sandstone,<br>Shale (Topanga formation) . . . . 8,000 ft<br>Granite . . . . . . . . . . . . . . 4,700 ft<br>Basalt . . . . . . . . . . . . . . 1,200 ft<br>Young Alluvium . . . . . . . . . . 430 ft<br>14,330 ft |
| Eventual Use | Sewer tunnel, City of Los Angeles |
| Contractor | L.E. Dixon Co.; lining, Kemper Construction Co. |
| Bid Price | $3,200,000 |
| Tunneling Period | 1954-56 |

### 6.3.2 Relation to Metro Rail Alignment

Except for the granite, Reach 7 of the Metro Rail alignment will pass through geologic formations similar to those encountered in this tunnel. Reach 7 is also very close to the Los Angeles City Sewer Tunnel (see Drawing 1 for location and Drawings 2 and 11 for subsurface relationships). Some recorded ground water inflows and geologic formations are presented in the profile on Figure 6-2.




NOTE: See Drawing No. 1, "Geologic Map" for location of City Sewer Tunnel

REFERENCE: 1. L.A. City "Key Plan & Profile", La Cienega & San Fernando Valley Relief Sewer, Drawing No. D-10565 Sheet 2 of 17 8-12-53

2. Campbell, D.G., 1955, Geologic Notes and Log of Los Angeles Sewer Tunnel

## LOS ANGELES CITY SEWER TUNNEL PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

| | |
|---|---|
| Scale — | Project No. 80-1280 |
| Date NOV., 1981 | Figure No. |
| Prepared by J.A.C. | |
| Checked by H.A.S. | 6-2 |
| Approved by H.A.S. | |

Form No. D60/78

This sewer tunnel encountered inflows greater than 100 gpm for a few hours to one day at seven locations along its 2.8-mile length. Maximum water flow from the down gradient south Portal was reported to be 1,200,000 gallons per day (gpd), or 800 gpm, but averaged 400,000 gpd (see Figure 6-2). Based on the final selected tunnel grade, the Metro Rail alignment will probably pass under the existing Los Angeles City Sewer tunnel. "Heavy ground pressures" were reported in the shaley parts of the Topanga Formation and in the sheared granite.

### 6.3.3 Problems in "Muddy" Young Alluvium - North End

The contractor installed steel ribs and lagging progessing southerly to Station 491+33 near the Los Angeles River at North Hollywood. At this point, under shallow cover, a wet, muddy Young Alluvium channel was encountered which had not been indicated in the geologists' report. The formation was chiefly mud, and the loads imposed on the steel ribs caused some sets to fail. This created an emergency condition, causing caving to the surface (see Figure 6-2) and the contractor installed heavy timber sets. This proved successful.

## 6.4 MWD HOLLYWOOD TUNNEL

### 6.4.1 Facts and Figures

| | | |
|---|---|---|
| Tunnel Length | 0.7 miles | |
| Tunnel Diameter | 8.0 ft | |
| Initial Support | Steel ribs & wood lagging | |
| Excavation Method | Drill & blast | |
| Geology | Conglomerate, Sandstone, | |
| | Shale (Topanga formation) . . . . | 1,600 ft |
| | Basalt . . . . . . . . . . . . . . | 2,100 ft |
| | | 3,700 ft. |
| Eventual Use | MWD water supply tunnel | |
| Contractor | J.F. Shea Co., Inc. | |
| Bid Price | $190,000 | |
| Tunneling Period | June 1940 - May 1941 | |

### 6.4.2 Relation to Metro Rail Alignment

The Hollywood Tunnel is located approximately 400 feet above the proposed Metro Rail alignment (see Drawing 1 for location). The basalt unit was mostly hard and dense and was described as good blasting rock (but contains soft ash and volcanic breccia layers, as revealed in logs of the tunnel). Water seeps were common, and on one occasion a maximum temporary inflow of about 600 gpm was recorded at the sandstone/basalt fault contact at the north end. No major faults were encountered and no squeezing ground was encountered. A geologic profile and plan view are presented on Figure 6-3.

## 6.5 LACFCD SACATELLA TUNNEL

### 6.5.1 Facts and Figures

The following tunneling data were received in an oral communication in June 1981 with the contractor, Donald Glanville of Glanville Construction Company, and John E. Witte, Tunnel Consultant, as well as LACFCD Pre-construction "Geologic Report", dated December 26, 1973; and Victor L. Wright's "Pre-Bid Geologic Appraisal" report, dated July 1975.

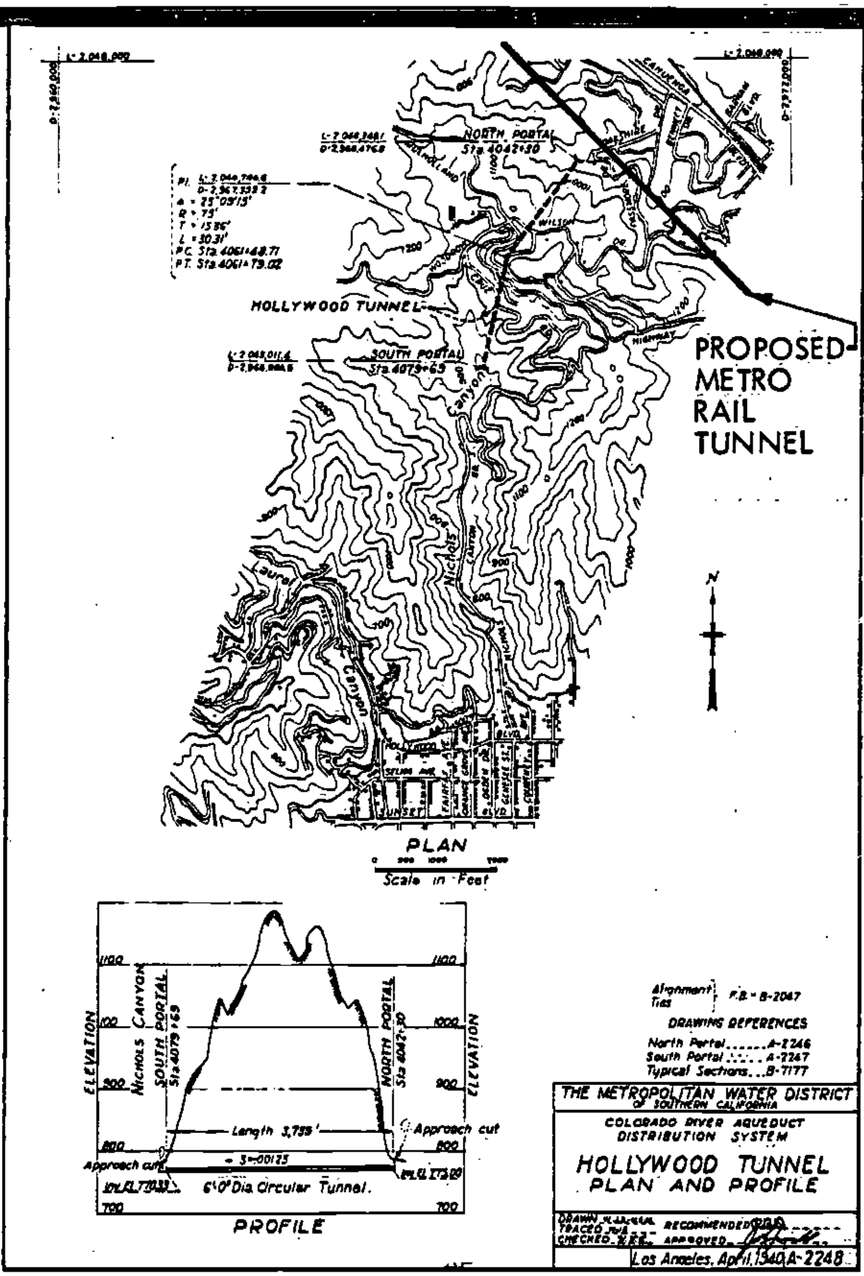
| | |
|---|---|
| Tunnel Length | 0.6 miles |
| Tunnel Diameter | 18 ft O.D. excavated; 14.5 ft I.D. |
| Initial Support | Precast concrete liner (3 segments/ring) |
| Excavation Method | Digger Gradall & shield |
| Advance Rate | Maximum 32 ft/8-hr shift; average 15 ft |
| Geology | Claystone, siltstone & occasional interbeds of very hard "calcareous" cemented sandstone |
| Eventual Use | Storm drain, LACFCD |
| Contractor | Glanville Construction Co. |
| Bid Price | ±$4,000,000 |
| Extras Awarded | ±$500,000 |
| Tunneling Period | 1975-77 |

### 6.5.2 Relation to Metro Rail Alignment

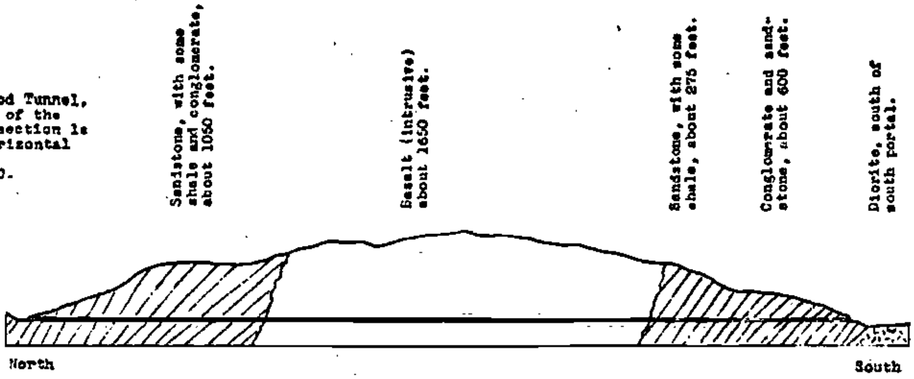
The Los Angeles County Flood Control District's (LACFCD) Sacatella Tunnel is in litigation for "changed (geologic) conditions" in the tunnel (settled) and at both portals (unsettled). For this reason, the LACFCD was reluctant to release information.

Geologic conditions and tunneling methods in this tunnel are very important to the Metro Rail alignment because:

- Tunnel was excavated in a "gassy" reach under Hoover Street, north of Wilshire Boulevard, in claystone, siltstone and sandstone of the Puente Formation (Unit C), at the location shown on Drawing 1.




Geologic Cross Section of Proposed Hollywood Tunnel, showing rocks to be traversed and attitude of the sedimentary strata. Elevation of base of section is 700 feet above sea level. Vertical and horizontal scale: 400 feet equals 1 inch.
Geology by John P. Buwalda. April 15, 1940.




## MWD HOLLYWOOD TUNNEL - GEOLOGIC PLAN AND PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No.
6-3

- Formation is similar to the material anticipated in Metro Rail alignment Reaches 1 to 5, inclusive, whether deep or shallow; the equivalent formation is designated Unit Cw on Metro Rail Project Drawing 2.
- Total cover above tunnel crown ranges from 22 to 25 feet.
- Total bedrock cover above tunnel crown ranges from 2 to 25 feet.
- Old Alluvium (Qalo) cover above the tunnel crown ranges from 5 to 32 feet.

### 6.5.3 Peak Unconfined Compressive Strength

LACFCD test results of peak unconfined compressive strength, from six core samples obtained in the Puente Formation, are tabulated as follows:

| LACFCD Boring | Unconfined Compressive Strength, Qu (psi) |
|---|---|
| 1 | 401 |
| 1 | 603 |
| 2 | 441 |
| 2 | 384 |
| 2 | 377 |
| 7 | 172 |
| Average | 396 |

Core samples from Borings 1 and 2 were taken essentially normal to the bedding, while the bedding at Boring 7 was inclined at about 45 degrees from the long axis of the core. This probably accounts for the considerably lower compressive strength test value for the sample from Boring 7. All core segments tested were selected for cross-sectional uniformity and freedom from cracking or damage and, as such, are considerably more competent than the average grade of rock encountered during drilling. Therefore, the values obtained for the compressive strength are probably greater than the average values which would be found during tunnel excavation (LACFCD, 1973).

### 6.5.4 Digger Excavator and Shield

The tunnel excavation was performed with a small (Model No. 2403) Gradall excavator. The rotating, telescoping boom was connected to a flat plate that had a single ripper tooth on one edge and several digger teeth on the other edge (Figure 6-4). Also note in Figure 6-4 Puente Formation bedding (same as Metro Rail alignment, Unit C) and lack of ground water inflow.

Digger Excavator
(LACFCD Sacatella Tunnel)

Figure 6-4

### 6.5.5 Geology

Puente Formation: Thin bedded, soft claystone and siltstone. The formation contained occasional interbeds of very hard "calcareous" cemented sandstone from 2 to 12 inches in thickness with unconfined compressive strength of 5,000 to 15,000 psi. These interbeds caused the "changed conditions", according to Donald Glanville, as they were not mentioned in the pre-construction reports. Some very hard interbeds were nearly horizontal and followed the face for several hundred feet; some were at a 45° angle to the tunnel alignment and followed the face for several tens of feet. This resulted in the following actions:

° replaced single-tooth ripper with hydraulic jackhammer to break up hard layer (removed jackhammer in weak ground)

° bent leading edge of shield, forcing contractor to stop and repair often; i.e., spent 8-hour shift digging and balance of day repairing shield

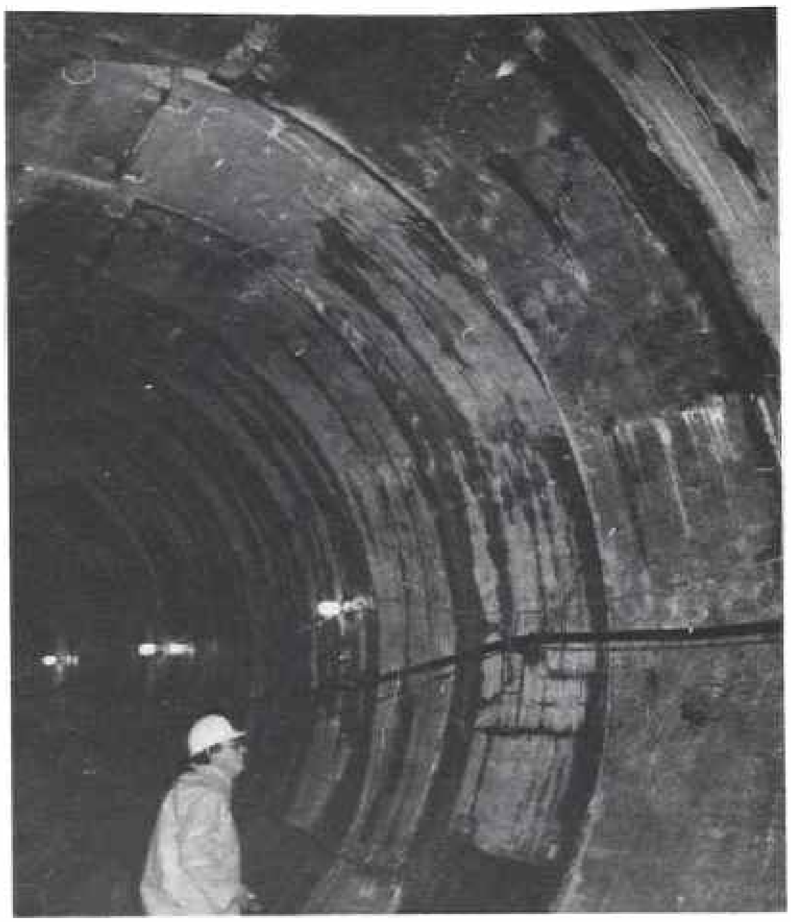
° difficult to maintain line and grade in hard rock layers (These hard layers, although 12 inches or less in thickness, made drilling of 5-foot diameter man-way shafts very difficult also.)

° advance rate cut drastically; i.e., often reduced advance rate to 1 to 5 feet daily.

### 6.5.6 Tunnel Gas Classification

The tunnel was classified "gassy" because it traversed the Los Angeles City Oil Field (see Drawing 1). No fire or explosion occurred during the project.

- The greatest apparent risk is where folding and a suspected fault may form significant traps (Wright, 1975, p. 8). Explosive-proof equipment was installed (although arc welding was permitted in the tunnel).
- Face was continuously monitored by a gas "sniffer" that automatically set off an alarm if high LEL readings were recorded. (Note: Alarm was never activated because ventilation was so effective.)
- Installed 4-foot-diameter ventilation duct and pumped air at 400 fpm through the vent pipe.
- Oil, seeping down the sides of the supports, was skimmed off the discharge water at the portal and hauled away by tank truck (personal communication, R.J. Proctor, 1981). Oil seeps are shown on Figure 6-5.



Oil Seeps
(LACFCD Sacatella Tunnel)

Figure 6-5

### 6.5.7 Abandoned Oil Wells

The tunnel encountered several uncharted, uncased, abandoned oil wells. Although oil was not encountered in these holes, several hundred gallons of water gushed into the tunnel for a few seconds, alarming the miners each time.

### 6.5.8 Ground Water

The tunnel was below a "permanent" water table. The water table was in the Puente Formation and the overlying Old Alluvium. The contractor drilled 12 dewatering wells at selected locations along the alignment prior to excavating the tunnel. This dewatering of twelve 24-inch-diameter wells, recommended by Vic Wright, Tunnel Consultant, appears to have successfully kept tunneling conditions in the "dry". According to Wright, 1975, "... ground water problems in the [Puente] formations are expected to be related more to softening and weakening, especially in the sticky shale zones, rather than to water volume."

The wells pumped about 20 gpm each from about 25 feet of overlying Old Alluvium and 20 feet of Puente Formation. The water was pumped to the surface, and the contractor believes this kept tunnel inflow to a minimum, i.e., "dripping" condition rather than 10 to 100 gpm local inflows.

The following ground water information on transmissibility, permeability and artesian conditions in Old Alluvium and Puente Formation at the Sacatella tunnel is not a substitute for dewatering pump tests for the Metro Rail Project. However, the data do provide some relative measure of inflow rates that could be locally applicable to the Metro Rail alignment. The following is excerpted from the LACFCD Geologic Report, pages 7 and 8:

> Ground water was found in all [LACFCD] borings. However, due to drilling fluid in the boring, it was not possible to accurately determine the depth at which ground water was first encountered or if there were artesian or perched water table conditions. The initial soils investigations were conducted by the City of Los Angeles between 1967 and 1972, using augers which did not require drilling fluid. Logs of these borings indicated, at least in several locations, that water is perched in the unconsolidated sediments [Old Alluvium] overlying the bedrock and is also found within the bedrock [Puente formation, Unit Cw], often under minor artesian head. Artesian head in the vicinity of Boring No. 3 was noted previously by the City as being particularly high with water rising from a depth of 33 feet to 13 feet overnight. Other borings in the vicinity had artesian heads of only 1 or 2 feet (City of Los Angeles Soils Investigation report, Test Boring Nos. 48, 48A and 48B). Static water levels in all borings were well above the top of the proposed tunnel, indicating that the excavation will probably be conducted under saturated conditions. The measurements for individual borings are listed in Table 6-1.

TABLE 6-1 Coefficients of Transmissibility and Permeability

| LACFCD Boring No. | Approximate Location* | Depth (ft) | Distance to Water (ft) | Saturated Material Overlying Tunnel (ft) | Est. Yield (gpm) | Est. T** | Est. P† | Max‡ Gas Reading | Ground Water Sulfate Content (ppm) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 292+70 | 47 | 10.5 | 9.5 | 0.15 | 2.43 | 0.07 | 17 | 66 |
| 2 | 287+58 | 52.25 | 7.2 | 19.0 | 0.49 | 4.96 | 0.02 | 12 | 778 |
| 3 | 282+46 | 53 | 10.0 | 15.0 | 1.23 | 7.66 | 0.18 | 0 | 928 |
| 4 | 277+00 | 47.17 | 9.2 | 10.0 | 0.09 | 0.63 | 0.02 | 7 | 1,350 |
| 5 | 272+60 | 50.75 | 10.6 | 12.2 | 0.10 | 1.13 | 0.03 | 20 | 154 |
| 6 | 229+94 | 54.08 | 17.6 | 5.5 | 0.03 | 0.41 | 0.01 | 0 | 252 |
| 7 | 227+27 | 59 | 23.7 | 4.3 | 0.17 | 2.28 | 0.06 | 0 | 182 |

*Refer to LACFCD DWG. No. 364-1102-D7.6 and D8.4-8-7.

**T = Coefficient of Transmissibility in gallons per day per foot.

†P = Coefficient of Permeability in gallons per day per square foot

‡In percent of lower explosive limit (LEL).

Core samples [LACFCD] of the bedrock appeared to have extremely low permeabilities, e.g., ranges from 0.01 to 0.18, hence it is presumed that ground water movement occurs through bedding planes, fracture fissures, rather than through pores in the rock. Estimates of bedrock transmissibility and permeability were made using the recovery time of the water surface in the borings after air jetting. The results are listed in Table 6-1. The Coefficient of Transmissibility "T" ranges from 0.41 to 7.66 and is defined as the rate of flow in gallons per day through a vertical section of the water-bearing material, in which the width is 1 foot and the height is the measured thickness. The Coefficient of Permeability "P" is the flow in gallons-per-day through a cross-sectional area of 1 square foot of saturated material. The average coefficient of permeability was calculated from the coefficient of transmissibility by dividing this value by the footage thickness of the saturated material.

### 6.5.9 Stand-up Time, Slabbing, Overbreak

Stand-up time was more than 2 to 3 hours prior to placing liner. Slabbing of flat-lying or steeply dipping beds did not occur. No overbreak was recorded, but minor air slaking developed due to the high air ventilation. Mr. Glanville called this "ideal" tunneling formation, except for the hard cemented layers.

#### 6.5.10 Ground Settlement Above Tunnel

The tunnel was excavated about 40 feet below the street surface in a residential area with one hotel. No settlement was noted, or reported, by the residents. No known complaints of noise, except at portals, were registered by the residents living above the tunnel during construction.

#### 6.5.11 Local Caving Problem

An abandoned 2-foot-diameter auger hole was penetrated. The hole caved upward to within 6 feet of the ground surface. The contractor drilled a hole from the surface into the cavern and filled the cavern with pea gravel prior to advancing the tunnel. The cave did not "daylight" to the surface.

#### 6.5.12 Portal Excavation Problems

Both portal excavations encountered local, as aforementioned, very hard sandstone interbeds which could not be excavated by small equipment. Therefore, heavy equipment (D-9 Caterpillar) was required. These are part of the "changed conditions" (as yet unsettled), according to Mr. Glanville.

#### 6.5.13 Ground Loading and Estimated Support Requirements

The following ground loading and estimated support requirements were reported (Wright, 1975, p. 5 and 6):

> Continuous light tunnel support will be necessary whether the tunnels are driven by boring machine or by drilling and blasting. The need for immediate support may often be marginal if the tunnel is machine-bored. However, the shales will need support eventually because of stress relief fracturing and slaking. Slaking was evident in a small percentage of the cores. The generally short core lengths are probably due to stress relief. Ground loading assumptions in the specifications seem unreasonably high at 3370 psf. Maximum estimated loads for this study are 2400 psf, where the ground is wet and highly unstable. Most loads should be on the order of only 800 to 1600 psf. Lateral loading up to possibly 800 psf may buildup in the wet unstable reaches.
>
> Six inch, 15.5# steel horseshoe sets spaced 3 to 5 feet apart will hold the estimated loads. A few invert struts may be necessary where the formation is extensively softened by ground water, especially through the low bedrock cover reaches.

## 6.6 MWD TONNER TUNNEL

### 6.6.1 Facts and Figures

Information for Tonner Tunnel No. 1 and Tonner Tunnel No. 2 is from MWD Historical Record (1976).

| | |
|---|---|
| Tunnel Length | 3.4 miles* |
| Tunnel Diameter | 11 ft O.D.; 8 ft I.D. steel pipe |
| Initial Support | Steel ribs & wood lagging |
| Excavation Method | Calweld rotary head TBM |
| Geology | Sandstone & Shale (Puente Formation) |
| Eventual Use | MWD water supply tunnel |
| Contractor | J.F. Shea Co., Inc. |
| Bid Price | $15,034,331; extra work, $314,000 |
| Estimated Completion Bid Time | 830 working days; actual - 1,522 days |
| Tunneling Period | 1972-76 |

*Tonner No. 1 - 4,600 ft, Tonner No. 2 - 18,400 ft, separated by several miles of pipeline

### 6.6.2 Application to Metro Rail Alignment

Geologic conditions and tunneling methods in Tonner Tunnel are very important to the Metro Rail alignment for the following reasons:

- Both tunnels, within 300 feet of the working face, were classified as "gassy" within the meaning of Appendix B of U.S. Bureau of Mines Bulletin 644, and were the first tunnels driven under California's strict new safety regulations since the fatal gas explosion in the San Fernando Tunnel, June 1971.
- Both tunnels penetrated the Puente Formation, a soft-ground geologic tunneling condition that may be similar to that anticipated in Metro Rail alignment Reaches 1 through 5 and designated Unit C on Drawing 2.
- Maximum cover on Tonner Tunnel No. 1 was 400 feet.
- Maximum cover on Tonner Tunnel No. 2 was 600 feet.

### 6.6.3 Unconfined Compressive Strength

Unconfined compressive strength of the bulk of the Puente Formation was 100 to 200 psi. However, one reach, about 1,500 feet long in Tonner Tunnel No. 1, encountered mixed face conditions - hard and weak zones - with the hard calcite cemented sandstone and conglomerate exhibiting unconfined compressive strengths of 12,000 to 15,000 psi. This hardness resulted in redesigning and rebuilding the Calweld TBM, and conventional drill and blast for the last 1,500 feet of tunnel. To permit conventional tunnel driving, the tunnel had to be reclassified as "potentially gassy" by the California Division of Industrial Safety (DIS). The DIS approved reclassification, with stringent conditions.

In the high unconfined compressive strength reach of the tunnel, the contractor drilled horizontal exploratory diamond bit core holes 70 to 75 feet in advance of the tunnel face, anticipating soft ground to accommodate the Calweld TBM.

### 6.6.4 Geology and Rate of Advance

The rate of advance per day for various tunnel reaches and the capsulized geologic conditions are tabulated below:

| Stationing | Distance (ft) | Rate of Advance/Day (ft) | Capsulized Geology and TBM Comments |
|---|---|---|---|
| Tonner Tunnel No. 1 | | | |
| 550+11 - 537+45 | 1,266 | 19 | >1" thick shale beds; slow progress due to high clay content & minor ground water inflows, formed muck, very difficult to handle by TBM |
| 537+45 - 532+10 | 535 | 33 | Shale with 25% to 50% sandstone, several minor faults, did not affect progress |
| 532+10 - 528+30 | 380 | 19 | Hard sandstone & conglomerate containing boulders to 2' diameter; locally cemented with calcite, required modification to TBM |
| 528+30 - 521+90 | 640 | 40 | Moderately dipping, highly fractured, soft shale, sandstone, claystone abruptly terminating at a minor fault at 521+90 |
| 521+90 - 518+20 | 370 | 9 | Hard, slightly fractured, sandstone & conglomerate, massive, poorly bedded clasts of boulder size; many TBM breakdowns |
| 518+20 - 504+22 | 1,398 | 26 | Abandoned TBM; conventional drill & blast fractured shale, hard sandstone & conglomerate. |
| Tonner Tunnel No. 2 | | | All progress with rebuilt TBM. |
| 891+60 - 875+00 | 1,660 | 40 | Thin (2") bedded sandstone & claystone; many minor fault zones, did not affect progress |
| 875+00 - 864+30 | 1,070 | 51 | Bedded to massive sandstone with siltstone & claystone interbeds, beds dip 15° to 20° south; many minor, seeps, quickly dried up |
| 864+30 - 858+50 | 580 | 58 | Zone of several faults & many seeps; highly fractured sandstone & shale |
| 858+50 - 814+80 | 4,370 | 54 | Shale with claystone, sandstone interbeds; locally hard; tar & water seeps common as were minor faults; large gas inflow at Sta 858+28 (DIS shut job down until ventilation modified & 20' long probe holes drilled ahead of face to test for gas). |
| 814+80 - 764+00 | 5,080 | 67 | Sandstone, siltstone, claystone, dip 5° to 10° north; water, tar, oil seeps common; shield stuck in minor fault zone, required additional support for the higher push pressures used to free the TBM* |
| 764+00 - 708+00 | 5,600 | 60 | Massive, coarse-grained sandstone, moderately to highly fractured, slightly petroliferous & damp; encountered 50' long seafloor landslide of sheared interbedded siltstone, claystone & sandstone** |

*One 75' long area required two months to over-excavate around the shield. (Solution: extend gage cutters on the wheel and excavate to a diameter slightly larger than the shield.)

**Although this was a small feature, TBM took two weeks to go through; became embedded and had to be freed by hand.

### 6.6.5 Oil and Gas

Neither oil nor gas was a problem in either of the tunnels. Petroliferous sands were a common occurrence in the first 1,300 feet excavated in Tunnel No. 1, but significant accumulations of gas were not encountered. In Tunnel No. 2, the contractor encountered slightly petroliferous rocks on a regular basis, along with numerous minor tar seeps. Only one area of tar seeps was large enough to cause concern. This was a 105-foot stretch near Station 824+00. A high concentration of gas which was encountered at Station 858+28 caused the sensors to automatically shut down the tunneling machine. This was the only significant gas accumulation encountered.

In some areas of Tunnel No. 2, particularly in the section where oil seepage had occurred during excavation operations, high gas concentrations were detected in the stagnant air space behind the steel pipe and the excavated tunnel wall (during pipe installation).

### 6.6.6 Disposal of Water and Oil

The California Regional Water Quality Control Board required less than 2,000 microhms electrical resistivity in all discharge water and a maximum of 20 ppm content of grease and oil in the discharge water. Daily sampling was performed by the contractor to ensure this requirement was being met. Water pumped from Tunnel No. 1 was discharged into a natural water course located at the southerly end of the work area and flowed into Arnold Reservoir approximately one-half mile away. The contractor had provisions for construction of oil-separating basins; however, removal of oil was originally successfully handled by holding the discharge water in a 12,000-gallon tank and then discharging water from the bottom of the tank. Oil was then removed from the top of the water as required and disposed of in accordance with specification requirements.

Essentially, the same setup was utilized at the south portal of Tunnel No. 2, a series of two tanks removing suspended solids and oil and grease. As the majority of this ground water had an electrical conductivity greater than 2,000 microhms, provisions had to be made to discharge water into the Yorba Linda sewer system. The District installed a 16-inch-diameter pipeline to a retention basin, located immediately below MWD's Robert B. Diemer Filtration Plant, which the contractor was allowed to use for temporary storage of discharge water. The retention basin had been lined previously with an impervious membrane to prevent seepage. A conductive recorder, with sensors that automatically operated valves, was installed in the 16-inch-diameter pipeline. When conductivity was less than 2,000 microhms, water was automatically discharged into Telegraph Canyon. Water directed into the retention basin was then pumped, through a pipeline, into the Yorba Linda sewer system.

## 6.7 TUNNELS IN THE LOS ANGELES AREA

Information pertaining to local tunnels where excavation was performed by tunnel boring machines (TBM) or where gas and oil were encountered appears in Table 6-2.

TABLE 6-2 Tunnels in the Los Angeles Area Excavated by Tunnel Boring Machines* or Where Gas and/or Oil Encountered

| Owner<br>Tunnel Name | Location<br>Length (mi) | Bore Diameter (ft) | Material<br>Geologic Unit | Year Begun | Method Used & Comments |
|---|---|---|---|---|---|
| L.A. Co. Flood Control Dist.<br>Storm Drain #1102<br>Hoover Street<br>Sacatella Tunnel | Downtown<br>Los Angeles<br>0.6 | 17 | Sandstone, Shale<br>Puente Formation | 1975 | Gradall excavator in a shield. Prior dewatering required by specs. GAS & SEEPING OIL encountered in L.A. City oil field but controlled by strong ventilation. |
| Metropolitan Water Dist.<br>Tonner 1 & 2 | Near Yorba Linda<br>Orange County<br>3.4 | 11 | Sandstone, Shale<br>Puente Formation | 1972 | Mainly rotary-head Calweld mole. Delay in Tonner 1 due to hard sandstone but rate in longer Tonner 2 averaged 60 ft/day; several days of over 100 ft/day. METHANE monitored. |
| Metropolitan Water Dist.<br>San Fernando | Sylmar area<br>5.5 | 22 | Sandstone, Siltstone,<br>Boulders<br>Saugus Formation<br>Alluvium | 1969 | Digger-type Robbins mole. Dry old alluvium stood well; wet old alluvium caved, requiring dewatering from within tunnel. Progress up to 277 ft/day (world record), including precast segment supports. Mole handled boulders in old alluvium & Saugus. METHANE & HEPTANE encountered. |
| Pacific Telephone Co.<br>Olive Street | Downtown<br>Los Angeles<br>0.1 | 7 | Siltstone<br>Puente Formation | 1969 | Rotary-head Calweld mole. Siltstone damp; no problems encountered. |
| Metropolitan Water Dist.<br>Balboa Outlet | Sylmar<br>0.7 | 16 | Sandstone, Siltstone<br>Saugus &<br>Sunshine Ranch Formation | 1968 | Rotary-head Scott mole. Conditions mostly dry to dripping; rates up to 111 ft/day, but averaged 30 ft/day due in part to short tunnel length & adjustments to new mole. |
| Metropolitan Water Dist.<br>Castaic 1 & 2 | Castaic-Saugus<br>3.5 | 26 | Siltstone, Sandstone,<br>Boulders<br>Castaic &<br>Saugus Formation | 1967 | Digger-type MEMCO mole handled large boulders in Saugus Formation with no significant problems. Average rate in Castaic 2 was 112 ft/day; best rate 202 ft/day (4100 cy excavated muck in 24 hrs). Precast concrete segment supports used. |
| Metropolitan Water Dist.<br>Newhall | Newhall-Sylmar<br>3.3 | 26 | Sandstone, Siltstone,<br>Mudstone<br>Saugus, Pico &<br>Towsley Formation | 1966 | Rotary-head Calweld mole from south portal, oscillating "windshield-wiper" mole from north portal. Long segment in wet sandstone with boulders of Saugus was dewatered with surface wells; GAS & SEEPING OIL handled with strong ventilation. |
| Los Angeles City<br>Aqueduct | Newhall-Sylmar<br>2.0 | 12 | Sandstone, Siltstone,<br>Mudstone<br>Puente, Pico &<br>Towsley Formation | 1910 | Spade, drill & blast. OIL encountered. |
| Southern Pacific Railroad<br>No. 25 | Newhall-Sylmar<br>1.3 | 22 | Sandstone, Silstone,<br>Mudstone<br>Saugus, Pico &<br>Towsley Formation | 1875 | Spade. OIL encountered. |

*Chase, A.P., and others, 1978, p. IV-5 and IV-6.

# Section 7.0
# Anticipated Ground Behavior in Underground Construction

# Section 7.0: Anticipated Ground Behavior in Underground Construction

This section deals with excavation, ground control and alternative methods of excavation and support for underground construction. The discussions must, at this stage, be general, as design has not been started and design concepts have not been reduced to concise working plans. The ultimate decisions regarding location, size, shape, alignment, grade, elevation of the line and station structures will affect the economics of the different methods of underground excavations and support systems. The following information is intended to provide geotechnical data with which designers can evaluate the merits of shallow (30- to 100-feet-deep) vs. deep (100- to 200+-feet-deep) tunnels as well as develop concepts regarding location and configuration of the tunnel and stations. For construction purposes, a tunnel should have at least one diameter of cover and therefore if the tunnel crown is less than 20 to 30 feet beneath the surface, cut-and-cover construction should be considered.

Based on this current 1980-81 geotechnical investigation, subsurface conditions along the Metro Rail alignment are believed favorable for mechanical excavation tunneling and at relatively high rates of advance, utilizing economical precast concrete segments to form both initial support and permanent lining in one construction operation.

## 7.1 ALTERNATIVE TUNNELING METHODS

Excavation and support systems will depend on the tunnel size and the type of ground materials anticipated in the different reaches. Depending on the grades established for the tunnels, it is likely that there will be significant intervals with comparable ground conditions. Reaches with anticipated similar ground conditions can be broken into contract reaches for construction purposes. For ease of interpretation, geologic formations have been divided or grouped into units which are expected to have comparable excavation characteristics. These units, $A_1$, $A_2$, $A_3$, $A_4$, SP and C are described in Sections 4.0 and 5.0, and on Drawing 2 and will be used in the following paragraphs of this Section.

### 7.1.1 Soft-Ground Tunneling

For purposes of this study, a soft-ground tunneling method is defined as one wherein ground conditions are such that mechanical excavation methods can be used within a shield to advance the tunnel, and wherein a support system is required immediately behind the shield. Such ground conditions along the proposed alignment include all alluvium units, San Pedro Formation and weak to moderately strong rocks of low hardness of the Fernando and Puente formations.

Methods of driving a soft-ground tunnel for the Metro Rail alignment will require a shield for certain reaches. Conditions will be uniform for long distances of tunneling. However, because of the variability of local ground conditions, which will result in areas of mixed face, with running ground at certain locations, shields should be equipped with breasting capabilities. Mechanical excavation methods have been proven to greatly increase production in similar local ground conditions (see Section 6.0). Possible mechanical methods include spade and claw backhoe-type diggers, roadheaders, and wheel-type (dragpick) machines. Where tunneling is to take place below the water table (perched or permanent), ground water control may be necessary. This subject is discussed in detail in Section 7.2.

The following discussion of possible tunneling methods by geologic unit is general and intended only to provide concepts. Subsequent paragraphs describe individual reaches and provide specific information on anticipated ground conditions.

#### 7.1.1.1 Young Alluvium

The Young Alluvium has two classifications: granular, dense sand and gravel ($A_1$) and fine-grained, stiff silt and clay ($A_2$). Ground water is likely in Unit $A_1$, and ground water control will be particularly important in the localities where tunneling may be required in this unit. Because of the uncemented and loose condition of sands and gravels, a shield with full-face breasting capabilities should be used. Mechanical excavation within the shield could be used. The use of compressed air or a combination of dewatering and compressed air may be a viable method of controlling ground water and flowing ground in the relatively cohesionless, permeable $A_1$ material. In the relatively cohesive, impermeable, fine-grained $A_2$ material, flowing ground and ground water inflow are not expected to be major problems. In borderline materials, a low air pressure may significantly reduce raveling behavior resulting from water seepage pressures.

#### 7.1.1.2 Old Alluvium

The Old Alluvium also has two classifications: granular ($A_3$) and fine-grained ($A_4$). The density of these materials is significantly greater than that of the Young Alluvium. Shield tunneling methods with mechanical excavation are also most appropriate for Units $A_3$ and $A_4$. Because of their greater density and strength, face stability is expected to be less of a problem in the $A_3$ and $A_4$ materials than in the $A_1$ and $A_2$ materials. However, there may be occasions where ground water control and face support will be necessary for satisfactory ground control, particularly in the $A_3$ Unit.

#### 7.1.1.3 San Pedro Formation

This unit, designated SP on Drawing 2, is chiefly dense sand and, where it will be encountered in Reach 5, it is likely to be impregnated with oil and tar.

Shield tunneling methods are expected to be required. Fundamental to successful and safe tunnel construction through the San Pedro sand, and any of the other potentially gassy formations, will be adequate ventilation.

### 7.1.1.4 Puente and Fernando Formations

These formations, grouped by the symbol C on Drawing 2, include claystone, siltstone and sandstone. Although long intervals of mixed face are not anticipated, available data indicates the presence of localized hard, calcareous sandstone beds and/or concretions. Mixed face conditions will be encountered at the contact between Unit C and the overlying Alluvium/San Pedro Formation and may also be encountered in some of the mapped and unmapped fault zones in these formations.

Mechanical excavation within a shield is expected to be appropriate for these materials.

At depths of up to 150 feet, there would not be any significant squeeze behavior in the Puente and Fernando formations (Unit C), e.g., using the standard index for the so-called overload factor, "OLF", in soft-ground tunneling (Clough and Smith, 1977):

$$\text{at 150 ft, OLF } \frac{\gamma H}{C} = \frac{150 \text{ ft} \times 130 \text{ psf}}{5040 \text{ psf}} = 3.9 = \text{no rapid squeezing}$$

$$\text{Where } C = \frac{70 \text{ psi}}{2} = 35 \text{ psi} \times \frac{144 \text{ in}^2}{\text{ft}^2} = 5040 \text{ psf.}$$

However, at depths in the range of 200 feet, there will be some tendency for squeeze behavior, particularly in the weaker portions of the Puente and Fernando formations, for example:

$$\text{at 200 ft, OLF} = \frac{200 \text{ ft} \times 130 \text{ psf}}{5040 \text{ psf}} = 5.2 = \begin{array}{l}\text{squeeze behavior}\\ \text{begins to be apparent.}\end{array}$$

Unit C materials are hard enough that squeeze behavior should not be a particular stability problem in normal shielded TBM operations, nor should the ground pressures exceed the capacity of normal support systems.

### 7.1.2 Rock Tunneling

A rock tunneling method is defined as one wherein drill and blast or mechanical methods can be used for excavation and wherein support may or may not be required. Rocks of this category include most of the Topanga Formation consisting of hard, well-cemented sandstone and conglomerate with some interbeds of weak siltstone (Geologic Map Symbol, Tt, on Drawing 1) and the crystalline basalt in the central part of the Santa Monica Mountains (Geologic Map Symbol, Tb, on Drawing 1).

Several significant and well-documented faults in the Santa Monica Mountains will be encountered. The hard-weak nature of beds and fault zones in the Topanga Formation and altered zones of the basalt indicate intervals of difficult ground can be anticipated.

Methods of excavation for these rocks include drill and blast, or hard rock tunnel boring machines (TBM). A TBM must be capable of coping with the expected intervals of difficult tunneling conditions. Drill and blast tunneling conditions through the fault zones are understood to the extent that the LAC Sewer Tunnel penetrated them successfully. It is possible that spiling, crown bars and partial breasting, along with slow and careful excavation may be necessary to penetrate these faults.

## 7.2 GROUND WATER CONTROL

Ground water will be present during tunnel construction in certain reaches of the project. For tunnels passing through alluvium, of units $A_1$ and $A_3$ below the water table, adequate ground water control will be necessary to ensure face control. Dewatering of the area from the surface by pumping wells is generally expected to be the most practical method of ground water control. Providing large areas are not dewatered to great depth, it is unlikely that serious ground settlement will occur, particularly in the granular materials (see Section 6.5.10). In specific areas where dewatering may not be appropriate because of local geologic or other constraints, compressed air, chemical grouting or freezing might be used in conjunction with, or in lieu of, dewatering. Grouting is generally limited to sands with less than 20% silts, while freezing can be used in fine-grained soils. Both techniques serve to restrict water flow and provide a stand-up time to a soil which otherwise might run or flow into the heading. Use of compressed air may be practical in either sand or silt materials. Compressed air may be of particular value in fine sand and silt which is too impermeable to dewater or grout, or to economically freeze. Use of low air pressures to fully or partially balance external water heads and reduce seepage pressures into the tunnel may be of value in stablizing the face in finer grained material, especially portions of units $A_1$ and $A_3$. Material which would otherwise be very difficult flowing ground may be thus transformed into slowly raveling ground which poses few problems to properly equipped soft-ground tunneling operations. Attempts to dewater the finer-grained cohesive materials may not be cost effective as these materials are of low permeability. Water seepage into the tunnel from fine-grained material would likely be of small amount unless non-cohesive sand lenses or zones are encountered. Flows from sand lenses could be of significant quantity, but are expected to be of limited duration, generally not more than one or two days (see Section 6.1.3).

In addition to the previous discussions concerning ground water control by the more conventional techniques, such as dewatering, it may prove desirable in some cases to utilize the newer techniques such as the earth pressure balance (EPB) or the slurry shields. The EPB is applicable to dry conditions or silty or clayey soils below the water table with relatively low heads. The slurry shield can be used in a variety of conditions, but is more likely to be applicable where ground water problems are significant. As of this writing (late 1981), only one EPB has been used in the United States. This was a sewer project in San Francisco, completed in silty soils with a water head of 20 feet above the tunnel (Clough, 1981).

## 7.3 ALTERNATIVE SUPPORT SYSTEMS

A support system will be required throughout most of the tunnel. Depending on how the system is designed, it may be advantageous to install an initial support which will serve also as the permanent lining.

### 7.3.1 Soft-Ground Tunnel Supports

A shield operation will require a strong initial support system that can serve as a buttress for jacking the shield forward. The most common and appropriate supports are ribs and lagging or precast concrete segments. If proper seals are used to minimize water and oil inflow, concrete segments could be both the initial support and permanent lining. Only small portions of the tunnel will have a significant potential external water head or water inflow. Concrete segments would not, however, keep gas from entering, since concrete is approximately 80 times more pervious to gas than it is to water. A lining system to seal out gas will have to be designed for gassy sections. Expansion of the supports or timely and thorough backpacking and grouting of the annular void between the support segments and tunnel wall is necessary to prevent surface settlement, particularly where the tunnel is driven at a shallow depth.

### 7.3.2 Rock Tunnel Supports

Rock bolts are an economical support in rock tunnels, with shotcrete or steel sets and lagging commonly used in more blocky ground and in fault zones. It is customary, although often unnecessary, to place a permanent concrete lining in subway tunnels in rock in the United States. Cast-in-place concrete should be used where repetitive form use renders concrete less expensive than shotcrete. Shotcrete may be appropriate for lining one- or two-of-a-kind structures (stations, transitions, connections of shafts to tunnels, etc.) where form work costs make cast-in-place concrete more expensive than shotcrete. Concrete lining in "hard" rock tunnels generally need not be reinforced due to minimum loads. Extensive, expensive measures to completely waterproof the lining are not recommended. Rather, drainage systems are generally most effective and economical.

## 7.4 STATION CONSTRUCTION BY MINING

Choice of deep tunnels will result in the need to excavate deep stations. For the mined station alternative, Table 7-1 gives minimum depth from surface to track level, plus suggested places where this alternative may be used. Open cut excavations in excess of 100 feet become increasingly difficult, and at about 150 feet become unprecedented. At these depths and below, mined stations should be considered. In general, it is judged feasible to mine a station configuration in soft-ground tunneling Units $A_4$ (Old Alluvium) and C (Fernando and Puente formations).

TABLE 7-1 Mined Station Alternative - Minimum Depth to Track Level

| Station | Minimum* Mined Depth Below Surface (ft) | Mined Station Geology | Comments |
|---|---|---|---|
| Union | 130 | Claystone | Recommend open cut of 30 ft |
| Civic Center | 50 | Claystone | Optional excavation method** |
| 5th St. | 160 | Claystone | Recommend open cut of 30 to 80 ft |
| Flower St. | 100 | Claystone | Optional excavation method** |
| Alvarado St. | 60 | Claystone | Optional excavation method** |
| Vermont Ave. | 70 | Claystone | Optional excavation method** |
| Normandie Ave. | 70 | Claystone | Optional excavation method** |
| Western Ave. | 130 | Claystone | Optional excavation method** |
| La Brea Ave. | 130 | Claystone | Optional excavation method** |
| Fairfax Avenue | 150 | Claystone, oil | Recommend open cut of 60 ft |
| Beverly Blvd. | 70 | Old Alluvium, oil | Recommend open cut of 70 ft |
| Santa Monica Blvd. | 100 | Old Alluvium | Optional excavation method due** to large planned storm drain |
| Hollywood | 120 | Old Alluvium | Recommend open cut of 30 to 120 ft |
| Hollywood Bowl | 100 | Basalt/Claystone | Optional excavation method** |
| Universal City | 90 | Claystone | Optional excavation method** |
| North Hollywood | --- | Old Alluvium | Open cut at Portal |

*Assumed 30 ft high station and 30 ft minimum cover in Claystone or Old Aluvium.

**Optional excavation method means that station excavation by mining from the tunnel or by surface open cut has no geological preference. The final decision should be based on other factors such as surface congestion, convenience of muck disposal in city streets, etc.

Mining of large open chambers in these soft-ground units, such as would be necessary for double track station structures, would require multiple drift methods and heavy support. For this reason, dual chamber stations, consisting of two enlarged tunnel sections with connecting galleries in between, should be considered. This configuration has been used in the design of the Forest Glen and Wheaton Stations on the WMATA Metro Route B (see Figure 7-1). Because of geologic conditions and community impact of cut-and-cover stations, these stations were designed for deep mining. Architectural criteria were changed to accommodate the dual chamber concept. For passenger access, the Forest Glen Station will use elevators which will travel a vertical distance of 175 feet, while the Wheaton Station will use an escalator with a vertical rise of 114 feet.

With the dual chamber configuration, the distance between track tunnels should be increased to allow room for separation of shafts or escalator raises from the track tunnels. If the shafts are too close to the station chambers, stability problems, requiring extensive support, may result.

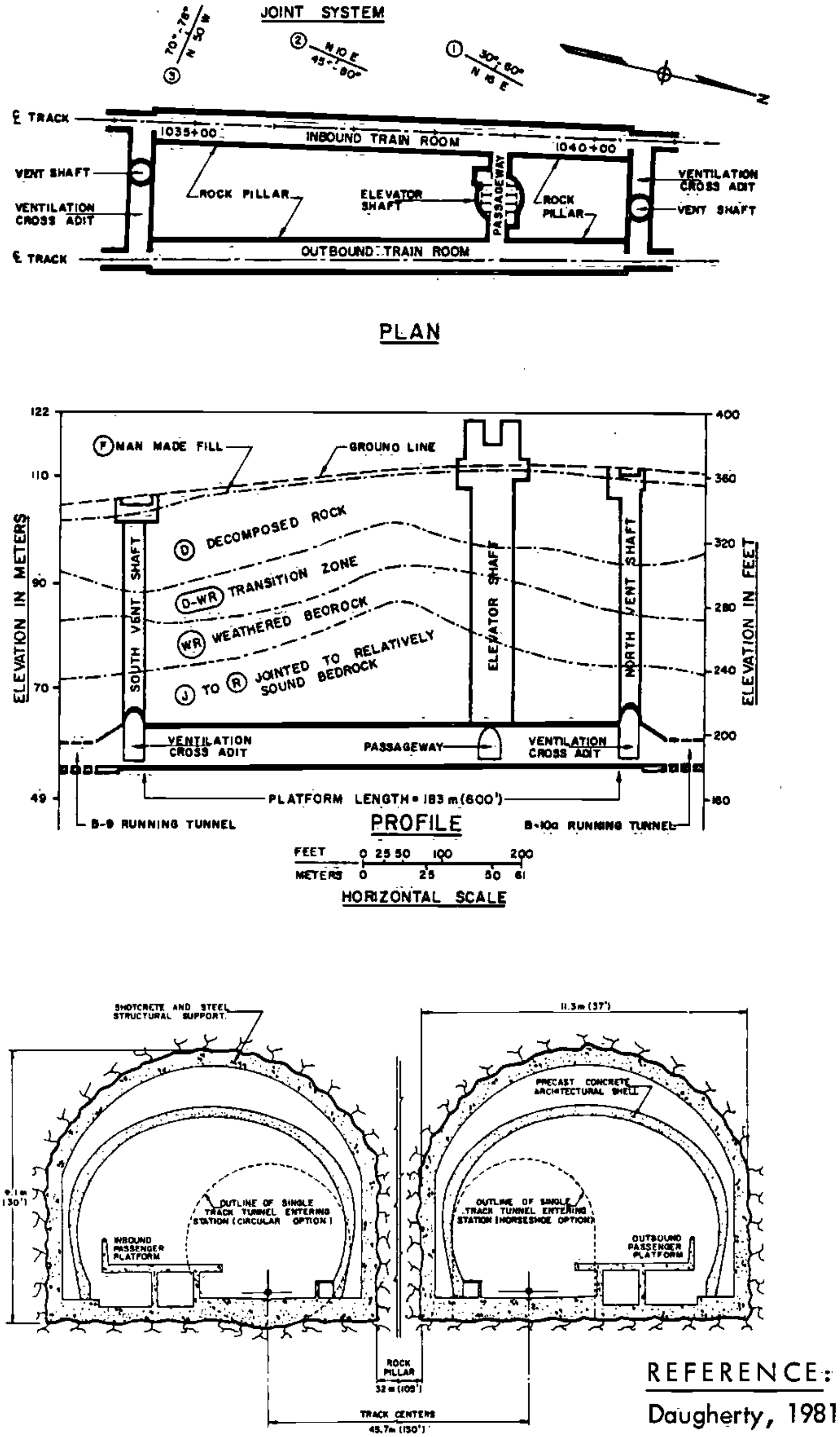
The smaller openings of the dual chamber configuration could be excavated with a heading and bench scheme (Proctor and White, 1977, p. 173), a much simpler operation than the multiple drifts required for a larger opening. Another advantage of the smaller openings is that they would not require as much depth of cover for stability as would a larger chamber. Thus, they could be situated at a shallower depth.

Chamber excavation can proceed by excavating access and/or vent shafts first and then mine out the connecting galleries and station chambers from the shafts. An alternative method would be to excavate the tunnels through the station area first and then enlarge the tunnels to form the platform chambers (Walton and Proctor, 1976). Special liner segments would be installed in the station reaches which could be removed, one segment at a time, as the enlargement progesses. Precedents for this kind of operation include the enlargement of old railroad tunnels to accommodate additional tracks and larger railroad equipment (Richardson and Mayo, 1975, p. 309).

In Japan, the Abeno Station was successfully mined between two lined tunnels in soft ground (Paulson, 1979). Figure 7-2 summarizes the construction sequence. In this case, construction of the tunnels and the connecting platform were accomplished under compressed air.

Mining of stations in the soft-ground units $A_4$ and C on the Metro Rail alignment can be expected to proceed slowly, possibly with use of a roadheader, using shotcrete, rock bolts and/or steel sets across the crown and directly behind the face. The bench would be removed after the arch is secured with support as required. Squeeze behavior in Unit C, as described in Section 7.1.1.4, will be a design factor if station depths approach 200 feet.

If underground station construction in the basalt and sedimentary rock of the Hollywood Bowl Station is considered, either a dual chamber or one large opening can be planned. Again, the heading and bench method is probably the most feasible. Rock excavation of the arch section by drill and blast methods would be followed immediately by support. It is anticipated that the rock arch can be supported with shotcrete and pattern rock bolting, as well as with steel ribs. Removal of the bench would take place after the arch is secured.




REFERENCE:
Daugherty, 1981

DESIGN OF FOREST GLEN STATION WMATA ROUTE B

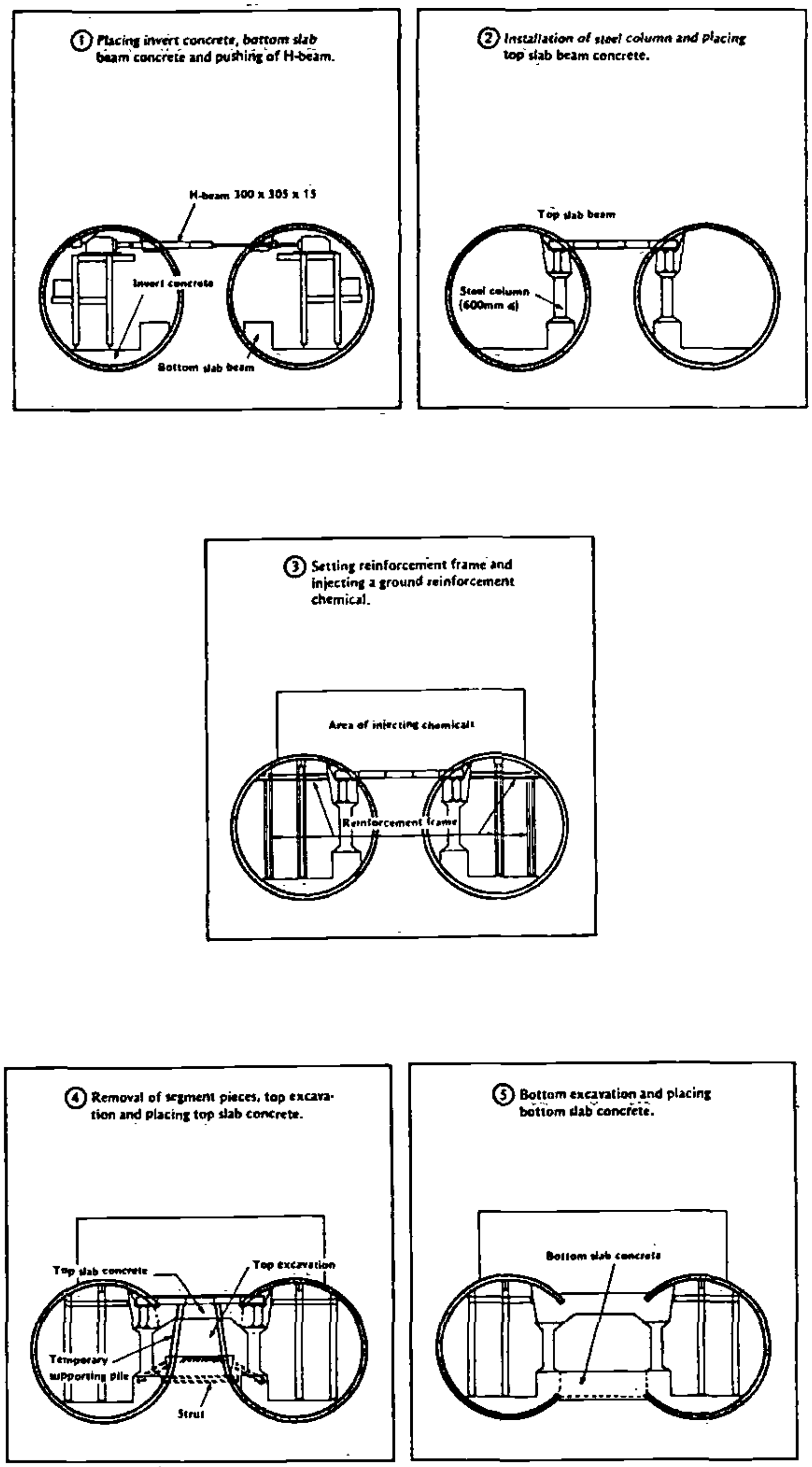
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 7-1

Process of construction method, roofing and excavating between two parallel shield tunnels for railway station.




REFERENCE:

Paulson, 1979

CONSTRUCTION SEQUENCE OF ABENO STATION PLATFORM

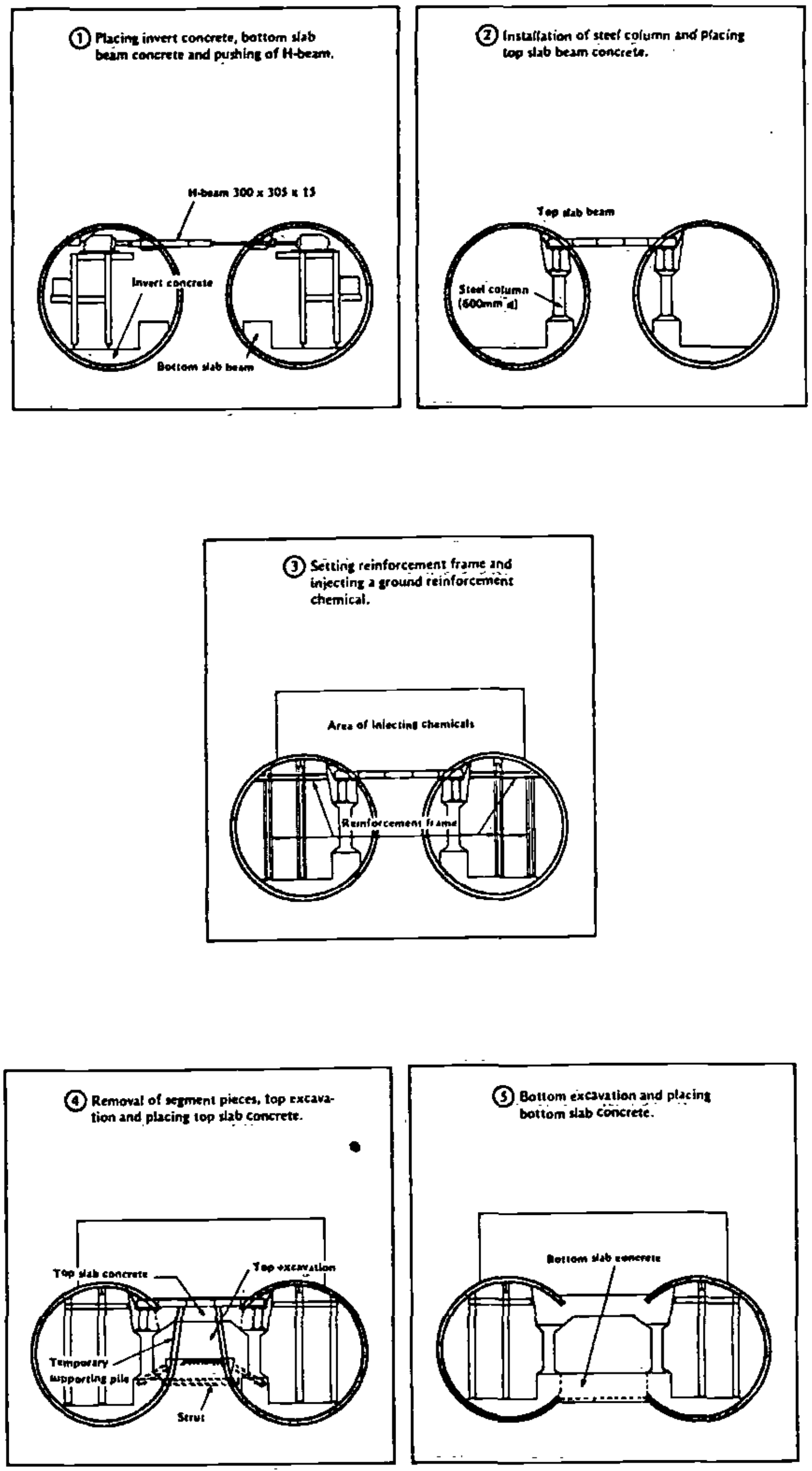
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 7-2

Process of construction method, roofing and excavating between two parallel shield tunnels for railway station.




REFERENCE:

Paulson, 1979

## CONSTRUCTION SEQUENCE OF ABENO STATION PLATFORM

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 7-2

## 7.5 WORKING SHAFT SITES FOR TUNNEL CONSTRUCTION

Possible working shaft sites for tunnel construction, general geology (surface to foundation) and suggested shaft depths are presented in Table 7-2. The present working site locations range from vacant areas to sites that may be possible only if enough working space is acquired by razing low buildings.

TABLE 7-2 Possible Working Shaft Sites for Tunnel Construction

| Working Site | General Geology Surface to Foundation | Suggested Shaft Depth |
|---|---|---|
| East Portal* | Claystone | (Portal) |
| Union Sta.* | Young Alluvium ($A_1$), wet below 25 ft | 30 ft |
| Civic Center Sta.* | Old Alluvium ($A_4$), claystone | 50 ft |
| Alvarado St. Sta.*† | Old Alluvium ($A_4$), claystone | 60 ft |
| Vermont Ave. Sta.† | Young Alluvium ($A_1$), claystone | 70 ft |
| Normandie Ave. Sta.† | Old Alluvium ($A_4$), claystone | 70 ft |
| La Brea Ave. Sta.*† | Old Alluvium ($A_4$), sand, claystone | 130 ft |
| Fairfax Ave. Sta.† | Old Alluvium ($A_4$), gas | 60 ft |
| Beverly Sta.* | Alluvium ($A_2$) ($A_4$), gas | 70 ft |
| Santa Monica Blvd. Sta.† | Alluvium ($A_2$) ($A_4$) | 100 ft |
| Hollywood Sta.† | Alluvium ($A_1$) ($A_2$) | 30 ft |
| Hollywood Bowl Sta.* | Basalt, claystone, wet | 100 ft |
| Universal City Sta.† | Young Alluvium ($A_2$), claystone, wet | 90 ft |
| North Hollywood Sta.* | Young Alluvium ($A_1$) | (Portal) |

*Vacant area at site (apparent available location).

†Possible site only if enough working space acquired by razing low buildings.

## 7.6 ESTIMATED TBM EXCAVATION RATES

The estimated tunnel boring machine (TBM) excavation rates listed in Table 7-3 are based on the present geotechnical study, previous tunneling experience in the area (Section 6.0) and judgment. The estimated advance for three 8-hour shifts is shown to range from 20 to 50 feet per day to 70 to 100 feet per day, depending on the type of formation being excavated. Oil and gas zones may reduce excavation rates by 10 percent to 50 percent, and there will be days of no advancement due to unforeseen circumstances. The approximate lengths of tunnel in each formation shown in Table 7-3 will vary depending on the final grade of the subway.

| Formation Type | Estimated Advance (ft/day) (three 8-hr shifts) | Approximate* Length of Tunnel in Formation (ft) | Percentage |
|---|---|---|---|
| Alluvium | | | |
| $A_1$ - Granular | — | 20,000** | 20 |
| $A_2$ - Fine-grained | 40 - 60 | 3,000 | 3 |
| $A_3$ - Granular | 70 - 100 | 20,000 | 20 |
| $A_4$ - Fine-grained | | | |
| Soft-ground Formations (C) Siltstone, Sandstone | 70 - 100 | 40,000† | 40 |
| Hard-rock Formations (T) Sandstone | 20 - 50 | 7,000 | 7 |
| Basalt | 20 - 50 | 7,000 | 7 |
| Fault Zones | 20 - 50 | 3,000 | 3 |
| | | 100,000 | 100% |
| Oil and Gas Zones | Reduce above by 10% - 50% | 16,000 | |

TABLE 7-3 Estimated TBM Excavation Rates

*These lengths will vary depending on final grade of subway.

**We recommend cut-and-cover excavation for certain Young Alluvium reaches, mainly to avoid tunneling in saturated, loose alluvium.

†Progress in Puente Formation may be slightly less than in Fernando Formation.

There are other variables in addition to ground conditions which can affect machine excavation rates, including skill of the operator and reliability of a particular machine. These estimated excavation rates are based on the assumptions of experienced crews and average machine downtime for maintenance.

## 7.7 UNDERGROUND CONSTRUCTION CONSIDERATIONS BY REACH

The geology along the proposed Metro Rail alignment, including a description of the stratigraphic units and ground water conditions, was given in Section 4.0. Physical properties of the different geologic units was given in Section 5.0.

Geologic units have been combined on the basis of similar properties to develop a profile along the alignment that shows anticipated tunneling ground conditions. The geologic profile (Drawing 2) is divided into eight reaches on the basis of predicted ground conditions.

### 7.7.1 Reach 1 - Downtown, Los Angeles River

This reach extends from the east Portal to the Hollywood Freeway and includes Borings CEG 1 through 6.

#### 7.7.1.1 General Description of Reach

The portion of this reach from the east Portal to the Los Angeles River is underlain by siltstone, claystone and sandstone of the Puente Formation (Unit C), with overburden up to 30 feet thick consisting of coarse-grained old alluvium (Unit $A_3$). From the Los Angeles River westward, the claystone surface dips steeply, and the overburden, which consists of river channel deposits, thickens to 100 feet at Boring CEG 4 (Drawing 5). Between Borings CEG 5 and 6, the micro-gravity surveys indicate that the overburden may reach a thickness of 130 feet. The river channel deposits consist predominantly of granular sands with some lenses of fine-grained sand and gravels. The gravels include cobbles up to 8 inches in diameter and possibly larger. Surface exposures (prior to lining the Los Angeles River) of the Young Alluvium indicate boulders up to 4 feet in diameter. Although it is assumed that this alluvial unit will contain similar boulders at depth, the actual quantity of boulders is unknown. Between Boring CEG 6 and the Hollywood Freeway, the alignment crosses the west side of the river channel where the overburden decreases in thickness and the claystone surface rises to within 10 feet of the ground surface.

#### 7.7.1.2 Shallow Tunnel and Cut-and-Cover

A bored tunnel 30 to 130 feet beneath the surface will pass through a considerable length of alluvium with a ground water table that was measured between 20 and 35 feet beneath the surface in the winter of 1981. A tunnel passing through these materials could expect flowing ground conditions (Table 7-4). Flowing ground caused by saturated alluvium can be controlled by surface dewatering with wells drilled into the alluvium below the tunnel invert. Total dewatering of the tunnel section will minimize but not entirely do away with stability problems at the tunnel face and could cause some ground settlements. Settlements are expected to be minimal in these granular materials unless the water table were lowered in excess of about 50 feet. These alluvial materials are estimated to have a permeability on the order of, or exceeding, $10^{-1}$ cm/sec. Assuming a saturated thickness of 100 feet, transmissibility would be on the order of 100,000 to 200,000 gallons per day per foot. If extensive dewatering is envisioned for construction, a full-scale pump test in this area is recommended. This will provide the detailed information needed by contractors for design and bidding a dewatering system.

Driving the tunnel using high compressed air may be difficult, if not impossible, due to the difficulty anticipated in maintaining the required air pressure in the coarse gravels. Compressed air tunneling at depths of 60 to 80 feet or more below the water table is likely to be uneconomical.

A combination of partial dewatering and the use of lower compressed air pressure in the tunnel is another method of driving a tunnel through these materials. EPB or slurry shields are also candidates for tunneling in this reach. Chemical grouting could also be useful in limited areas with critical ground support problems.

Cut-and-cover construction across the wet alluvium appears more feasible. Most of the excavation would be above the water table, with only minimal dewatering required. Cut-and-cover construction would require crossing the concrete channel of the Los Angeles River, as well as several existing railroad tracks.

The quality of ground water in Reach 1 is poor and is likely to be corrosive, requiring special type of cement. Borings CEG 1 and 3 contained sulfate exceeding 150 ppm, a level considered deleterious to cement. The total dissolved solids (TDS) ranged from 412 ppm (Boring CEG 2) to 20,230 ppm (Boring CEG 6). The high TDS in CEG 6 is due to sodium chloride water (NACL), apparently originating from the nearby Union Station oil field as a brine.

Traces of gas were encountered in Borings CEG 1 through 4 along this reach, but gas was not detected in Borings CEG 5 and 6. Based on these results, classifications assigned to this reach are: gassy from the beginning of the line to Boring CEG 4, and potentially gassy from Boring CEG 5 to the Hollywood Freeway. This gas probably originates in the Puente Formation beneath the alluvium.

TABLE 7-4

| Classification | | Behavior | Typical Soil Types* |
|---|---|---|---|
| Firm | | Heading can be advanced without initial support, and final lining can be constructed before ground starts to move. | Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed. |
| Raveling | Slow raveling | Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling. | Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress. |
| | Fast raveling | | |
| Squeezing | | Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress. | Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind surface. |
| Running | Cohesive-running | Granular materials without cohesion are unstable at a slope greater than their angle of repose (± 30°-35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose. | Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running. |
| | Running | | |
| Flowing | | A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases. | Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed. |
| Swelling | | Ground absorbs water, increases in volume, and expands slowly into the tunnel. | Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite. |

*Includes weak Rock Types

Ref: Tunnelman's Ground Classification (modified from Heuer, 1974, as modified from Terzaghi, 1950)

#### 7.7.1.3 Deep Tunnel

A tunnel constructed as much as 150 feet in depth would be contained in claystone of the Puente Formation. The tunnel could be 150 feet below the surface beneath the river channel and still maintain a grade of less than 6 percent. The advantages of better ground conditions would have to be weighed against the disadvantages of constructing a deep station at Union Station (Table 7-1). The claystone unit (Unit C) can be excavated by TBM as described in paragraph 7.1.1. Of particular note in the characteristics and excavatability of this formation are the hard calcareous sandstone beds or concretions that are occasionally encountered. These beds appear to range in thickness from 2 inches to 2 feet. However, because of changing dip of the formation, it is possible to encounter a section of tunnel wherein a hard calcareous bed may persist in the tunnel face for a considerable distance. The calcareous sandstone has a compressive strength which ranges from 5,000 to 15,000 psi (see Section 6.5.5). These beds have been of sufficient hardness to cause bending of the shield cutting edge in the LACFCD Sacatella Tunnel (Section 6.5). This potential mixed face condition in the Puente Formation should be considered when selecting excavation equipment for this reach of the tunnel.

Laboratory tests of Unit C in this reach (aside from the hard beds) give values as follows:

° Puente Formation

Unconfined compressive strength (Qu) - Range 10 to 175 psi; average 79 psi (20 tests)

Aside from the occasional hard cemented sandstone beds, the materials would excavate quite rapidly with most mechanical excavating equipment. With rocks of these general strengths, light to moderate squeezing of the ground on the shield and support system should be anticipated because of overburden pressure (depending on tunnel depth).

Information from the boring logs indicates that bedding in the claystone unit dips between 10° and 60°, with average dips of 40° to 60° from the horizontal. Although the direction of dip cannot be determined from the borings, projections from surface outcrops suggest that the bedding is inclined to the south or southwest. Based on this projection, the strike of bedding will nearly parallel the tunnel along this reach. This orientation of bedding means that, where occasional hard cemented beds are encountered, they will follow the tunnel alignment for some distance.

A deep tunnel would be below the ground water table throughout this reach. The formation is generally impervious and large ground water inflows would not be expected. Dripping conditions would likely be common, and some localized measurable flows can be expected.

Quality of water in the claystone unit is probably similar to ground water of the overlying alluvium. That water is of very poor quality (see Appendix G, Volume II), and could be corrosive to improperly designed concrete and metal lining systems.

A fault has been inferred by previous investigators to account for the step in the buried claystone surface near the Los Angeles River, although this step could also have resulted simply from erosion of the river channel. The Puente Formation has been folded and faulted, as evidenced in surface outcrops, so it is likely that many small faults will be encountered in the tunnel. Ground conditions at the fault zones are probably not significantly different from conditions in the unfaulted, thinly bedded, sedimentary rocks of low hardness that comprise the claystone unit. Some fault zones may, however, provide localized sources of ground water seepage, and provisions should be available to handle occasional small water flows and raveling ground at the face. In addition, fault zones may contain altered materials with physical properties different from that of the wall rock unit, such as swelling and/or slaking characteristics. Design of the permanent lining should provide for proper drainage and removal of water within the tunnel.

### 7.7.2 Reach 2 - Broadway to Harbor Freeway

Reach 2 extends from the Hollywood Freeway to the Harbor Freeway and includes Borings CEG 7, 8 and 9.

#### 7.7.2.1 General Description of Reach

Beginning at the Hollywood Freeway, this reach is claystone (Unit C) with only a few feet of alluvial overburden at ground surface. The overburden increases in thickness along the alignment towards Broadway, reaching a depth of about 40 feet near 4th Street. The remainder of this reach crosses an ancestral channel of the Los Angeles River where the overburden is at least 130 feet thick. The west side of the channel is crossed near 7th and Hope Streets, and the overburden decreases in thickness from about 55 feet near Boring CEG 9 to about 10 feet at the west side of the Harbor Freeway. The overburden materials in this reach are Young Alluvium deposits consisting primarily of dense sands (Unit $A_1$), but including lenses of stiff clay (Unit $A_2$) up to 40 feet thick and medium dense cobbly sand (Unit $A_1$). Through this reach, the claystone (Unit C) is the Fernando Formation. Properties of this formation are essentially the same as those of the Puente Formation, except that the hard calcareous beds and concretions appear less frequently in the Fernando Formation.

#### 7.7.2.2 Shallow Tunnel and Cut-and-Cover

Along the first part of this reach, Hollywood Freeway to 4th Street and Broadway, the most favorable tunneling conditions will be in the claystone (Unit C). Tunneling conditions will be similar to conditions described for Unit C in Reach 1, except that hard sandstone beds are less frequent. From a geotechnical standpoint, the design of the tunnel profile is quite flexible along this part of the reach. The profile should be deep enough to maintain at least one tunnel diameter of claystone above the tunnel crown.

Bedding in the claystone dips about 60° in Boring CEG 7; 5° to 10° in Boring CEG 8; and about 30° in Boring CEG 9. Projections from surface outcrops suggest that the beds are inclined to the south or southeast. The strike of beds will be at high angle, or nearly perpendicular to the tunnel alignment along the Broadway segment of this reach. Hard, cemented beds will be encountered in the crown of the tunnel, projecting towards the invert as the face advances toward the Harbor Freeway. Where the alignment turns onto 7th Street, the bedding will become nearly parallel to the tunnel, inclined from the upper right to lower left side of the face. Small faults can be anticipated in the claystone.

Laboratory tests give the following strength values for units in this reach:

° Young Alluvium

Undrained, Quick, Direct Shear - $\phi$ - Range = 28° to 38°; average 33°
(2 tests) C - Range = 0 to 1.0 ksf; average 0.5 ksf

° Puente Formation

Unconfined Compressive Strength (Qu) - Range = 36 to 107 psi; average 70 psi
(9 Tests)

For the remainder of the reach, 4th Street to Harbor Freeway, a shallow tunnel could be driven through the alluvial channel deposits, mainly above the water table. Ground water across this reach appears to be at the alluvial-claystone contact except across the center of the old alluvial channel, where it is on the order of 100 feet below the surface. Tunnel excavation conditions would be more favorable than those ground conditions anticipated in the wet alluvium in Reach 1. Available data indicate the alluvial material to be generally moist, dense sands; slow raveling ground conditions are anticipated. Fast ravelling to running ground conditions could be encountered where lenses of sand with little or no cohesion are penetrated. Settlement of the surface should not be a problem, providing the contractor uses proper excavation and support techniques and accomplishes the backfill operation behind the initial support in a timely fashion.

Cut-and-cover line construction would be a shallow tunnel. Any advantages would have to be balanced against the problems of dealing with utilities, surface obstacles, and traffic congestion. Geotechnically, there does not appear to be a significant advantage between cut-and-cover line construction and tunnel construction, except that the open excavation may allow more flexibility to handle large boulders if randomly encountered in the alluvial deposits.

The alluvial deposits are not expected to be gassy, although gas was encountered in Boring CEG 8. Gas was not detected in Borings CEG 7 and 9. This reach has been given a classification of potentially gassy.

7.7.2.3 <u>Deep Tunnel</u>

A deep profile would be driven below the alluvial channel in the claystone. This deep alignment would encounter favorable tunneling conditions, but would require the mining of two deep stations approximately 100 and 180 feet deep. At depth close to 200 feet, ground squeeze behavior may become apparent (see Section 7.1.1.4). Tunneling conditions through the claystone (Unit Cw) of the

Fernando Formation would be similar to those previously described. As previously noted, the Fernando Formation appears to have less frequent occurrences of the hard calcareous beds that can slow mechanical excavations. However, some occurrences of the hard beds can be expected. A deep alignment would be driven beneath the water table. The clay units of this reach, similar to those in Reach 1, are generally of low permeability, and large water flows would not be expected. Dripping conditions and occasional small flows can be expected from the more sandy beds and at fault zones in the formation.

### 7.7.3 Reach 3 - East Wilshire Boulevard

Reach 3 extends from the Harbor Freeway to Normandie Avenue, and includes Borings CEG 10 through 14.

#### 7.7.3.1 General Description of Reach

Geologic conditions along this reach between the Harbor Freeway and Normandie Avenue are relatively uniform. The overburden ranges in thickness from about 20 to 40 feet and primarily consists of stiff clays (Unit $A_4$) with a few lenses of dense sand (units $A_1$ and $A_3$). Interbedded claystone, siltstone and sandstone of the Puente Formation (Unit C) underlie the overburden.

#### 7.7.3.2 Shallow and Deep Tunnels

Tunneling along this reach within the claystone (Unit C) at depths between about 40 to 200 feet should be favorable. A deep profile through this reach would minimize the influence from foundation loads of the tall structures along Wilshire Boulevard. However, profiles approaching 200 feet deep will have a tendency for squeeze behavior in the weaker beds of the Puente Formation, due to overburden loads (see Section 7.1.1.4). Tunneling conditions in the claystone will be similar to those conditions described in subsection 7.7.1.3 for Reach 1 in the Puente Formation. The potential for hard lenses will likely affect design of the excavation equipment. Bedding dips about 20° to 55°, generally to the southwest, so that the strike is nearly parallel to the tunnel alignment. Stations along this reach can be either mined in the claystone or constructed by open-cut methods (Table 7-1). At least one tunnel diameter of claystone cover should be maintained over the crown of the tunnel.

Laboratory strengths of Unit C in this reach are as follows:

Unconfined Compressive Strengths (Qu) - Range 7 to 157 psi; average 66 psi (39 tests)

A perched ground water level occurs within the overburden. A permanent water level occurs within the claystone unit at depths of 140 to 180 feet. Near MacArthur Park, the perched ground water level is near the ground surface. The claystone unit has a low permeability and large inflows of water are not expected.

The eastern part of the reach, including Borings CEG 10 through 12, has been assigned a classification of gassy, and minor oil seeps should be expected. The remainder of the reach is classified as potentially gassy, with occasional minor oil seeps anticipated.

Geotechnically, tunneling is preferred over cut-and-cover line construction in Reach 3.

### 7.7.4 Reach 4 - Central Wilshire Boulevard

This reach extends from Normandie Avenue to La Brea Avenue and includes Borings CEG 15 through 18.

#### 7.7.4.1 General Description of Reach

From Normandie Avenue westward, the surface of the claystone (Unit C) slopes downward to a depth of about 90 feet at Boring CEG 15, remaining at this depth to La Brea Avenue. Throughout the reach, the claystone is overlain by the San Pedro sand (Unit SP), which has an average thickness of about 20 feet. The San Pedro sand is relatively clean, cohesionless, and is saturated. The San Pedro sand is overlain by 50 to 70 feet of Old Alluvium, consisting primarily of stiff clay and dense sands (units $A_4$ and $A_3$, Drawing 2).

#### 7.7.4.2 Shallow Tunnel

A shallow alignment would best be located in the Old Alluvium at relatively shallow depth, with the intent of avoiding the San Pedro sand. Tunneling conditions in the San Pedro sand are expected to be poor, with raveling to flowing ground conditions possible. Perched ground water level is at depths of 20 to 30 feet. Therefore, a tunnel driven in the Old Alluvium would be below or partially below the perched ground water level. The $A_4$ Unit is generally very stiff to hard cohesive clay alluvium. Laboratory tests gave the following strength values for geologic units in this reach:

° Old Alluvium

| | |
|---|---|
| Unconfined Compressive Strength (Qu) (3 Tests) | - Range = 37 to 40 psi; average 39 psi |
| Undrained, Quick, Direct Shear (3 Tests) | - ø - Range = 23.5° to 33°; average 29°<br>C - Range = 0.46 to 1.25 ksf; average 0.93 ksf |

° San Pedro Sand

| | |
|---|---|
| Undrained, Quick, Direct Shear (3 Tests) | - ø - Range = 32° to 33°; average 32°<br>C - Range = 0.07 to 0.29 ksf; average 0.18 ksf |

Blow count, N-values, through the $A_4$ unit range from 14 to 50+ with the greatest number in the 20 to 30 range. The $A_3$ material is generally dense to very dense, coarse-grained alluvium. Blow counts again are in the 10 to 50 range, with the greatest number in the 20 to 30 range. The cohesiveness and density of the $A_4$ unit indicate that this material would be conducive to any of the mining techniques discussed in section 7.4. Loose sands could be encountered in the $A_3$ unit. Therefore, running or flowing ground is a possibility depending on the ground water conditions. Some dewatering of the alluvium from the surface could be done, although the permeability of the Old Alluvium is expected to be erratic, and surface dewatering would be only partially successful.

This reach has been classified as potentially gassy.

7.7.4.3 Deep Tunnel

A deep tunnel (greater than 110 feet) could be confined to more favorable tunneling conditions in the claystone. The claystone unit includes materials of the Puente and Fernando formations. Bedding dips range from about 10° to 50° toward the southwest, with average dips of 20° to 40°. Tunneling conditions in the claystone would be similar to those previously described. Some mixed face conditions should be anticipated in claystone throughout this reach; because several hard sandstone beds were encountered in the drill holes, which varied in thickness from less than one-inch to three-feet.

Claystone units through this reach gave test results as follows:

Unconfined Compressive Strength (Qu) - Average 124 psi
(20 Tests)

The perched ground water level is at depths of 20 to 30 feet. A deeper permanent water level was recorded in the claystone at depths of 140 to 160 feet. As previously stated, the permeability of the claystone units is generally low, and although the materials might be saturated, large water flows are not expected.

7.7.5 Reach 5 - La Brea Area

This reach extends from La Brea Avenue to Melrose Avenue and includes Borings CEG 19 through 23A.

7.7.5.1 General Description of Reach

In the most eastern segment of this reach, between Borings CEG 18 and 19, the claystone unit is at depths of 60 to 85 feet. The claystone is overlain by San Pedro sand (Unit SP) ranging in thickness from 20 to 30 feet, and by Old Alluvium consisting of stiff clay (Unit $A_4$), which ranges in thickness from about 30 to 60 feet.

At Fairfax Avenue the alignment turns north and the surface of the claystone unit is at a depth of about 150 feet near Boring CEG 21. Near old Boring WC-7 the claystone is at a depth of 110 feet, then at depths of 200 feet or more at Borings CEG 22 and 23. Where the claystone surface steepens, both the overlying San Pedro sand and the Old Alluvium increase in thickness. San Pedro sand is 150 feet or more thick near Boring CEG 23. Old Alluvium is nearly 140 feet thick near Boring CEG 23A. From about Boring CEG 22 northward, the Old Alluvium (Unit $A_4$) is overlain by a thickening blanket of Younger Alluvium (Unit $A_2$) consisting mostly of clay and silt.

This reach is adjacent to the La Brea Tar Pits, the Salt Lake Oil Field and South Salt Lake Oil Field (Drawing 1). A number of faults have been mapped by the oil companies in the Puente Formation (Unit C) including the 3rd Street, 6th Street and San Vicente faults. These faults are oil traps in the formation, and in some places provide conduits for upward migration of oil and gas into overlying materials such as San Pedro sand and alluvium. This upward migration of oil along faults and fractures has resulted in the formation of the La Brea Tar Pits. The San Pedro sand is impregnated with oil and tar throughout most of this reach, and lenses of tar sand were encountered in the overlying alluvium, particularly in the vicinity of the La Brea Tar Pits. Gas was encountered in most of the borings along this reach. This reach has been classified as gassy, necessitating extra precautions for tunnel construction. Additionally, uncharted, abandoned oil wells might be encountered during tunneling in this reach. (Drawing 1 shows location of nearby known oil wells.)

7.7.5.2. Shallow Tunnel and Cut-and-Cover

A relatively shallow tunnel through the north-south part of this reach is estimated to encounter the most favorable tunneling conditions. Ideally the tunnel should be kept above the San Pedro sand, where possible, while maintaining about 40 feet of cover in the alluvium. Exploration data indicate the San Pedro sand to be uncemented and saturated with gas, oil and tar products. The Old Alluvium (Unit $A_4$) is stiff to hard and generally of low permeability. It is possible that portions of the more granular $A_3$ materials will be encountered. Exploration data indicate that a shallow profile would encounter less of the oil-saturated ground, except in the immediate vicinity of the La Brea Tar Pits. Gas can be expected in any portions of the tunnel as persistent seepage. High gas pressures are not believed to be present. Localized areas adjacent to structural folds and faults which may form traps may have gas under some pressure, however. Much of the ground may be oil-saturated, although flows of oil are expected to be small, yet persistent, particularly in the vicinity of some of the projected faults. Possible excavation equipment includes breasted shields with mechanical diggers or wheel-type excavators. Consideration should be given to the effect of oil-saturated ground and tar on equipment. Explosion-proof equipment, as well as abundant ventilation, are required for tunneling through gassy reaches.

Laboratory data for materials of this reach include the following:

- Old Alluvium

Undrained, Quick, Direct Shear - ø - Range = 4.5° to 39.5°; average 33°
(10 Tests) C - Range = 0.26 to 2.64 ks; average 1.20 ksf

- San Pedro Sand

Undrained, Quick, Direct Shear - ø - Range = 29° to 35°; average 32°
(7 Tests) C - Range = 0.25 to 0.56 ksf; average 0.39 ksf

Perched ground water is found from 10 to 20 feet deep along this reach but should drain away within a few days.

Although other reaches of the Metro Rail alignment are known to contain gas, gas seepage into the finished tunnel is expected to be worse in this reach. A secondary lining, with special provisions to prevent gas seepage may be necessary through this reach. The exact locations of gas seepage should be mapped during tunnel excavation to give required limits of the secondary lining.

Cut-and-cover line construction is an alternative through this reach, especially north of Wilshire Boulevard. Such construction would have the distinct advantage of venting gas to the atmosphere. Unanticipated problems with oil seeps might be more easily handled in an open excavation. The advantages of such construction must be weighed against the disadvantages of attempting surface excavation in a busy urban area, along with the problem of handling utility lines which cross the excavation.

7.7.5.3 Deep Tunnel

An alternative profile would be a deep tunnel (approximately 120 feet deep) driven in the claystone unit. This alternative is preferred between the proposed La Brea Avenue and Fairfax Avenue stations, under Wilshire Boulevard. North of Wilshire Boulevard, the deeper tunnel alternative is not preferred, as it would require a minimum tunnel depth of about 180 feet between the proposed Fairfax Avenue and Beverly Boulevard stations. Beyond the Beverly Boulevard Station the depth of the claystone is unknown, as Borings CEG 23 through 25 did not reach the claystone. Therefore, a deep tunnel here would pass out of the claystone and into San Pedro sand where it would be necessary to tunnel several thousand feet through that unit. Abundant oil seeps and gas would have to be anticipated.

The claystone in this reach is part of the Fernando Formation. Tests of this material indicate the following:

Unconfined Compressive Strength (Qu) - Range 3.5 to 99 psi; average 56 psi (18 Tests)

Numerous faults should be anticipated through this reach. Oil, gas and water as initial surges, continuing as seepage, would be anticipated problems associated with any of these faults.

A permanent water level exists within or at the upper contact of the claystone.

7.7.6 Reach 6 - Hollywood Area

This reach, extending from Melrose Avenue to Yucca Street, includes Borings CEG 24 through 28.

### 7.7.6.1 General Description of Reach

This reach through Hollywood crosses a deep alluvial basin where the claystone unit is at depths below 200 feet (Drawing 2). It contains Old Alluvium mantled by 50 to 90 feet of Young Alluvium. The Old Alluvium consists generally of stiff sandy clay and clayey sand (Unit $A_4$), with lenses of dense silty sand (Unit $A_3$). The Young Alluvium is composed of firm to stiff clays and silts (Unit $A_2$) with lenses of compact medium dense sand (Unit $A_1$).

### 7.7.6.2 Tunneling Considerations

The most favorable ground conditions for tunnel construction for the first three miles of the reach is in Old Alluvium at depths below about 70 feet. Above that depth a tunnel alignment would encounter substantial lengths of Young Alluvium (Unit $A_2$) with a high water table. Ground conditions in the Older Alluvium will generally be firm ground. This material is similar to the Old Alluvium in the San Fernando Tunnel where rapid TBM excavation progress was achieved (see Section 6.1). Limited occurrences of flowing ground should be anticipated if lenses of saturated coarse-grained (Unit $A_3$) Old Alluvium are encountered.

Laboratory tests of samples from this reach gave the following values:

° Young Alluvium (Unit $A_2$)

| | |
|---|---|
| Unconfined Compressive Strength (Qu) (1 Test) | - 72 psi |
| Undrained, Quick, Direct Shear (6 Tests) | - ø - Range = 20° to 39.5°; average 30°<br>C - Range = 0.18 to 1.12 ksf; average 0.70 ksf |

° Old Alluvium (Unit $A_4$)

| | |
|---|---|
| Unconfined Compressive Strength (Qu) (14 Tests) | - Range = 28 to 94 psi; average 50 psi |
| Undrained, Quick, Direct Shear (10 Tests) | - ø - Range = 5° to 40°; average 29°<br>C - Range = 0.64 to 2.58 ksf; average 1.35 ksf |

Ground water along most of this reach is probably perched within the more shallow sand lenses above the stiffer relatively impervious clays. Depth to water in the winter of 1981 ranged from about 90 to 100 feet in Borings CEG 24 and 25, to 25 to 30 feet in Borings CEG 26 and 27, although the cores did not appear to be saturated below these depths during drilling. Occasional water seeps can be expected in the $A_4$ unit of the Old Alluvium, although large flows are not likely. Penetration of lenses of the more granular $A_3$ material may result in inflow of stored water into the tunnel. Such flows, which could be measured in hundreds of gallons for a short duration, would normally be expected to drain themselves within a period of several days.

Gas was not detected in any of the borings along this reach. Therefore this reach is classified as non-gassy.

Cut-and-cover line construction should be considered along the northern end of this reach, along Cahuenga Boulevard. Here the abundance of sand lenses increases, becoming coarser and more gravelly, and a higher perched water table exists, 30-60 feet deep. Cut-and-cover line construction would avoid tunneling through the sandy and gravelly ground along this part and would also allow for a shallow excavation above the water table for the proposed Hollywood Station. Cut-and-cover line segment may simplify construction where crossing the active Hollywood fault at the end of this reach.

7.7.7 Reach 7 - Santa Monica Mountains

This reach extends from Yucca Street to Universal City Station and includes Borings CEG 28A through 34.

7.7.7.1 General Description of Reach

This reach crosses the Santa Monica Mountains and will be the only hard rock reach along the 18-mile alignment (Drawing 1). The Topanga Formation rocks, from south to north, include:

- hard, well-cemented massive sandstone and conglomerate with local soft, thin siltstone beds
- basalt, which is deeply weathered at the surface, but hard and jointed at tunnel depths
- relatively soft, thinly bedded siltstone, with local hard sandstone beds.

Based on surface outcrops and core boring data, these rocks have been classified according to Terzaghi rock condition numbers, as shown on Drawing 2. In general, the sedimentary rocks range from hard and stratified to moderately blocky and seamy. At tunnel depth the basalt is generally massive, to moderately jointed.

Five major fault zones--the Hollywood, Hollywood Bowl, Benedict Canyon, and two unnamed faults--cross the tunnel alignment. Crushed rock and fault gouge should be anticipated in the fault zones, with zones of very blocky and seamy rock on each side of the faults. Squeezing or swelling ground is possible in the vicinity of the major fault zones. The width of each individual fault zones is not known, but is believed to be several hundred feet.

Throughout most of this reach, the sedimentary rocks are steeply dipping, in the range of 40° to near vertical. Bedding strikes at 30° or more to the tunnel alignment. Major fault zones cross the alignment at nearly right angles. With this orientation, most zones of weak or seamy ground will be crossed in a distance about equal to the width of the zone.

7.7.7.2 Tunneling Considerations

Tunnels can be driven through these rocks by conventional drill and blast methods, or with a properly equipped hard-rock boring machine (TBM). Different methods of initial support can be used in this reach, depending upon the construction method selected. Throughout most of the reach, the arch can be stabilized with rock bolts (O'Neill, 1966). More extensive supports, such as conventional steel ribs and lagging, will probably be needed in the fault zones. Rock bolts could be the standard expansion shell type. Epoxy resin-anchored bolts will be more effective in some of the low hardness and seamy rock areas. If TBM excavation is considered, the machine must be designed for the placement of initial support, such as steel sets and lagging or rock bolts, close behind.

Mixed face conditions should be expected at each end of this reach where the hard rock drops beneath the Hollywood and North Hollywood alluvial basins (Drawing 1).

If a cut-and-cover line alternative is selected for the north end of Reach 6 along Cahuenga Boulevard, the south Portal for Reach 7 can be constructed in rock just north of Boring CEG 29. At the north end of Reach 7, the rock surface slopes beneath the alluvium more gradually. A transition from rock to alluvium, with mixed face conditions, must be expected.

Hardness and unconfined compression strength tests for machine borability were conducted on several of the competent core samples by TBM manufacturers. Results of these tests are included in Appendix 1, Volume II, but some unconfined compressive strengths are also tabulated below:

- Weak Topanga Formation

  Unconfined Compressive Strength (Qu) - Range = 326 to 775 psi
  (1 hole - 2 Tests)

- Massive Topanga Formation

  Unconfined Compressive Strength (Qu) - Range = 3,622 to 9,072 psi
  (1 hole - 2 tests)

- Fractured Basalt

  Unconfined Compressive Strength (Qu) - Range = 1,811 to 5,040 psi
  (1 hole - 2 tests)

- Massive Basalt

  Unconfined Compressive Strength (Qu) - 9,300 psi
  (1 Test)

A tunnel through the Santa Monica Mountains will be below the ground water level (Drawing 2). The generally dripping conditions should be expected with occasional flows of up to several hundred gallons per minute (gpm). These large flows should diminish within a few hours to a few days. The records from the two tunnels previously driven through the Santa Monica Mountains suggest the flows of water that can be anticipated (Sections 6.3 and 6.4).

The 9-foot-diameter L.A. Sewer Tunnel was constructed in 1954-56 by drill and blast methods. It traverses through the Santa Monica Mountains near the proposed Metro Rail alignment, but at a higher elevation (Drawing 1 for location, and Drawing 11 for subsurface location) relative to the proposed Metro tunnel. This sewer tunnel encountered inflows greater than 100 gpm for a few hours to one day at 7 locations along its 2.8 mile length. Maximum water flow from the down-gradient south portal was reported to be 1,200,000 per day (gpd), or 800 gpm with average flows between 100,000 and 400,000 gpd (Figure 6-2). Based on the final selected tunnel grade, the Metro Rail tunnel could pass under the existing L.A. Sewer Tunnel (Figure 6-2).

MWD's Hollywood Tunnel (Figure 6-3) was constructed in 1940-41 by drill and blast methods. It is located at about elevation 770, about 300 feet above the L.A. Sewer Tunnel. For a few hours, maximum ground water inflow was 600 gpm, from water stored in fractured basalt. Most common water sources were seeps that totalled 250 gpm (maximum) at the downslope portal (vs. 800 gpm in the L.A. Sewer Tunnel).

Gas was not detected in any of the borings drilled throughout this reach, nor was gas reported in the previous two tunnels. Therefore, Reach 7 is classified as non-gassy.

### 7.7.8 Reach 8 - San Fernando Valley

This reach, which extends from Universal City Station to the north Portal, includes Borings CEG 35 through 38.

#### 7.7.8.1 General Description of Reach

The San Fernando Valley Reach is underlain by a relatively thin layer of granular Young Alluvium over a thicker layer of granular Old Alluvium (Drawing 2). Interbedded claystone and siltstone underlie the alluvial sediments at increasing depths as the alignment proceeds northward. At Boring CEG 34, the depth to rock is about 50 feet. The rock surface drops to a depth of about 125 feet in Boring CEG 35, and to more than 200 feet throughout the remainder of the reach.

The alluvial sediments consist of 50 to 100 feet of Young Alluvium, represented predominantly by Unit $A_1$ medium dense, coarse-grained sands and gravels. The Young Alluvium is underlain by Old Alluvium which is predominantly of the $A_3$ Unit, dense to very dense, silty to gravelly sands. Toward the mountains, layers of clayey and sandy silt are interbedded with the sands.

#### 7.7.8.2 Shallow Tunnel and Cut-and-Cover

If the tunnel profile rises near the surface at the Universal City Station and exits the hard rock formations in the vicinity of the Los Angeles River, a shallow tunnel alignment within 50 feet of the surface is feasible, but is not preferred. Such a tunnel would pass through Young Alluvium (Unit $A_1$) and would encounter sands and gravels with the potential for running and raveling ground. The tunnel alignment would be above the water table, hence ground water problems would not intensify the tunneling conditions. Similar to Young Alluvium in Reach 1, these alluvial deposits could contain boulders as large as 4 feet in diameter. The method selected to drive the tunnel must be capable of handling material that size.

Laboratory tests of cohesive units of Young Alluvium in this reach gave the following results:

| | |
|---|---|
| Undrained, Quick, Direct Shear (5 Tests) | $\phi$ - Range = 32.5° to 39°; average 36° |
| | C - Range = 0.19 to 0.56 ksf; average 0.41 ksf |

Ground water would be encountered at the southern end of this reach, where it is within 20 feet of the surface, as measured in Boring CEG 34. North of the Los Angeles River, the water level is at depths of 125 to 150 feet. This water level can change radically within one year, depending upon water pumping or re-charge by local agencies in San Fernando Valley (Section 4.5 for further discussion).

Gas was not detected in borings along this reach, and therefore it is classified as non-gassy.

Cut-and-cover construction is preferred to tunneling in this reach.

7.7.8.3 <u>Deep Tunnel</u>

A deep tunnel is considered feasible through the Old Alluvilum, unit $A_3$. Ideally, the tunnel should be kept above ground water level which, as measured in 1981, ranged from a depth of 110 feet in Boring CEG 36, to 140 feet in Boring CEG 38. Except for the portion of tunnel leaving the Universal City Station, which would be constructed in the relatively hard rock Topanga Formation, the tunnel would be driven through Old Alluvium, predominantly of the $A_3$ coarse-grained type.

Tests of the Old Alluvium gave the following values:

| | |
|---|---|
| Unconfined Compressive Strength (Qu) (8 Tests) | Range 9.3 to 46.2 psi; average 28 psi |
| Undrained, Quick, Direct Shear (16 Tests) | $\phi$ - Range = 6.5° to 40.5°; average 34° |
| | C - Range = 0.16 to 1.4 ksf; average 0.48 ksf |

The ground is firm, but is potentially a running and raveling type, which could be handled with appropriate provisions on the shield.

The tunnel will cross at least 600 feet of saturated Young Alluvium at the south end of the reach after exiting the hard-rock Topanga Formation. Flowing ground conditions in the $A_1$ and $A_3$ units can be expected. With the Los Angeles River above, dewatering of the Young and Old Alluvium by pumping wells is probably not an economical solution. Further exploration and testing of ground water conditions in this area are necessary to confirm this conclusion.

# Section 8.0
# Anticipated Ground Behavior in Surface Excavations

# Section 8.0: Anticipated Ground Behavior in Surface Excavations

## 8.1 GENERAL

This section presents an assessment of the anticipated ground behavior in surface excavations. The purpose of this assessment is to provide geotechnical information suitable for current design, evaluating alignments, and cost estimating. To minimize repetition, this section includes references to other sections, particularly previous Section 5.0, Geologic Features of Engineering Significance, and Section 7.0, Anticipated Ground Behavior in Underground Construction, and upcoming Section 9.0, Design Considerations.

Surface excavations associated with this project include:

- Portal excavations
- Cut-and-cover stations
- Cut-and-cover line segments
- Shafts for stations and ventilation.

To aid in determining the optimum profile elevations, we have evaluated several alternative excavation depths, consistent with those depths discussed in Section 7.0.

The primary geotechnical constraints are: maximum practical cut-and-cover excavation depths, potential problems associated with cuts into the tar sands, and protection/underpinning of existing structures and ground water.

## 8.2 SHORING REQUIREMENTS - GENERAL CONSIDERATIONS

Owing to the limited available construction space and nature of the near-surface soils along the alignment, virtually all surface excavations will require shoring.

For some grade alternatives being considered, there is the possibility at proposed stations for constructing shored excavations in excess of 120 feet. These depths are unprecedented in the Los Angeles area. There is a practical limit to the depth of a shored surface excavation. This limit depends on factors including: the ground condition, ground water level, nature of existing adjacent structures and economics. For the purpose of current design and cost estimating, the following limits are noted:

- <u>Standard Practice</u>
  Shored excavations in the "soft-ground" geologic units (i.e., the Young and Old Alluvium, the San Pedro Formation (sand), and the Fernando and Puente formations) to depths of 80 feet can be designed and constructed along the majority of the alignment, using proven and routinely available practices. Exceptions may include Reach 1, an area of high ground water table (see Section 8.6) underlain by permeable Young Alluvium, and Reach 5 in the tar sands (see Section 8.10).

- State-of-the-Art Practice
  Excavations in excess of 80 feet have been successfully completed in the Los Angeles area for an alluvium overlying Fernando/Puente Formation condition, with a current maximum excavated depth of about 110 feet. However, such depth should be considered exceptional, requiring detailed analyses, since it might not be appropriate for all site conditions. The detailed analyses would have to consider subsurface conditions, excavation geometry, specific design and construction procedures (type of wall, etc), and adjacent existing structures. For purposes of current studies, it is recommended that, under ideal conditions and with an adequate detailed investigation, the maximum practical limit for tieback excavations be about 125 feet, and for internally braced excavations, 150 feet.

## 8.3 SHORING SYSTEMS

There are numerous available shoring systems that could be utilized along the alignment, with a wide assortment of specific detailed design and construction options. In addition, considering the magnitude of the Metro Rail Project, innovative designs and construction procedures may be developed, particularly in conjunction with experienced contractors. For the purposes of this study, the shoring systems evaluated were those that appear suitable based on available project information and current area practices. The shoring systems discussed in this section include:

- Soldier Pile Walls
  These could be supported with tieback anchors, cross-braced struts, and/or a combination of these. Depending on the material being supported, lagging may be required. Lagging conventionally consists of wooden timbers, but could also consist of pre-cast concrete or shotcrete which would be incorporated into the permanent wall. One major advantage of soldier pile walls is the extensive local experience with such walls developed over the last 15 to 20 years.

- Slurry Walls (cast-in-place)
  Installation of a cast-in-place slurry wall consists of excavating a narrow trench or slot full-depth along the wall line in short sections, typically 10 to 20 feet in length. The excavation is carried out in the wet, using modified excavating tools, with support of the trench walls provided by the fluid pressure of bentonite slurry. Reinforcing is lowered into the excavation and tremie concrete poured to construct the wall. Trench stability is normally evaluated on the basis of experience and test sections. Single-panel units can be combined to form virtually any wall geometry composed of line segments. The wall could be braced with tieback anchors, struts, and/or a combination of these.

  Although in the Los Angeles area slurry walls have historically been used mostly for temporary walls, there may be selected station locations where slurry walls can be utilized for both the temporary and permanent walls.

It should be realized, however, that there are usually undesirable irregularities that show up on an installed slurry wall which defy design and construction control, and there are generally problems involved with integrating the slurry wall with the horizontal slab members to produce the permanent frame construction. In addition, the tightness of the slurry wall itself and its interfaces with the roof and invert slab would be particularly critical at the Metro Rail alignment locations where oil, gas and impure water must be fought. Also, certain U.S.A. experiences in station installations have revealed that the utilization of slurry walls as the permanent wall actually adds to the total cost of the construction to an appreciable degree.

Slurry walls also provide the opportunity to investigate the use of top-down or inverted construction procedures for station locations in areas of cut-and-cover line segments. Such a procedure would not be feasible for a deep line section, since temporary bracing elements will be required for a deep cut and the time for excavation and backfill will be significant. Also, where underground stations with mezzanine levels are involved, it cannot be assumed that temporary tiebacks or struts can be eliminated, since the distance from the roof slab to the excavated subgrade level will be significant. Further, top-down construction can sometimes, but not always, reduce the impact on local traffic, since the traffic will have to be detoured completely from the project site during the entire period of excavation, concreting and backfilling operations. The advantage of top-down construction is normally only in the earlier final restoration of the street surface, compared to a conventional temporary decking scheme.

- Pre-Cast Walls
  Pre-cast units could be installed in either augered holes (similar to soldier pile walls) or in slurry trenches similar to the slots excavated for cast-in-place slurry walls.

- Interlocking Steel Sheet Piles
  In most areas, steel sheet piles will probably not be appropriate due to the difficulty of driving the sheets, particularly into bouldery soils and/or weak rock. In some areas, however, sheet piles may be feasible for relatively shallow excavations.

As discussed above, either tiebacks or struts, or a combination of these, could be used for supporting temporary walls. General comments concerning tiebacks and struts include:

- The economics of tiebacks versus struts depends on many factors, but is primarily a function of excavation width. For narrow excavations, such as cut-and-cover line segments on the order of 40 feet or less, struts are normally more economical. In fact, tiebacks are probably not feasible in narrow excavations, due to the space limitations and the size of the tieback installation equipment. For wide excavations, on the order of 100 feet or more, tiebacks are normally more economical. Unless there are constraints due to the permanent structure, the contractor should probably be given the option to choose the most appropriate shoring system.

- Based on the available field data, there does not appear to be a significant difference between the anticipated performance of properly designed and constructed tieback and strutted excavations.

- Along the majority of the alignment, tiebacks would extend below existing structures. Obtaining easements, the problems of basements obstructing tiebacks, and the possible effect of tieback stresses on the overlying stuctures are all considerations which must be resolved.

- There has been limited experience in the Los Angeles area with depths over 100 feet for tieback walls for long, narrow excavations.

- There is some evidence (Peck and Palladino, 1977) that deep excavations, particularly with plane strain conditions, into materials such as the Puente claystone (which behaves essentially like a heavy, overconsolidated clay), could fail or sustain significant movement at some limiting depth. This problem involves release of lateral stresses combined with bedding plane weakness along plastic claystone layers. Strutted excavations would be less susceptible to this problem since the majority of the tieback anchors would be founded above the potential failure plane and would provide little resistance to movement along such planes of weakness. Using an extended no-load zone for locating the tieback anchors will reduce this problem.

- Tiebacks have several advantages over strutted excavations which tend to result in less ground movements (after Goldberg-Zoino, 1976):

  a. In granular soils, in which the soil modulus increases with stress level, the prestressed soil mass engaged by tiebacks is made more rigid and therefore less deformable.

  b. Tiebacks are typically prestressed to about 125 percent of the design load and then locked-off between 75 and 100 percent of the design load. Prestressing in this manner prestrains and stiffens the soil and pulls the wall back toward the soil to remove any "slack" in the contact zone.

  c. Internal bracing, if prestressed, is usually to about 50 percent of the design load. Typically, the bracing gains in load as the excavation deepens. Elastic shortening of the strut continues after installation of the member.

  d. Temperature strains are more important with bracing than with tiebacks because the latter are insulated in the ground.

  e. Internal bracing is removed, then rebraced to facilitate construction, whereas tiebacks do not have to be removed.

  f. There is less incentive for the contractor to excavate a significant depth below the designated support level prior to installing tiebacks than struts. Thus, the risk of the contractor excavating too far prior to installing lateral supports (which can result in a significant increase in ground movement) is considered less for tiebacks than struts.

- The advantage of struts over tiebacks in reducing settlement relates to the way each of these systems derives its support. Struts transmit the earth pressure loads through the bracing from one side of the excavation to the opposite side. Tieback walls transmit the earth pressures back into the soil mass, which may result in affecting a larger area of the soil mass behind the support system.

For current design purposes, subject to the depth limitations presented in Section 8.2, we recommend that it be assumed tieback and strutted excavations will have similar performance.

## 8.4 DEEP EXCAVATIONS IN LOS ANGELES

Current practice for major and/or deep excavations in the Los Angeles area consists of soldier piles with tieback anchors. Four case studies involving deep excavations in materials similar to those which will be encountered along the proposed alignment follow.

### 8.4.1 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 an unconfined 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.

### 8.4.2 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 prestress loads in the anchors.

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

### 8.4.3 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 deflections 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.

8.4.4 <u>Mutual Benefit Life Building</u> (Converse Davis and Associates, 1968)

Although this project involved only a 40-foot deep excavation, it is of special interest since the excavation extended some 10 feet into the tar sands. The project is near Borings CEG 19 and 20 and the proposed location of the Fairfax Avenue Station. Key elements of the design and construction included:

- Subsurface materials consisted of 25 feet of silt, sands and clay with isolated lenses of asphalt (Old Alluvium) underlain by tar sands.

- Shoring system consisted of soldier piles with belled tieback anchors.

- A permanent collector system was provided below the bottom of the slab to collect water and oil and to limit hydrostatic pressures and infiltration of oil into the structure. It was noted at the time that pits had been installed at the nearby California Savings and Loan Building to relieve the tar pressure; thirty-eight 50-gallon barrels of oil were removed every two months; there the oil tars were penetrating areas such as construction joints, around pipes, conduits, etc; pulsating equipment acted as a pump to pull the oil tar through cracks and joints.

- Gas was noted bubbling out around the piles and pile cap (building supported on friction piles bearing below the tar sands).

- Some movement at the perimeter of the excavation developed. The major movement developed when the excavation was 20 to 30 feet deep along Wilshire Boulevard. The tar sand started at about 20 to 25 feet. Movement appeared to be mostly horizontal, increasing to a maximum of about 3 inches, then virtually stopping. Cracks in the street extended 30 to 40 feet away from the edge of the excavation. Some settlement occurred near the edge of the excavation and the soldier piles showed little tilting or deformation.

## 8.5 PROTECTION/UNDERPINNING OF EXISTING STRUCTURES - GENERAL CONSIDERATIONS

### 8.5.1 General

Most of the proposed stations and potential cut-and-cover line segments will be constructed in close proximity to existing structures. In many areas, such as along Wilshire Boulevard, there may be approximately 10 to 20 feet between the excavation and the existing building foundations. The protection of these existing structures, which are normally supported on foundations bearing above the proposed line segment/station level, must be incorporated into the design and construction of the project.

In general, the alternatives available include:

- Select Undeveloped Areas
  To the extent possible, major surface excavations should be located adjacent to undeveloped areas (such as parks and parking lots) and/or areas containing small structures.

- Relocate or Condemn Structures
  Small and/or relatively inexpensive structures could be relocated and/or condemned. In many cases, the additional cost of an excavation designed to protect these structures may be more costly than the structures themselves.

- Construct Adequate Temporary Walls
  In many cases, it may be feasible to design and construct a temporary wall which, with adequate bracing, limited excavation stages and controlled dewatering, would minimize earth movements and allow the excavation to be constructed adjacent to existing structures. The example cited in Section 8.4.3 involved this type of solution. However, regardless of the care taken in the design and construction of the wall, there is a risk that the adjacent structure may be damaged. This risk increases with increasing excavation depths and decreasing distances between the existing building foundations and the new excavation. To the extent possible, deep excavations should be constructed adjacent to non-critical areas first. The knowledge gained from these excavations will improve the level of confidence for subsequent construction in more critical areas and reduce the risk of causing damage.

- Underpin
  There will be locations where the risk and consequence of damage due to earth movements will be unacceptable, and underpinning may be prudent. These include areas of poor soil conditions, deep excavations in close proximity to existing structures, and/or areas of major structures.

There are two additional foundation strengthening alternatives in lieu of underpinning:

a) Chemical grouting in sandy soils can be used to prevent soil runs and strengthen soil in critical areas. Grout could be injected from the surface under foundation elements.

b) Compaction grouting in sands, silts and clays to lift lightly-loaded structures. Again, the grouting is carried out from the surface.

Both of these techniques have been successfully used in the Los Angeles area, in the Washington, D.C. and Baltimore Metro projects and are used extensively in Europe and Japan.

### 8.5.2 Guidelines for Evaluating the Need to Underpin

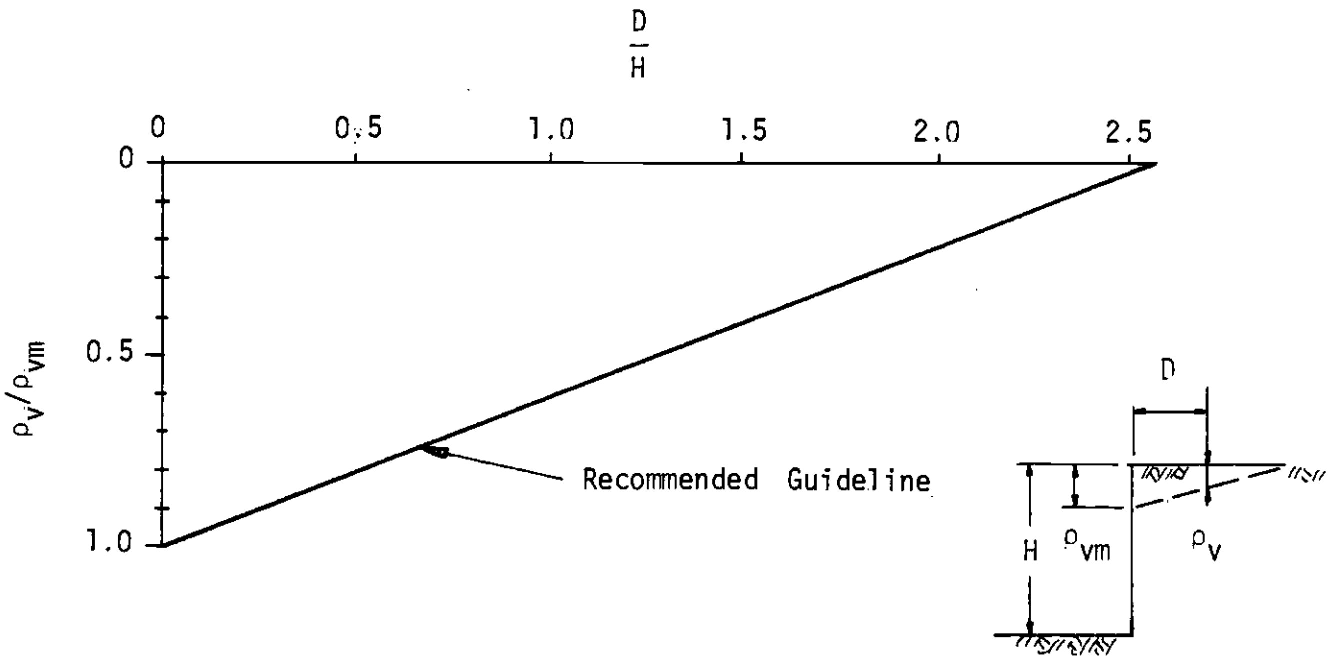
In assessing the need to underpin, both the range in anticipated earth movements and the resulting impact on the adjacent structure should be evaluated. This would include consideration of the type of structural framing, allowable building movement, (vertical, horizontal and angular distortion) and soil-structure interaction.

It is not possible to eliminate all ground movement adjacent to a shored excavation. However, for the generally good ground conditions encountered along the proposed alignment, it is feasible to design and construct an excavation which will result in minimal ground movements. Based on past experience for excavations in similar subsurface conditions in the Los Angeles area and elsewhere, Figure 8-1 was developed. Figure 8-1 shows the maximum vertical ground movement anticipated adjacent to a properly designed and constructed shored excavation under ideal ground conditions.

In subsequent sections correction factors are provided for increasing the estimated movements of Figure 8-1 under less than ideal conditions at specific reaches. These are guidelines to evaluate possible ground movement for current design purposes and are subject to revision based on future additional data and analyses. The maximum horizontal movement at the top of the wall is anticipated to be about 50% greater than the maximum vertical movement. There is insufficient data to estimate the distribution of horizontal movement away from the wall.

Figure 8-2 presents the general underpinning criteria recommended for the Metro Rail Project. These criteria may be used for the current design and cost estimating. It may be possible, at some future stage of the design, to develop less conservative criteria at sites which are studied in detail.

As an alternative approach to the criteria presented on Figure 8-2, a method using the guidelines for the distribution and magnitude of ground movement behind properly designed and constructed excavations shown on Figure 8-1 is suggested. These guidelines can be used in conjunction with the foundation design of the existing buildings (i.e., depth, basement levels, piles or spread footings, etc.) and the depth and width of the proposed line segment or




NOTES:

H: Depth of cut, in feet.

D: Horizontal distance away from excavation, in feet.

$\rho_{vm}$: Maximum vertical ground movement at wall, in feet. For ideal conditions with adequate groundwater control, $\rho_{vm} = 0.003 \cdot H$

$\rho_v$: Vertical ground movement at specified distance from wall, in feet.

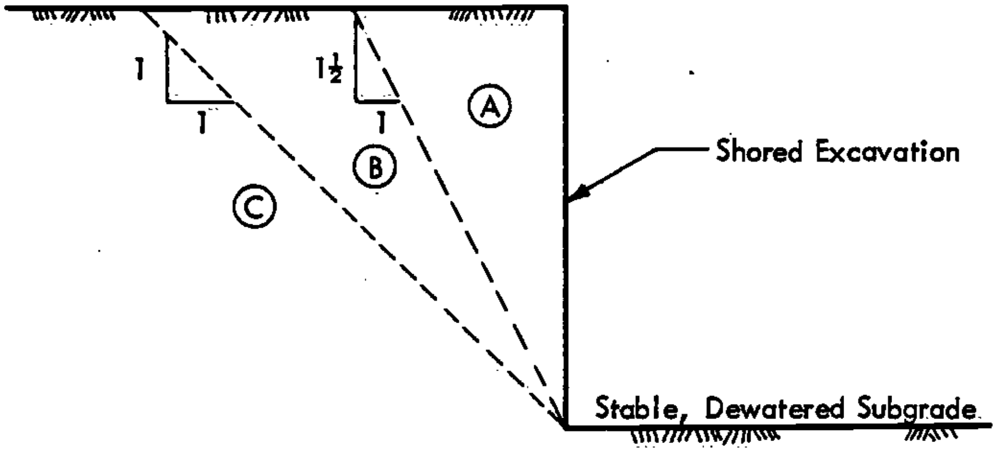
GUIDELINES FOR ESTIMATING VERTICAL GROUND MOVEMENT ADJACENT TO A SHORED EXCAVATION

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

Figure No. 8-1

CWDD/ESA/GRC Geotechnical Consultants




Ⓐ Foundations of Important Structures in this Zone Generally Must be Underpinned.

Ⓑ Foundations in this Zone Generally Not to be Underpinned Except Where Underlain by Weaker Clays, or Structure is Especially Sensitive.

Ⓒ Underpinning Elements to Receive their Support in this Zone or below Subgrade Level.

CRITERIA FOR UNDERPINNING

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No.
8-2

station to evaluate anticipated impact of the excavation on the adjacent existing structure(s). Based on these evaluations, the need to underpin can be determined.

Shoring systems adjacent to existing buildings that will not be underpinned should be designed to support the additional earth pressure imposed by the building loads.

### 8.5.3 Methods for Controlling Ground Movement

The guidelines presented on Figure 8-1 are estimates based on past experience. In marginal areas, where underpinning could be minimized or eliminated by limiting ground movement, we recommend that the following design and construction procedures be considered:

- There is some evidence that use of cast-in-place slurry walls may result in less ground movement than more conventional soldier pile walls. It is understood that in areas where slurry walls were used in the BART system in the San Francisco area, underpinning was normally not required. Where slurry walls were used on the Washington, D.C. Metro, underpinning also was not required. In Baltimore, for the Charles Street Station, underpinning was specified but could not be accomplished. After an expensive try, the process was abandoned and a slurry wall concept was used very successfully. In this case, the wall passed very close to adjacent buildings and even cut off parts of several footings of the buildings. Chemical grouting was also used to prevent soil runs in very critical areas.
- Requiring a close vertical spacing between tieback or struts can be effective in reducing movement.
- Requiring a high anchor or strut pre-stress load can be effective in reducing movement. However, there is a limit to pre-stressing beyond which the wall is actually pulled into the soil, creating soil heave behind the wall.
- Dewatering prior to excavating in granular soils and/or soils with granular zones can minimize loss of ground associated with piping/flowing ground.
- Probably the single most important factor is the quality of the construction. Thus, appropriate specifications, experienced contractors, and field observations during construction are essential.

### 8.5.4 Underpinning Method

Underpinning will probably involve installation of piles which will extend below the zone of influence of the excavation. For current design purposes, the zone of influence can be assumed to extend upward from the base of the excavation at a 1H:1V slope, as shown in Figure 8-2. Depending on the proximity of the existing structure to the proposed excavation, it may be feasible to use soldier piles for the temporary wall for the permanent vertical support of the adjacent structures. This is a common practice in many areas of the country.

The process of underpinning can often result in settlement. For current design purposes, it should be assumed that properly designed and constructed underpinning may result in 1/4 to 1/2 inch of settlement (Goldberg, Zoino, 1976). Underpinned structures can still be damaged by lateral movements toward the excavation. This problem should be assessed during the design stage.

## 8.6 REACH 1 - DOWNTOWN, LOS ANGELES RIVER

### 8.6.1 General

Reach 1 extends from the east Portal near Gallardo Street westerly about 1.2 miles to the Hollywood Freeway. Through this area, the proposed alignment crosses several major streets, the Hollywood Freeway, the concrete-lined Los Angeles River, the railyard of Union Station, and major gas and oil transmission lines. The area includes numerous one- to four-story industrial buildings with some parking lots and undeveloped areas.

The current design concept includes a station in the vicinity of Union Station in the existing railyard and possibly some cut-and-cover line segments. Based on the recommendations presented in Section 7.0, the following line structure depths may be considered:

- Shallow (less than about 30 feet deep)
  This would involve constructing the rail at the surface or in a line structure by cut-and-cover construction. At this depth, most of the excavation would be above the ground water table, minimizing dewatering requirements.
- Intermediate (30- to 130-foot-depth)
  At these depths, much of the excavation will be below the ground water table in coarse-grained alluvium. Thus, flowing ground conditions can be anticipated throughout, requiring dewatering.
- Deep (130- to 200-foot-depth)
  At this depth, the line would be excavated entirely within the Puente Formation below the alluvium.

This reach crosses the Los Angeles River which consists primarily of coarse-grained Young Alluvium ($A_1$) with some local areas of man-made fill underlain by the Puente Formation (C), as shown on Drawings 2 and 5. Based on available information, the general conditions along the proposed alignment consist of the following:

- East of Los Angeles River
  Borings CEG 1 and 2 encountered about 10 to 30 feet of Old Alluvium ($A_3$) consisting primarily of silty sands and gravelly sands. These soils are medium to dense based on standard penetration resistances which ranged from 20 to 80 blows per foot and averaged about 40 blows per foot. The underlying Puente Formation at Boring CEG 1 was primarily claystone, while at CEG 2, it was primarily siltstone.

- West of Los Angeles River

  West of the Los Angeles River, the thickness of the Young Alluvium ($A_1$), as encountered in Borings CEG 3 through 6, averaged about 80 feet and consisted primarily of gravelly sand and sandy gravel with cobbles and boulders. The standard penetration resistances ranged from 40 to over 100 blows per foot.

  Although the gravel content affected the blow counts, the alluvium is probably medium to dense. The underlying Puente Formation at Borings CEG 3, 4 and 5 was primarily claystone, while at Boring CEG 6 siltstone and sandstone were encountered.

- Near the Hollywood Freeway

  As shown on Drawings 2 and 5, the thickness of Young Alluvium decreases from about 80 feet at Boring CEG 6 to less than 10 feet at the Hollywood Freeway, a horizontal distance of about 2,000 feet.

Based on the data collected for this study in the winter of 1980-1981, the ground water table through this reach occurs within the Young Alluvium at a depth of about 20 to 35 feet.

### 8.6.2 Excavation Conditions

Above the ground water table, the Young Alluvium can be readily excavated with conventional earthmoving equipment. Boulders up to about 1 cy should be anticipated. Safe, temporary construction slopes should be the responsibility of the contractor; however, for estimating purposes, 1H:1V temporary cut slopes can be assumed, provided there is no seepage of ground water. Below the ground water table, excavations in unsupported cuts will require dewatering and flatter slope due to the relatively high permeability of the Young Alluvium.

Experience shows that the underlying Puente Formation can be excavated with conventional earthmoving equipment and generally does not require ripping. In sections containing sandstone concretions, blasting or heavy ripping may be required. Hard sandstone concretions, up to 2 to 3 feet in maximum thickness, are judged to represent less than 1/10 of 1 percent of the unit in vertical sections. Unsupported cut slopes are expected to be stable at 3/4H:1V unless the bedding is unfavorable; i.e., dips into the excavation. In areas of unfavorable bedding, flatter excavation slopes on the order of 1 1/2H:1V and benching may be required. Based on surficial mapping, unfavorable bedding will probably be encountered in the vicinity of the Hollywood Freeway in excavations having free faces oriented about east-west and sloping south.

### 8.6.3 Dewatering Requirements

From the available information, the permeability of the Young Alluvium ($A_1$) is probably high and may be on the order of $10^{-1}$ to $10^{-2}$ cm/sec with local zones of gravel where the permeabilty may exceed $10^{-1}$ cm/sec. The effective dewatering of such a permeable unit may be difficult and expensive. To the extent possible, the station and cut-and-cover line segments should be constructed above or no more than about 10 feet below the ground water table, (i.e, at a maximum depth of about 30 feet), or below the alluvium, entirely within the underlying Puente Formation. The permeability of the Puente Formation is low, due primarily to fractures. Inflow from this unit is anticipated to be minor. However, excessive water in the claystone unit can soften the materials, causing vehicle mobility problems, slowing excavation rates and reducing claystone strength.

Should it be necessary to excavate below the ground water table in the alluvium to construct either the station or a cut-and-cover line segment, several procedures should be considered for control of ground water. These include:

- Shored Excavation/Open Pumping
  Depending on the depth of cut and actual soil permeability, it may be feasible to construct a shored excavation by pumping from ditches and shallow sumps within the excavation. This procedure is probably feasible only for excavating into predominately silty sands and/or for depths less than 10 feet below the natural ground water table. To minimize piping/flowing of soil through the shoring lagging, a heavy filter fabric could be installed behind the lagging as the excavation is advanced. If stratified zones of low permeable soils, underlain by the permeable alluvium, are encountered, it may be necessary to install deep wells inside the excavation to intercept and depressurize this confined zone and minimize problems associated with bottom heave of the excavation.

- Deep Wells
  A series of properly designed and installed deep wells could be installed around the excavation to lower the ground water table.

- Wellpoints
  In some situations, wellpoints may be an economic alternative to deep wells.

- Slurry Walls
  A slurry wall could be used to limit the lateral seepage of ground water. However, the bottom of the excavation would still require dewatering. For deep excavations, it may be feasible to extend the slurry wall into the underlying Puente Formation, thus providing an effective ground water seal. This would allow excavation with a minimum of seepage.

- Slurry Cutoff Trench
  A continuous slurry cutoff trench (filled with soil-cement or soil bentonite) extending into the Puente Formation could be placed around the proposed excavation and effectively cut off seepage from the Young Alluvium.

To evaluate the above procedures and estimate seepage rates for current design and estimating purposes, reasonable ranges of anticipated inflow rates are presented in Table 8-1.

TABLE 8-1 Anticipated Inflow Rates

| Depth Below Ground Water Table (ft) | Estimated Inflow Rates/Side (gpm/100 ft of Excavated Length) |
|---|---|
| 10 | 50 - 500 |
| 20 | 75 - 1,000 |
| 40 | 100 - 1,250 |

Excavation of shafts below the ground water table in the Young Alluvium ($A_1$) will also require control of the ground water. These might include ventilation shafts to the tunnel and/or inclined shafts for pedestrian access to a deep station. The shaft will require a water-tight lining below the water table and a positive seal in the underlying Puente Formation. To advance the shaft and install the lining, it will be necessary to either dewater the permeable zone or seal the permeable zone. As an alternative to normal shaft construction, it may be feasible to sink a caisson through the alluvium and seal it in the rock.

### 8.6.4 Shoring Requirements

General shoring requirements are discussed in Sections 8.2 and 8.3. Section 9.0 presents design criteria. Through Reach 1, shored excavations may be required at the east Portal, at Union Station, and for any cut-and-cover line segments.

From the east Portal to about the Los Angeles River, subsurface conditions consist of shallow Old Alluvium above the water table underlain by the Puente Formation. These conditions are similar to those encountered at the Atlantic Richfield Project, as discussed in Section 8.4.1, and are suitable for deep shored excavations. Soldier piles with tiebacks or internal bracing would be appropriate in this area. Slurry walls may also be appropriate, provided the walls can be economically justified. Steel sheet piles may not be feasible even for shallow cuts due to the occurrences of cobbles and boulders in the Old Alluvium. Ground water will probably be encountered below the Old Alluvium, but flows are expected to be small. Lagging or similar support will be required to eliminate raveling of the Old Alluvium. If conditions are similar to those encountered at the Atlantic Richfield Project, continuous lagging will generally not be required in the underlying Puente Formation. However, a method such as gunite, wire mesh, or asphalt emulsion would be required to prevent deterioration of the Puente Formation and to protect workers from loosened material. Locally in seepage zones, lagging may be required.

North-westward from the Los Angeles River to within a few thousand feet of the Hollywood Freeway, the subsurface conditions consist of deep Young Alluvium saturated below a depth of 30 feet. The proposed design location of Union Station is in the vicinity of Boring CEG 5. For excavations into the Young Alluvium to depths of about 40 feet (or some 10 feet below the ground water table), soldier piles with lagging (or similar support) or slurry walls would be appropriate. These shoring systems would also be appropriate at greater depths, but would require control of ground water. Ground water control will become more and more difficult and expensive to accomplish as the excavation is deepened. Soldier piles with lagging will have the additional potential problem of piping/flowing of soil through the lagging. A heavy filter fabric placed behind the lagging would minimize the problem. For excavations below the water table, slurry walls have an advantage, since they would virtually eliminate seepage through the wall and eliminate the problem of soil piping/ flowing. Slurry wall design must avoid waterflow through joints or tieback holes. Wells should be employed when appropriate to control flow under the wall. For station depths exceeding about 60 feet, it may be economical to extend the slurry wall into the underlying Puente Formation and effectively seal the excavation from ground water seepage. Other methods of ground water control were discussed in Section 8.6.3.

In the vicinity of the Hollywood Freeway, the Puente Formation is exposed at the surface. Through this area shoring conditions are similar to the conditions east of the Los Angeles River.

### 8.6.5 Potential for Ground Movement

Through this reach, the potential mechanisms for ground movement include movements associated with surface excavations, lowering of the ground water, construction vibrations and earthquakes.

The guidelines for movement associated with excavations presented in Section 8.5 are applicable to this area above the ground water table. Below the ground water table, for soldier pile walls and if the ground is not adequately dewatered, it can be assumed that movements would be twice those suggested in Section 8.5.2. This increased estimate is to quantify the risk of loss of ground through soil piping/flow. In general, we would not consider soldier pile walls without dewatering as an acceptable support system in this reach.

Based on available information and past Los Angeles experience, lowering the ground water table is not anticipated to be detrimental.

From standard penetration resistances and the gravelly and cobbly nature of the deposit, the Young Alluvium is generally medium to very dense. The soils are not likely to be susceptible to liquefaction under seismic loading nor to significant densification due to construction vibrations.

### 8.6.6 Protection/Underpinning of Existing Structures

The need for underpinning structures through this reach will probably be minimal. This assessment is based on the type of material to be excavated and the relatively small size and low density of existing structures. For current design and cost estimating, underpinning may be required for the following conditions where the risk of damage to adjacent structures would be unacceptable:

- Structures which are very close to the proposed excavation and could not tolerate the estimated deflection guideline for braced excavations are presented in Section 8.5.2.

- If it is determined during subsequent studies that there are some specific areas of alluvium which are loose and susceptible to densification due to construction vibrations, underpinning of major structures within about 100 feet may be required. Alternatively, it may be more economical to require special procedures to minimize construction vibrations.

- Deep soldier pile excavations below the water table without adequate dewatering could result in loss of ground due to piping/flow of soils. If soldier piles with lagging and inadequate dewatering are anticipated, it is recommended that major structures be underpinned where foundations lie within a 1.5H:1V line extending up from the base of the proposed excavation.

## 8.7 REACH 2 - BROADWAY TO HARBOR FREEWAY

### 8.7.1 General

Reach 2 extends under the Hollywood Freeway southwest along Broadway about one mile to 7th Street and then west along 7th Street about one-half mile to the Harbor Freeway. Through this area, the proposed alignment lies generally along Broadway, then along 7th Street, and finally under the Harbor Freeway to Wilshire Boulevard. The area is heavily developed with commercial and industrial buildings up to about 20 stories, most with basements.

The current design concept includes two stations along Broadway at the Civic Center and 5th Street, and one station at 7th Street near Flower Street. Based on the recommendations presented in Section 7.0, the following line structure depths may be considered:

- Shallow (less than about 40 feet deep)
  This would involve a cut-and-cover line structure.
- Intermediate (40- to 100-foot depth)
  This would include excavations in the Young Alluvium above the ground water table.
- Deep (100- to 200-foot depth)
  This would allow tunneling below the alluvium in the Fernando Formation and would require mining of the stations.

This reach crosses an ancestral buried channel of the Los Angeles River consisting of Young Alluvium ($A_1$) with some local areas of man-made fill underlain by claystone and siltstone of the Fernando Formation (C), as shown on Drawings 2 and 5. Based on the available information, the general conditions along the proposed alignment consist of the following:

- <u>Hollywood Freeway to Approximately 3rd Street</u>
  Boring CEG 7 encountered 18 feet of fill and dense sandy gravel underlain by claystone.

- <u>Approximately 3rd Street to Harbor Freeway</u>
  Boring CEG 8 encountered dense gravelly sands to a depth of 74 feet underlain by stratified dense sandy silts and silty sands to the top of the Fernando Formation at a depth of 122 feet. Boring CEG 9 encountered 14 feet of fill underlain by dense sandy gravel with cobbles and boulders to a depth of 28 feet. Underlying the sandy gravel in Boring CEG 9 were stratified, medium dense to dense gravels, sands, and silts and stiff to very stiff clays which extended to the top of a claystone at 55 feet. Through this area, the depth to the Fernando Formation appears to range from about 50 feet on the sides of the prehistoric channel to as much as 150 feet in the center of the channel.

Based on data collected for this study in the winter of 1980-1981, ground water is deep and occurred along the interface of the alluvium and Fernando Formation.

### 8.7.2 <u>Excavation Conditions</u>

Due to the deep ground water table, excavation conditions along Reach 2 are substantially better than those anticipated through Reach 1 which has a relatively high ground water table. Dewatering will not be a constraint in this area, and geotechnical conditions are considered favorable.

Excavation conditions within the Young Alluvium are similar to those discussed for Reach 1 in Section 8.6.2 above the ground water table. For estimating purposes, it is recommended that 1 1/2H:1V temporary slopes be assumed for unsupported cuts in man-made fill and stratified silts and clays, provided there is no significant seepage.

Excavation conditions within the siltstones and claystones of the Fernando Formation are considered to be similar to those in the Puente Formation, as discussed in Section 8.6.2.

### 8.7.3 <u>Dewatering Requirements</u>

The ground water table appears to be near or below the bottom of the Young Alluvium. Thus, for excavations up to at least 100 feet deep within the alluvium, no significant dewatering is anticipated. However, local zones or pockets of perched water could occur within granular zones underlain by silts and clays.

Ground water inflows from the Fernando Formation are anticipated to be minor. As with the Puente Formation, excessive amounts of water in the claystone can soften the material, causing vehicle mobility problems, slowing excavation rates, and reducing claystone strength.

### 8.7.4 Shoring Requirements

General shoring requirements are discussed in Sections 8.2 and 8.3. Section 9.0 presents design criteria. Through Reach 2, shored excavation may be required at the Civic Center Station, 5th Street Station, the Flower Street Station and any cut-and-cover line segments.

The excavation for the Civic Center Station and any cut-and-cover line segments in the vicinity will be almost entirely in the siltstones and sandstones of the Fernando Formation and will be ideal for deep shored excavations. Shoring requirements in this area will be similar to those discussed in Section 8.6.4 between the east Portal and the Los Angeles River. The Atlantic Richfield Project, discussed in Section 8.4.1, involved a maximum of 112-foot-deep excavation under conditions similar to those anticipated to be encountered in this area.

The excavation for the 5th Street, Flower Street, and any cut-and-cover line segments will be primarily within the Young Alluvium above the ground water table. Shoring requirements will be similar to those discussed in Section 8.6.4 for Young Alluvium above the ground water table. Based on Boring CEG 9, frequent cobbles and boulders will be encountered. Near the Harbor Freeway, the depth to the Fernando Formation is shallow, and conditions will be similar to those discussed for the Civic Center Station. The existing Atlantic Richfield Project, discussed in Section 8.4.1, is located some 1,000 feet north of the proposed location of the Flower Street Station.

### 8.7.5 Potential for Ground Movement

Through this reach, the potential mechanisms for ground movements include movements associated with surface excavations and construction vibrations.

The general guidelines for movements associated with excavations presented in Section 8.5 are applicable through Reach 2.

The Young Alluvium is medium to dense and probably not susceptible to densification from construction vibrations.

### 8.7.6 Protection/Underpinning of Existing Structures

This reach is located in one of the most densely developed areas of downtown Los Angeles, with major buildings up to about 20 stories located along the majority of the reach. Proposed station locations are at the intersection of major streets adjacent to major structures. Due to limited street widths, the station excavations may extend virtually to the foundation lines of the existing buildings. Depending on the width and depth of the station excavation, most of the major structures adjacent to the station excavations may have to be underpinned.

Section 8.5 presents general guidelines for underpinning.

## 8.8 REACH 3 - EAST WILSHIRE BOULEVARD

### 8.8.1 General

Reach 3 extends along Wilshire Boulevard from the Harbor Freeway west some 2.5 miles to Normandie Avenue. The area is moderately to densely developed with small to large commercial and residential structures on the order of one to four stories high, but with some taller buildings on the order of 20 stories. Much of the area adjacent to Wilshire includes paved parking lots with some open space. The alignment will traverse MacArthur Park, which contains a small pond, and Lafayette Park.

The current concepts include stations at Alvarado Street, Vermont Avenue and Normandie Avenue. Based on the design recommendations presented in Section 7.0, the line segments and stations will probably be excavated in the Puente Formation at depths ranging from 40 to as much as 200 feet.

Ground conditions through this reach appear to be relatively uniform and consist generally of 20 to 30 feet of Old Alluvium ($A_4$) underlain by claystones and siltstones of the Puente Formation (C). The Old Alluvium encountered in the borings ranged from firm clayey silt (Boring CEG 11) to dense sands and silty sands. Borings CEG 12 and 14 encountered 32 and 17 feet, respectively, of loose sands and soft clays, which may be fill. The underlying Puente Formation consisted primarily of claystones and siltstones with a few thin, hard sandstone beds. Borings CEG 10 and 13 encountered soft, weathered claystone with standard penetration resistances of 20 to 40 blows per foot overlying the unweathered Puente Formation.

The permanent water table occurs within the Puente Formation at depths in excess of 100 feet. A perched ground water table probably occurs above the Puente Formation within the Old Alluvium at depths of about 15 to 25 feet. Boring CEG 11 encountered an artesian condition during drilling. There is insufficient information at this time to determine what caused this condition (i.e., deep or shallow artesian zone, related to expanding gas bubbles, etc.) and its impact on surface excavations. Additional information will be required during later design stages. Heavy inflow of ground water was encountered in Boring CEG 10 with a cobbly sand zone at a depth of 18 to 22.5 feet. The lake in MacArthur Park is located within a few hundred feet of the proposed Alvarado Street Station.

### 8.8.2 Excavation Conditions

The excavation conditions through Reach 3 will be similar to those of Reach 2 but may require some dewatering, as discussed below in Section 8.8.3

The fill encountered in Borings CEG 12 (32 feet deep) and CEG 14 (17 feet deep) contains building debris and rubble and may be difficult to handle due to their low density and high silt content. Construction cuts in these fill materials should be 2H:1V or flatter. The weathered claystones and clayey silt alluvium will also be difficult to handle, particularly in the presence of water. For estimating purposes, 1H:1V construction slopes in these materials are appropriate.

### 8.8.3 Dewatering Requirements

Excavations that penetrate to the top of the Puente Formation will probably have ground water inflows near the alluvium-claystone contact. In general, the inflows will be small, except locally within zones of clean sand and gravel (such as in CEG 10, at depths of 18 to 22.5 feet). The anticipated inflows can be dewatered with open pumping inside the excavation. Should the artesian condition in Boring CEG 11 be confirmed and be caused by a zone within about 20 feet below the base of the proposed excavation, the affected area should be dewatered prior to excavation. Suitable dewatering procedures would involve deep wells, wellpoints, and/or deep sumps.

Ground water inflows from the Puente Formation are anticipated to be minor.

The lake at MacArthur Park is not anticipated to adversely affect dewatering conditions, since the lake appears to be entirely within the Puente Formation.

### 8.8.4 Shoring Requirements

General shoring requirements are discussed in Sections 8.2 and 8.3. Section 9.0 presents design criteria. Through Reach 3, shored excavations may be required at the Alvarado Station, Vermont Avenue Station, and Normandie Avenue Station. Shoring requirements are similar to those discussed in Section 8.6.4 between the east Portal and the Los Angeles River where Old Alluvium is believed to be underlain at a relatively shallow depth to the Puente Formation. The Atlantic Richfield Project discussed in Section 8.4.1 included a maximum 112-foot-deep excavation in similar materials including up to 32 feet of silts and sands underlain by claystones.

Specific shoring considerations through this reach include:

° Old Alluvium and fills overlying the Puente Formation will require lagging.

° To minimize the potential for loss of ground within saturated, permeable zones near the alluvium-Puente contact, it may be necessary to place filter fabric behind the lagging.

° To limit ground movement, temporary walls through fill and alluvium clays should be designed for higher earth pressures, as discusssed in Section 9.0

° Tieback anchors should not derive support from any fill materials.

### 8.8.5 Potential for Ground Movement

The general guidelines for movements associated with excavations presented in Section 8.5 are applicable through Reach 3.

### 8.8.6 Protection/Underpinning of Existing Structures

The distance across Wilshire Boulevard between buildings ranges from about 80 feet on the eastern side of Reach 3 to about 120 feet on the western side. The distance from the sides of the station excavations to the existing building foundations will probably range from less than 10 feet to about 20 feet.

Section 8.5 presents general guidelines for underpinning.

Although underpinning may seem like the logical approach for the safety of structures in this reach, the nature of particular building structures may mitigate against underpinning in favor of protection schemes. Therefore, protection methods should be thoroughly investigated.

## 8.9 REACH 4 - CENTRAL WILSHIRE BOULEVARD

### 8.9.1 General

Reach 4 extends along Wilshire Boulevard from Normandie Avenue west some 2.5 miles to La Brea Avenue. The area is moderately to densely developed with small to large commercial and residential structures on the order of one to four stories but with some taller buildings over 20 stories. Much of the area adjacent to Wilshire Boulevard consists of paved parking lots with some extended areas of open space.

The ground conditions through this reach consist of 60 to 70 feet of Old Alluvium ($A_4$) overlying 15 to 25 feet of San Pedro sand (SP). Below the San Pedro sand, the borings encountered siltstones and claystones of the Puente and/or Fernando Formations (C) at depths ranging from about 65 to 90 feet. The Old Alluvium consists primarily of silty sands, sandy clays, clayey sands, and silty clays with some zones of clean sand. The soils generally are dense and stiff, based on standard penetration resistances ranging from 30 to 50 blows per foot. Locally, some deposits appear to be medium and firm, based on a lower standard penetration resistance. (In Boring CEG 15 at a depth of 22 feet, the SPT resistance was only 11 blows per foot in a sand, and in Boring CEG 17 at a depth of 22 feet the SPT resistance was only 10 blows per foot in a clay.) The San Pedro sand consists of dense, clean to silty, fine to medium sands. Through Reach 4, sands did not appear to contain oil or tar. The underlying Puente and/or Fernando Formations consist of claystones and siltstones with a few thin, hard sandstone beds.

The permanent ground water table occurs within the Puente and/or Fernando formations at depths in excess of 150 feet. A perched ground water table occurs above the Puente and/or Fernando formations within the Old Alluvium at depths of about 15 to 30 feet. Due to the interbedding of fine- and coarse-grained sediments, perched ground water levels may occur within the alluvium unit. The perched ground water table shown on Drawings 2 and 7 is a simplification based on the limited field data.

### 8.9.2 Excavation Conditions

Above the ground water table, the Old Alluvium can be readily excavated with conventional earthmoving equipment. For estimating purposes, 1H:1V temporary cut slopes can be assumed, provided there is no significant seepage.

Below the perched ground water table (at depths ranging from about 15 to 30 feet), seepage in the more permeable zones should be anticipated. For estimating purposes, it would be appropriate to assume that material handling and vehicle mobility would be more difficult and that the dewatering requirements presented below should be implemented.

Excavation conditions within the Puente and/or Fernando formations are considered to be similar to those in other reaches.

### 8.9.3 Dewatering Requirements

Based on the available information, it is anticipated that inflow rates may be small, as the majority of the water-bearing zones appear to be silty sands and/or fine sands. Dewatering in these types of materials can be accomplished by open pumping or deep sumps within the excavation. Zones of more permeable soils may be encountered, causing piping and loss of ground unless the zone is dewatered prior to excavation.

As a result of the interbedding of granular and fine-grained soils within the Old Alluvium and the nature of the perched ground water in this reach, it may be difficult to effectively dewater these deposits. To be effective, a suitable system might require a relatively close spacing of low-capacity wells or wellpoints to intercept sufficient water-bearing zones. The underlying San Pedro sands appear to be a continuous hydrologic unit and can probably be effectively dewatered with deep wells or wellpoints.

As the proposed station excavation will probably extend into the Puente and/or Fernando formations, slurry cut-off walls would be an alternative to dewatering. Slurry walls would probably not be cost effective at these station depths if based on dewatering considerations only.

Ground water inflows in the Puente and/or Fernando formations are anticipated to be minor.

For current design purposes, it should be assumed that the Old Alluvium can be dewatered by open pumping from within the excavation, while the San Pedro sand will require dewatering prior to excavation.

### 8.9.4 Shoring Requirements

General shoring requirements are discussed in Sections 8.2 and 8.3. Section 9.0 presents design criteria. Through Reach 4 shoring will be required if stations are constructed by cut-and-cover methods at the Western Avenue Station and La Brea Avenue Station. Shoring will also be required if a station is located at Crenshaw Boulevard. Assuming deep stations, the shoring system will be supporting about 80 feet of stiff/dense soil and 30 feet or more of soft rock, for a total depth possibly in excess of 110 feet. For current design purposes, it is recommended that tieback excavations in this reach be limited to 125 feet and strutted excavations to 150 feet. The ability to construct such deep excavations would have to be evaluated in detail during the design phase. The Century City Project discussed in Section 8.4.2 involved a cut within stiff/dense sands, and clays similar to the Old Alluvium anticipated to be encountered through Reach 4.

Specific shoring considerations through this reach include:

° Lagging or equivalent support will probably be required through the granular zone within the Old Alluvium and the San Pedro sands.

° It may be prudent to place filter fabric behind the lagging to minimize potential for loss of ground in granular zones below the water table.

° As a result of the depth of the anticipated excavation, design and construction procedures should be implemented to minimize ground movement as discussed in Section 8.3.

° The dewatering requirements presented in Section 8.9.3 should be implemented.

### 8.9.5 Potential for Ground Movement

General guidelines for movements associated with shored excavations are presented in Section 8.5. If deep excavation depths are considered, as anticipated, we recommend that the ground movement estimated by guidelines in Section 8.5 be increased by 50 percent.

The excavation will dewater the Old Alluvium and San Pedro sands for a considerable distance around the excavation. This will increase the effective stress in the soils and tend to cause some compression. These settlements should not be detrimental, although the borings did encounter some clays that may be moderately compressible.

### 8.9.6 Protection/Underpinning of Existing Structures

The distance across Wilshire Boulevard between buildings is generally about 120 feet, except adjacent to open areas where the distance can be several hundred feet. The distance from the sides of the station excavation to the existing building foundations may be as small as 20 feet.

For deep excavations, with some 80 feet being in soil, it is recommended that a conservative underpinning criterion be developed for current design purposes. It is recommended that the anticipated ground movement, estimated by the guidelines presented in Section 8.5, be increased by 50 percent. Based on these estimates, in conjunction with the foundation design of the existing buildings and the depth and width of the proposed station excavation, the need for underpinning can be assessed.

To simplify installation and to minimize the need for underpinning, consideration should be given to locating the stations immediately adjacent to the building line on one side of the street. This will allow, in some cases, for the soldier piles to be utilized as both temporary lateral support members and permament underpinning piles. This location will also maximize the distance from the other side of the excavation to the adjacent line of buildings, and possibly reduce the need to underpin on that side.

## 8.10 REACH 5 - LA BREA AREA

### 8.10.1 General

Reach 5 extends along Wilshire Boulevard from La Brea Avenue west about one mile to Fairfax Avenue, then northward along Fairfax Avenue about 1.4 miles to Melrose Avenue. The Wilshire Boulevard area is moderately to densely developed, with small to large commercial and institutional buildings on the order of one to four stories, but some larger buildings are over 20 stories. Much of the area adjacent to Wilshire Boulevard consists of paved parking lots with some open space areas. Northward toward and along Fairfax Avenue, the area is residential and commercial, with structures generally less than four stories and many single-family residences.

The current design concept includes stations at Fairfax Avenue and Beverly Boulevard, and possibly some cut-and-cover line segments. Based on the design recommendations presented in Section 7.0, the line segments and stations will probably be relatively shallow at a depth of about 60 feet. The alignment would generally be within the Old Alluvium ($A_4$), except in the vicinity of La Brea Tar Pits, where it would be within the San Pedro sands (SP). An alternative concept would be a deep grade below the San Pedro sands at depths of about 100 to 180 feet.

Ground conditions through this reach consist of Old Alluvium and San Pedro sands underlain by the Fernando Formation (C). The surface of the Fernando Formation drops from a depth of about 80 feet at the eastern end of Reach 5 to an estimated depth of 300 feet near Boring CEG 23A. The thickness of both the Old Alluvium and San Pedro sand increases. The area north of Boring CEG 22 is overlain by Young Alluvium ($A_2$) that thickens toward the north. Drawings 2 and 8 show general subsurface profiles through this reach. Composition and general properties of geologic units as related to surface excavations include:

- Young and Old Alluvium
  The Young and Old Alluvium consist primarily of silty sands, sandy clays, and silty clays with zones of sand. The soils are dense to very dense and locally contain zones of tar. Boring CEG 20 at the proposed location of the Fairfax Avenue Station encountered 20 feet of medium silty sands and firm sandy clays.

- <u>San Pedro sands</u>
  The borings through this reach, except for Boring CEG 23A, encountered San Pedro sands impregnated with oil and tar, commonly referred to as "tar sands." The engineering behavior of tar sands is not fully understood, although some building excavations in the Los Angeles area have penetrated them. Locally, the San Pedro sands are interbeded with claystone and siltstone. Based on past experience and currently published work on the Athabasca Oil Sands of Canada (Byrne, 1980; Brooker, 1980), the following general properties of tar sands are inferred:

  a. Successful 30- to 40-foot excavations have been made into the tar sands with conventional engineering practices, with provisions made for seepage of oil and gas into the excavation.

  b. Based on the laboratory tests, the behavior of tar sands is similar to that of a weakly cemented sand with some cohesion. This cohesion decreases with increasing temperature.

  c. As a result of the high viscosity of the tar/oil, the permeability of the tar sands can be low, comparable to a clayey silt. This results in the tar sands behaving as an undrained material under short-term loading conditions.

  d. The oil and tar may play only a minor role in the mechanical behavior of the tar sands, other than its effect on permeability and providing a cohesion. Strength tests performed on similar samples both with and without oil/tar impregnation yielded similar results (CDA, 1968; Barnes, 1980).

  e. The presence of gas within the tar sands may have a significant effect on its mechanical behavior. During unloading (such as caused by an excavation), the gas tends to expand and can significantly decrease the effective stress on the sand while causing expansion of the sand. During rapid unloading of a tar sand with high gas content, the effective stress can drop to near zero, and the material can lose most of its strength while undergoing a large expansion. Large expansion was encountered in samples obtained during drilling in the tar sands in Boring CEG 23.

The above comments on tar sands are based on limited data. Depending upon the structure type and location selected, it is recommended the mechanical behavior of tar sands be studied in greater detail in the future. However, for the purpose of current design, it is suggested that the following properties of the tar sands be assumed:

a. Tar sands with minor gas content will probably behave as a dense sand with some cohesion and low permeability.

b. Tar sands with significant gas content may, under rapid unloading, lose much of their strength and swell. It may be feasible to minimize the adverse effects of the gas by excavating slowly and/or by installing drains to reduce gas pressure prior to excavating.

- Fernando Formation
  The borings encountered claystone and siltstone.

Based on data collected for this study in the winter of 1980-81, it is believed that the permanent ground water table is relatively deep, at depths probably exceeding 100 feet. A shallow perched water table occurs within the alluvium deposits at depths of about 10 to 25 feet. Because of the interbedding of fine- and coarse-grained sediments, various perched ground water levels may occur within the more permeable alluvium layers. The perched ground water table shown on Drawings 2 and 8 is a simplification based on the available field data.

### 8.10.2 Excavation Conditions

Excavation conditions within the alluvium are anticipated to be similar to those in Reach 4, as discussed in Section 8.9.2.

Conditions within the underlying tar sands may change, depending upon the amount of gas and the construction procedures. Areas that contain significant gas and are excavated rapidly may become very soft and require excavation with a clam shell or large backhoe. In those areas, stable construction slopes within the tar sands may be quite flat, possibly no steeper than 3H:1V or 4H:1V. In areas containing limited gas and/or in areas excavated slowly, the tar sands can probably be excavated without major difficulty. Provisions will have to be made for the collection and disposal of the oil and gas that will be encountered during construction.

Excavation conditions within the Fernando Formation are considered to be similar to those in other reaches.

### 8.10.3 Dewatering Requirements

Based on the available information, it is believed that seepage within the alluvium below the perched water table will be limited to sandy zones and/or lenses. As the soils encountered in the borings were generally fine-grained, the inflows should not be large, although local sandy or gravelly zones may result in moderate seepage. It is recommended, for purposes of current design, dewatering can be accomplished with open pumping from within the excavation.

The high viscosity of the oil would indicate that the permeability of the tar sands is low. It is not anticipated that there will be significant seepage or dewatering problems within the tar sands. Locally, such as in Boring CEG 23A, the San Pedro sands may not be impregnated with oil. In these areas, dewatering requirements may be similar to those discussed in Section 8.9.3.

### 8.10.4 Shoring Requirements

Through Reach 5, shoring will be required at the Fairfax Avenue Station and the Beverly Boulevard Station. Assuming a shallow to intermediate station depth (on the order of 60 feet deep), most of the excavation will be in the alluvium containing the oil sands. Shoring requirements in the alluvium are anticipated to be similar to those discussed in Sections 8.8.4 and 8.9.4.

A 60-foot-deep station at Fairfax Avenue will penetrate some 20 feet into the tar sands. This depth could increase toward the east. Boring CEG 19, some 2,000 feet to the east, encountered tar sand at a depth of 14 feet. The Mutual Benefit Life Building (CDA, 1968) which involved a 40-foot excavation including 10 feet into the tar sands, is located some 500 feet east of the proposed Fairfax Avenue Station. Conventional soldier piles with lagging were successfully utilized to shore the excavation. Depending upon the structure type and location selected, additional subsurface information and testing and modeling the behavior of tar sands may be required during later designs to evaluate effects, such as the depth of penetration into tar sand, the amount of gas in the tar sands, temperature, shoring design and construction procedures and others. For current design purposes, it is recommended that:

- Condition of Minimum Penetration into Tar Sands
  For excavations that penetrate less than 20 feet into the tar sands, conventional shoring design and construction practices can be assumed to be appropriate. Increasing the earth pressures (provided in Section 9.0) for alluvium sands by 25 percent is recommended for current design purposes.

- Condition of Deep Penetration into Tar Sands
  For excavations that penetrate more than 20 feet into the tar sands, the following design criteria are suggested:

  a. In computing strut and wall loads, assume a uniform earth pressure within the tar sand which is equal to 35H, where H is the total depth of cut.

  b. Avoid the use of tiebacks, i.e., use internal bracing. If tiebacks are evaluated, assume that anchors are set beyond a no-load zone defined by a horizontal line extending from the bottom of the excavation horizontally a distance of H/2 (where H is the depth of the cut) and then upward at an angle of 60° with the horizontal. The intent of this extended no-load zone is to account for potential loss of strength of the tar sands near the excavation as a result of the effects of unloading and gas pressure.

  c. In computing the required depth of penetration of soldier piles to resist both lateral and vertical loads, the resistance provided by the upper 15 feet of penetration should be ignored.

### 8.10.5 Potential for Ground Movement

General guidelines for ground movements associated with shored excavations are presented in Section 8.5. For excavations extending into the tar sands, it is recommended for the current design the anticipated ground movements estimated by these guidelines be increased by 100 percent for up to 20 feet of penetration into the tar sands and by 200 percent for deeper penetrations.

Although ground swelling, or heaving of the bottom of excavation, is a phenomenon that occurs to some degree in all excavations, rapid unloading of tar sands containing a significant amount of gas could result in significant ground swelling. In addition, depressurization of the oil and gas adjacent to the excavation may cause the swelling to extend to other areas adjacent to the excavation. Although this problem has not been encountered on previous local projects, deep excavations into tar sands and/or into areas with very high gas content could result in some ground heave. There is insufficient current information to predict the magnitude and distribution of ground heave and the reversal settlement under load.

8.10.6 Considerations Related to Oil and Gas

Provisions will have to be made to collect and dispose of any oil that seeps into the excavation during construction. Design considerations involve collection systems for oil and gas, and provisions for corrosion protection of exposed subgrade elements.

8.10.7 Protection/Underpinning of Existing Structures

At the proposed location of the Fairfax Avenue Station, the distance across Wilshire Boulevard between buildings is about 120 feet. At the proposed location of the Beverly Boulevard Station, there is about 80 feet across Fairfax Avenue between buildings. On the east side of Fairfax in this area, however, there currently is a large parking lot that extends for a distance of some 900 feet along Fairfax. By locating the station in this area toward the east side of Fairfax, it may be possible to minimize the need for underpinning.

Section 8.5 presents general guidelines to evaluate underpinning requirements. At the proposed Fairfax Avenue Station, where the excavation extends into the tar sands, the estimated ground movements should be increased, as discussed above in Section 8.10.5. Based on these recommendations, it is believed that underpinning will probably be required at the Fairfax Avenue Station, particulary for excavations in excess of about 60 feet. For excavations less than 60 feet in depth, protection methods, versus underpinning, should be thoroughly investigated.

The general comments on underpinning presented in Section 8.5 are applicable through this section. Conventionally, piles in this area of Los Angeles have been designed to obtain support entirely from the soils below the tar sands. If underpinning is required in areas where the tar sands extend to considerable depths, it may be feasible to support the underpinning piles within the tar sands. This would require additional study and concurrence from the City of Los Angeles Engineering Department. For excavations with less than about 20 feet of penetration into the tar sands, it may be feasible to locate the excavation immediately adjacent to the building line on one side of the street, thus minimizing the need to underpin the buildings on the opposite side. This would also allow, in some cases, for soldier piles to be utilized as both the temporary lateral support members and permanent underpinning piles. For deep excavations into the tar sands, it is believed that the estimated movements will be unsatisfactory, and underpinning on both sides of Wilshire Boulevard probably will be required.

## 8.11 REACH 6 - HOLLYWOOD AREA

### 8.11.1 General

Reach 6 extends north along Fairfax Avenue to Fountain Avenue, then east along Fountain Avenue to Cahuenga Boulevard, and north along Cahuenga Boulevard to Yucca Street at the base of the Santa Monica Mountains. Through this reach along Fairfax and Fountain, the area is primarily residential, with some small commercial buildings and parking areas. Along Cahuenga, the area includes primarily large commercial buildings, generally one to four stories with some up to about 10 stories. Much of the area adjacent to Cahuenga consists of paved parking lots.

The current design concept includes stations at Santa Monica Boulevard, La Brea Avenue (possible future station) and Hollywood Boulevard. Several line segment and station depths appear feasible, ranging from shallow (on the order of 40 feet), allowing for cut-and-cover line segment construction, to deep (in excess of 70 feet) within the Old Alluvium. A shallow cut-and-cover line segment section approaching Reach 7, which crosses the Santa Monica Mountains, would allow daylighting of the grade to avoid crossing the active Hollywood fault at depth.

Ground conditions consist of thick deposits of Young ($A_2$) and Old Alluvium ($A_4$) in excess of 200 feet thick. The 60- to 70-foot thickness of Young Alluvium is based on an interpretation of the boring logs and regional geologic sequence. The deposits exhibit considerable variation both laterally and vertically, but consist primarily of clayey sandy and sandy clays with 10- to 20-foot zones of silty sand, sandy silt and clean sand. Boring CEG 26 at a depth of 114 feet encountered a 50-foot zone of silty clay while Boring CEG 25 encountered primarily silty sand and sandy silt. In Boring CEG 28 some hard cemented zones were encountered within a sand zone at a depth of 86 feet. Below a depth of about 20 to 40 feet, standard penetration resistances ranged from 40 to 100 blows per foot. The alluvium below this depth is believed to be predominately dense to very dense, and stiff to hard. In the upper 20 to 40 feet, however, the alluvium appeared to be medium, with some loose to soft zones based on standard penetration resistances ranging from 7 to 30 blows per foot.

Ground water was encountered at depths of 80 to 100 feet in Borings CEG 24 and CEG 25, at depths of 25 to 30 feet in Borings CEG 26 and CEG 27, and at a depth of 75 feet in Boring CEG 28. These depths represent, we believe, the permanent ground water levels at the time the measurements were made in the winter of 1980-1981.

### 8.11.2 Excavation Conditions

Excavation conditions in the alluvium are anticipated to be similar to those in the alluvium in Reaches 4 and 5. The ground water table appears to be in excess of 70 feet deep in Borings CEG 24, 25 and 28; therefore, the excavation may be predominately dry along much of Reach 6.

Temporary cut slopes within the softer, looser soils encountered within the upper 20 to 40 feet can be assumed to be 1 1/2H:1V. Granular soils within this looser upper zone which encountered ground water seepage may be particularly susceptible to piping/flowing. Hard cemented zones within the Old Alluvium sands (such as in Boring CEG 28 at a depth of 86 feet) may be encountered.

### 8.11.3 Dewatering Requirements

It is believed that significant seepage within the alluvium, below permanent or perched ground water levels, will be limited to sandy zones and/or lenses. As the soils encountered in the borings were generally finer-grained, the inflows should not be large, although local sandy or gravelly zones may result in moderate seepage. To a depth of 70 to 80 feet, excavations for the Santa Monica and Hollywood Stations may be essentially dry.

It is recommended that dewatering can be accomplished with open pumping from within the excavation. Locally, such as in the vicinity of Boring CEG 27, which encountered some 20 feet of clean sands and gravelly sands below the observed ground water table, some pre-excavation pumping from wells, deep sumps, or well points may be required.

### 8.11.4 Shoring Requirements

Through Reach 6, shoring will be required at the Santa Monica, La Brea (possible future station) and Hollywood stations and for any cut-and-cover line segments. The shoring requirements are anticipated to be similar to those discussed for alluvium in Sections 8.8.4 and 8.9.4. Within the upper 20- to 40-foot zone of softer/looser soils, the design earth pressures provided in Section 9.0 should be increased by 25 percent.

### 8.11.5 Potential for Ground Movement

As a result of the softer/looser upper 20- to 40-foot zone through this reach, we recommend that the anticipated ground movements estimated by the guidelines as presented in Section 8.5 be increased by 100 percent.

Although not believed to be a major concern, construction vibrations could induce densification of the medium sandy zones within the upper 20 to 40 feet.

### 8.11.6 Protection/Underpinning of Existing Structures

At the proposed location of the Santa Monica Station, the distance across Fairfax Avenue between buildings is about 100 feet, with several parking lot areas where the distance exceeds 150 feet. At the proposed Hollywood Station, the distance across Cahuenga Boulevard between buildings is about 80 feet, except adjacent to parking lot areas, where it exceeds 200 feet.

It is believed underpinning for commercial buildings adjacent to the stations will probably be required, or in lieu thereof, protection methods could be considered. For a cut-and-cover line segment, depending on its location, underpinning may not be required, but protection methods might be necessary.

## 8.12 REACH 7 - SANTA MONICA MOUNTAINS

### 8.12.1 General

Reach 7 extends about 3.3 miles across the Santa Monica Mountains and will approximately parallel the Hollywood Freeway. Through this reach, the alignment will be almost entirely within hard rock. In this area, development is light to moderate with single-family residences, small commercial buildings and considerable open space. The existing Hollywood Freeway includes rock cuts up to at least 100 feet through the mountains.

The current design concept includes a deep tunnel through the mountains, with an optional station at the Hollywood Bowl. To avoid crossing the Hollywood fault zone at depth, consideration should be given to a surface line through the southern section of Reach 7. The line would then go underground just north of the fault location and continue in a tunnel through the mountains. Surface excavations for this option would include only tunnel portals.

The ground conditions through this reach consist almost entirely of the Topanga Formation, except for a cover of Young Alluvium up to about 40 feet thick at the northern and southern extremes of the reach. Five fault zones including the Hollywood, Hollywood Bowl, Benedict Canyon, and two unnamed faults appear to cross the proposed alignment. The Topanga Formation consists of a massive sandstone and conglomerate unit with some claystone and a basalt unit. Locally, such as at Boring CEG 29 (where 26 feet of soft claystone was encountered) the upper 10 to 25 feet may be weathered and soil-like in behavior. Throughout this reach, the rocks are steeply dipping in the range of 40° to near-vertical. Sections 5.0 and 7.0 discuss in detail the properties of the Topanga Formation. The Young Alluvium encountered in the borings varied considerably, ranging from silty sands to gravelly cobbles at Boring CEG 29 to sandy clay to clayey sand at Borings CEG 30 and 33. Based on the standard penetration resistances ranging from 8 to 20 blows per foot, these deposits are loose/soft to medium/firm. Boring CEG 33 encountered 38 feet of soft to very soft sandy clays which may, at least in part, be a sliver fill placed in the past to construct Cahuenga Boulevard.

Ground water conditions through this reach are erratic, as the Topanga Formation is essentially nonwater-bearing except for limited quantities of water in joints and fractures. Significant temporary ground water inflows can occur in highly jointed or fractured zones. The alluvial deposits may contain perched ground water. Ground water levels encountered in the borings for the alluvial deposits ranged from about a 20-foot depth at Borings CEG 30 and 33 to in excess of 70 feet at Boring CEG 28A.

8.12.2 Recommendations

Surface excavations in this reach will be in rock, except possibly at the southern and northern ends where the rock surface slopes downward below the alluvial deposits of the Hollywood and North Hollywood basins.

Possible Portal excavations for the south end of the reach, near Boring CEG 29, would be mostly dry, classified as moderately blocky and seamy rock (Terzaghi Rock Condition No. 4), consisting of claystone, sandstone and siltstone of the Topanga Formation covered by a thin wedge of alluvial deposits. The rocks are generally weathered, friable, and closely to moderately fractured. Local highway cuts in this rock stand at slopes of about 1 1/2H:1V if bedding does not daylight unsupported out of cuts, but raveling commonly requires regular maintenance. Portal excavations in the rock and alluvial material will require protection against raveling, erosion and possible material determination. Benching of cuts, gunite slope cover, bolts and wire mesh are possible preventive measures to reduce these problems.

Surface excavation conditions for the middle section of the reach will be different depending upon the rock types encountered and site-specific topographic conditions. Conditions will be similar to those described for tunneling in Section 7.0, although ground water problems are not anticipated. Surface excavations associated with tunneling, such as ventilation shafts, will probably require some strengthening around the opening because rocks are generally more weathered and fractured near the ground surface. Such strengthening could be accomplished by perimeter grouting prior to excavation, or by installation of rock bolts, wire mesh and shotcrete as excavation proceeds. Conventional shaft sinking methods can also be applied with ring beams and lagging.

Cut-and-cover line segment construction along the northern and southern extremes of the reach would involve excavation in Young Alluvium. The alluvium through this reach is among the least dense/stiff along the alignment, ranging from loose/soft to medium/firm. For current design purposes, it is recommended that stable construction slopes be assumed to be 1.5H:1V or flatter and that any shoring be designed for the loose sands or soft clay categories presented in Section 9.0. Dewatering conditions in the alluvium are anticipated to be similar to those in Reach 6, but may locally require pre-excavation pumping in areas of clean granular soil. Because of the lower density, these soils will probably be more permeable than the Old Alluvium in Reach 6. Underpinning, or prescribed protection method, may be required along

Cahuenga Boulevard south along the southern extreme of Reach 7. Shoring and underpinning requirements are anticipated to be similar to those in the Young Alluvium of Reach 6.

## 8.13 REACH 8 - SAN FERNANDO VALLEY

### 8.13.1 General

Reach 8 is about 2.2 miles long and extends north along Vineland Avenue. The area is residential with some commercial buildings, parking lots and open space. The proposed alignment will cross the Ventura Freeway and Los Angeles River.

The current design concept includes the Universal City Station near Boring CEG 34 and the North Hollywood Station at the northern Portal. Based on the design recommendations presented in Section 7.0, the Metro Rail Project alignment may be shallow within about 50 feet of the surface or deep at a depth of about 100 feet. Cut-and-cover line segments would be feasible for the shallow alignment.

The ground condition through this reach consists of about 50 to 70 feet of Young Alluvium ($A_1$) underlain by a thick sequence of Old Alluvium ($A_3$). Borings CEG 34 and 35 encountered sandstones and claystones of the Topanga Formation below a depth of about 50 feet and 120 feet, respectively. Immediately north of Boring CEG 35, as shown on Drawings 2 and 12, the top of the Topanga Formation is interpreted to be at a depth of about 250 feet. The alluvium includes an upper zone some 30 to 40 feet thick that is only firm/medium with standard penetration resistances generally only 10 to 30 blows per foot. Materials in this upper zone consist predominantly of silty sands and sandy silts, but include clean sands, clayey sands, and silty clays. Below the upper zone, the boring soils graded much stiffer/denser materials with standard penetration resistances generally exceeding 50 blows per foot (CEG 35 is an exception where the blow/ft ranged only up to about 30 blows per foot to depths exceeding 100 feet). These lower soils consist primarily of clean and silty sands with zones of clayey sand-sandy clay. A 20- to 60-foot zone of clean sand and gravel with cobbles was generally encountered at depth below about 40 feet in the borings in this reach.

Based on the data collected for this study in the winter of 1980-1981, the ground water table north of the Los Angeles River appears to be deep at depths generally exceeding 100 feet. South of the Los Angeles River, at Boring CEG 34, ground water was encountered at a depth of 15 feet.

### 8.13.2 Excavation Conditions

Excavation conditions within the alluvium north of the Los Angeles River, where the ground water appears to be deep, are anticipated to be similar to Reach 6. Cobbles and boulders should be anticipated within the sand and gravel zones below depths of about 40 feet. Because of lower densities, the soils in the upper 30 to 40 feet will be easier to excavate but may require flatter temporary construction slopes on the order of 1 1/2H:1V.

South of the Los Angeles River at the proposed Universal City Station, Boring CEG 34 encountered 38 feet of silty sands underlain by 12 feet of clean gravelly sands over rock with a shallow ground water table at 15 feet. Excavations below the ground water table will require dewatering. Cobbles and boulders should also be anticipated in the gravelly materials in this area.

### 8.13.3 Dewatering Requirements

In the vicinity of the proposed Universal City Station dewatering will be required because of gravelly sands below 38 feet. Excavations that extend to depths within about 10 feet of the gravelly sands could encounter a quick condition in the excavation unless the ground water level in the sands is lowered. For current design purposes it should be assumed pre-excavation dewatering from wells extending into the gravelly sands will be required. The Young Alluvium, in the vicinity of the Los Angeles River (between Borings CEG 34 and 35), is estimated to have a permeability on the order of $10^{-1}$ cm/sec or more, which is a transmissibility on the order of 100,000 to 200,000 gallons per day per foot. As an alternative, a slurry wall or slurry cut-off trench could be extended into the underlying Topanga Formation. This would effectively seal the excavation and minimize dewatering requirements.

North of the Los Angeles River, the ground water table is deep and dewatering will probably not be required.

### 8.13.4 Shoring Requirements

General shoring requirements are discussed in Sections 8.2 and 8.3. Section 9.0 presents design criteria. Through this reach shoring will probably be required at the Universal City Station, North Hollywood Station and any cut-and-cover line segments. Alternatively, due to the limited development and size of adjacent structures, consideration might be given to unsupported cuts for relatively shallow excavations. Shoring requirements in the vicinity of the Universal City Station, where ground water may be shallow, are anticipated to be similar to Reach 1. North of the Los Angeles River, where ground water appears to be deep, shoring requirements are anticipated to be similar to Reach 6. The relatively loose sands in the upper 30 to 40 feet are judged to be very susceptible to raveling. Lagging, in conjunction with soldier piles, will be required for the majority of the soils through this reach.

### 8.13.5 Potential for Ground Movement

General guidelines for ground movements associated with shored excavations are presented in Section 8.5. Because of the softer/looser soils in the upper 30 to 40 feet, it is recommended that the anticipated ground movements estimated by these guidelines be increased by 100 percent.

Construction vibrations, although unlikely, could induce densification of the soils within the looser upper 30 to 40 feet.

Below the observed ground water table at Boring CEG 34 (Universal City Station), some 15 feet of medium sandy silts and silty sands were encountered. The standard penetration resistance recorded in these materials ranged from about 9 to 20 blows per foot. These soils are considered susceptible to liquefaction and may require densification to minimize liquefaction potential.

### 8.13.6 Protection/Underpinning of Existing Structures

The proposed Universal City Station is located in a residential area with narrow streets and single-family houses or small apartment units. Vineland Avenue is a six-lane road with a wide median strip and some 160 feet across between existing structures. The proposed North Hollywood Station is located in an area of one-story industrial buildings, with considerable open space.

The criteria presented in Section 8.5 for underpinning existing structures are applicable through this reach with the estimated ground movement being increased by 100 percent. Based on the width of Vineland Avenue and the type of structures in the area, it is believed that the protection/underpinning requirements may be minimal through this reach.

# Section 9.0
# Design Considerations

# Section 9.0: Design Considerations

## 9.1 GENERAL

In this section, geotechnical design parameters and criteria are presented consistent with the available information. These parameters and criteria are suitable for current design, evaluating alignment and profile alternatives, and cost estimating.

Based on the above and results of any future geotechnical investigations, the parameters and design criteria presented below may be subject to revision.

## 9.2 SOFT-GROUND DESIGN VALUES

The general engineering characteristics of the geologic units encountered in each reach along the project alignment are discussed in Sections 5.0, 7.0 and 8.0. Table 5-2 in Section 5.1.3 summarizes the results of the laboratory testing program for each major geologic unit. For purposes of current design, Table 9-1 presents recommended design values for each major soft-ground unit anticipated along the alignment. The values shown on Table 9-1 are based in part on the laboratory results and field Standard Penetration Test (SPT) values, while incorporating past experience and practice in the Los Angeles area. As appropriate, comments are presented where different parameters are more suitable for specific areas or reaches.

It is recommended that the design engineer review the information presented in Sections 5.0, 7.0 and 8.0 for additional background. Utilization of the design values presented in Table 9-1 without this additional information may be misleading.

## 9.3 FOUNDATION DESIGN VALUES

In general, soils underlying the proposed station locations will be stiff/dense providing excellent foundation support for the station structures. Possible exceptions include stations founded less than 40 feet deep through Reaches 1, 6 and 8 where the upper alluvium may be only firm/medium and will require either a continuous mat or deep foundation system for heavy concentrated loads. Surface facilities and aerial structures can generally be supported on spread footings with the exception of Reaches 1, 6 and 8 where heavy concentrated loads may require a deep foundation system (i.e., piling).

Artificial fill, including buried basements, may be encountered locally along the alignment (e.g., materials encountered in Boring CEG 7). Although stations in most cases will extend below this zone, surface structures may require piles and/or excavation/replacement in areas of existing fill. The tar sands through Reach 5 present special engineering challenges as discussed

TABLE 9-1 Soft-Ground Design Values

| Geologic Unit[1] | Dry[2] Density (pcf) | Wet[2] Density (pcf) | Undrained Shear[2] Strength - $\frac{Qu}{2}$ for $\phi$ = o Analysis (psi) | Effective[2] Stress Parameters $\phi'$ (deg) | c' (psi) |
|---|---|---|---|---|---|
| Young Granular Alluvium ($A_1$) | 110 | 128 | -- | 34 | 0 |
| Young Fine-grained Alluvium ($A_2$) | 102 | 124 | 20 | 30 | 2 |
| Old Granular Alluvium ($A_3$) | 112 | 130 | -- | 38 | 0 |
| Old Fine-grained Alluvium ($A_4$) | 104 | 128 | 25 | 30 | 4 |
| San Pedro Sands | | | | | |
| • Without Significant Tar (SP) | 105 | 125 | -- | 38 | 0 |
| • Tar Sands ($SP_p$)[3] | 100 | 110 | 2 | 34 | 0.5 |
| Puente & Fernando Formations (C) | | | | | |
| • Siltstone/Claystone | 95 | 120 | 42 | 32 | 6 |
| • Sandstone | 105 | 125 | 30 | 38 | 2 |

[1]See report Sections 5.0, 7.0 and 8.0 for detailed information.

[2]See Table 5-2 in Section 5.1.3 for summary of all laboratory test results.

[3]Cohesion (both $\frac{Qu}{2}$ and c') sensitive to temperature. For example, the undrained shear anticipated to decrease to less than one half its original value when heated from 70° to 100°F.

| LEGEND | |
|---|---|
| pcf | = pounds per cubic foot |
| psi | = pounds per square inch |
| deg | = degrees |
| $\phi'$ | = angle of internal friction |
| c' | = cohesion |

in Sections 8.10 and 10.3. In particular, the Fairfax Avenue Station may be founded on the tar sands. Current practice in Los Angeles precludes supporting foundation loads within tar sands. However, it is reasonable to assume that it is feasible for the Fairfax and Beverly stations to be founded on the tar sands if future detailed studies of the final structure type and location are performed. However, for current design purposes, recommendations assume that foundation bearing within the Fernando Formation below the tar sands will be required.

Table 9-2 presents foundation recommendations for each major soft-ground geologic unit. General comments are made on Table 9-2 concerning the support characteristics of each geologic unit as related to a deep foundation system (i.e., driven piles, drilled piers, etc.). It is judged, due to vibration considerations and local area practice, that drilled piles may be the most suitable.

TABLE 9-2 Foundation Design Values

| Geologic Unit[1] | Allowable[2] Maximum Bearing (psf) | Coefficient of Base Friction | Allowable[3] Passive Pressure (pcf) | Deep Foundation Support Characteristics |
|---|---|---|---|---|
| Young Alluvium ($A_1$, $A_2$) | | | | |
| • Reaches 1, 6 and 8 | 3,000 | 0.4 | 250 | Unsuitable |
| • Reaches 2, 3, 4, 5 and 7 | 5,000 | 0.4 | 350 | Probably suitable |
| Old Alluvium ($A_3$, $A_4$) | 6,000 | 0.4 | 350 | Suitable/Good |
| San Pedro sands | | | | |
| • Without Significant Tar (SP) | 8,000 | 0.45 | 350 | Good |
| • Tar Sands ($SP_p$) | 3,000 | 0.25 | 150 | Probably unsuitable |
| Puente and Fernando Formations (C) | 10,000 | 0.45 | 450 | Excellent |

[1]See report Sections 5.0, 7.0 and 8.0 for detailed information.

[2]Allowable maximum bearing represents estimated average values; very large footings may require lower values due to excessive settlement potential.

[3]Equivalent fluid density.

| LEGEND |
|---|
| psf = pounds per square foot |
| pcf = pounds per cubic foot |

## 9.4 UNDERGROUND STRUCTURES CONSTRUCTED IN SURFACE EXCAVATIONS

### 9.4.1 General

This section presents design criteria for underground structures constructed in surface excavations including stations, cut-and-cover line segments and shafts. In addition to criteria for permanent structures, general criteria for temporary shoring systems are provided.

### 9.4.2 Temporary Shoring

As discussed in Section 8.3, there are numerous available shoring systems that could be utilized with a wide assortment of design and construction options. In addition, depending on protection and underpinning criteria and proximity of adjacent buildings, it is anticipated that different amounts of ground movement may be acceptable along the alignment. Thus, the design criteria should be a function of, but not necessarily limited to: the subsurface material conditions, groundwater conditions, surcharge from adjacent structures, the shoring scheme and acceptable level of ground movement. Based on available project information and past experience and judgment, the following criteria for a shoring system are recommended:

- Design Pressures
  Figure 9-1 presents the recommended earth pressures on the temporary shoring without surcharge loads for computing strut loads, tieback loads and required depth of shoring embedment. The design pressures for the case of supporting several geologic units can be determined using judgment and taking weighted averages. Figure 9-2 shows a general procedure for computing the increased wall pressure developed by adjacent structures without underpinning. To allow for construction surcharge loads, the level of applied loading should be increased by two feet.

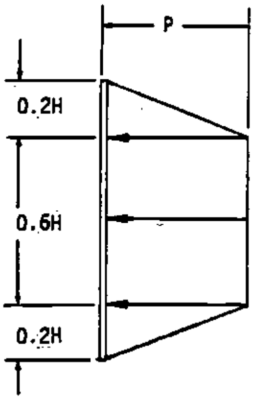
- Embedment
  The required shoring wall embedment cannot be generalized. It should be based on specific conditions at each site developing sufficient vertical (particularly in the case of tieback walls) and horizontal capacity to resist the computed loads.

- Strut Considerations
  All struts should be prestressed to a minimum of 50 percent, and, if practical, to 75 percent of the computed design load to minimize earth movements.

- Tieback Considerations
  All tiebacks should be prestressed to at least 80 percent of the computed design loads to minimize earth movements. Typically, the tieback anchors are set beyond a no-load zone defined by a line extending upward from the bottom of the excavation at an angle of 60 degrees from the horizontal. For excavations in excess of 75 feet, it is recommended that the no-load zone be extended to lie beyond a zone defined by a horizontal line extending from the bottom of the excavation horizontally a distance of H/3 (where H is the depth of the cut) and then upward at 60 degrees with the horizontal. This extended no-load zone is intended to minimize problems associated with release of "locked-in" lateral stresses and the plane strain condition associated with long stations and cut-and-cover line excavations.

  The current practice in the Los Angeles area, which has been field-tested numerous times, is to utilize large-diameter/low grout pressure anchors for tieback systems. Since there is little field experience in the Los Angeles area for small-diameter/high grout pressure anchors, it is recommended for current design that proven area anchor designs be utililized. The required anchor length, for large-diameter/low grout pressure anchors, can be based on 1 kip per square foot of anchor surface area for anchors in stiff fine-grained alluvium and 2 kips per square foot of surface area in dense granular alluvium and soft rock. At this stage, anchors should not be used in fine-grained Young Alluvium in Reaches 6 and 8 or in fill.

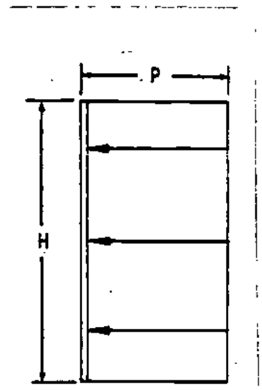
- Bottom Heave in Fine-Grained Soils
  The factor of safety against bottom heave should be evaluated for deep excavations in fine-grained soils. According to Terzaghi and Peck (1968), the factor of safety (FS) can be estimated from:




| | Geologic Unit | P* (psf) |
|---|---|---|
| a. | Young Alluvium ($A_2$: Reaches 6 and 8) | 34H |
| b. | Young Alluvium ($A_2$: Reaches 1, 2, 3, 4, 5 and 7) and Old Alluvium ($A_4$) | 24H |
| c. | Puente and Fernando Formations (C) | 18H |

Where: H is maximum depth of excavation, in feet.

EARTH PRESSURE ON TEMPORARY BRACED SHORING IN FINE-GRAINED SOFT-GROUND




| | Geologic Unit | P* (psf) |
|---|---|---|
| a. | Young Alluvium ($A_1$: Reaches 1, 2 and 8) and Artificial Fill | 26H |
| b. | Young Alluvium ($A_1$: Reaches 3, 4, 5 and 7) and Old Alluvium ($A_4$) | 20H |

Where: H is maximum depth of excavation, in feet.

EARTH PRESSURE ON TEMPORARY BRACED SHORING IN GRANULAR SOIL

*Assumes no groundwater pressure, a level backslope and no surcharge loads.

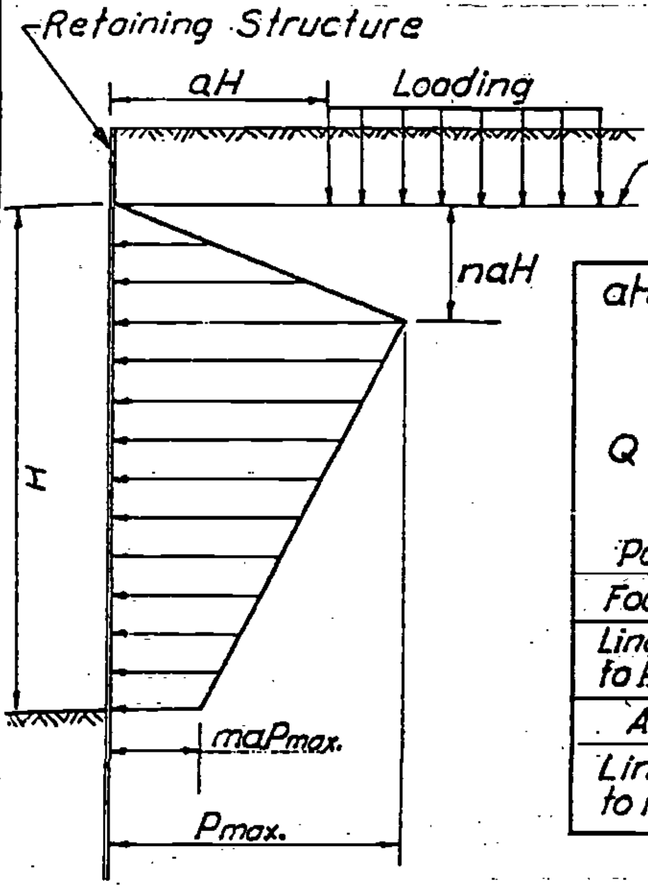
RECOMMENDED DESIGN WALL PRESSURES

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 9-1




aH = distance from retaining structure to footing load or parallel line load or to leading edge of areal load or perpendicular line load.
Q = total load per ft. of length parallel to retaining str., or total footing load.

| Parameters | n | m | Pmax. |
|---|---|---|---|
| Footing Load | 0.6 | 0.4 | $(2.1-1.8a)Q/H^2$ |
| Line Load Perp. to Retaining Structure | 0.6 | 0.4 | $(1.4-1.2a)Q/H^2$ |
| Areal Load | 0.4 | 0.25 | $(0.8-0.5a)Q/H$ |
| Line Load Parallel to Retaining Structure | 0.4 | 0.25 | $(1.1-0.5a)Q/H$ |

(After Mueser and others, 1967)

LATERAL PRESSURE ON TEMPORARY BRACED AND PERMANENT WALLS
DUE TO ADJACENT BUILDINGS NOT UNDERPINNED

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 80-1280

CWDD/ESA/GRC Geotechnical Consultants

Figure No. 9-2

$$FS = \frac{c\ N_c}{\gamma H}$$

where: FS = factor of safety
c = undrained shear strength (psf)
$\gamma$ = unit weight of soil (psf)
H = depth of excavation (ft)
$N_c$ = bearing capacity factor depending on friction angle and geometry; typical values for anticipated excavations would be 6 to 7.

To limit ground movement, it is judged that the factor of safety should exceed 1.5 since large strains occur as the soil approaches failure. Based on the design values presented in Table 9-1, the recommended limiting excavation depths are as follows:

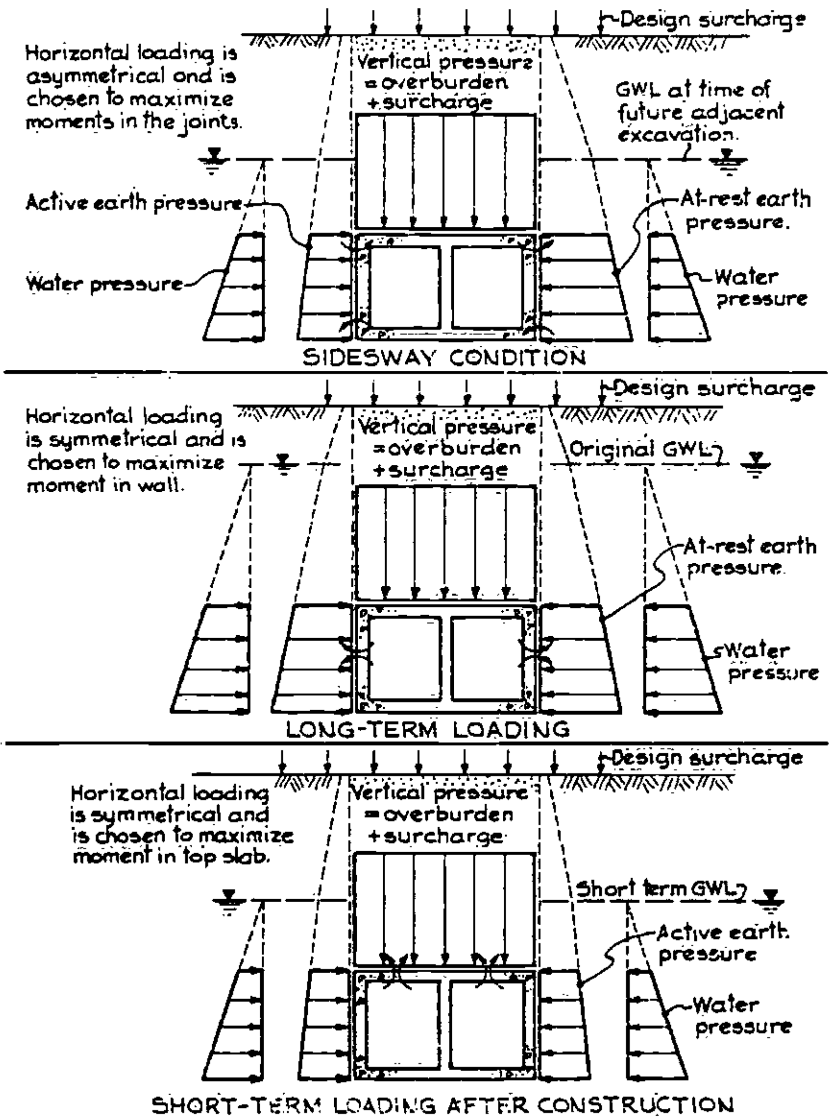
| Geologic Unit | Limiting Excavation Depth (ft) |
|---|---|
| Fine-Grained Alluvium ($A_2$ and $A_4$) | 110 |
| Puente and Fernando Formations (C) | 175 |

It is recommended that the limiting depth be determined at each location based on the average strength values and judgment. Bottom heave in granular soils is generally not a constraint unless excessive seepage pressure occurs.

### 9.4.3 Pressures on Permanent Underground Structures

The earth pressure on permanent underground structures is not well understood. Factors such as behavior of the shoring system during construction phase, stiffness of permanent wall, long-term creep and vibration effects and others are difficult to incorporate into earth pressure predictions. Current practice is to assume, with time, that the pressure on permanent walls tends toward the at-rest ($K_o$) condition. In heavily overconsolidated materials (such as the Puente and Fernando formations) where the initial $K_o$ may exceed 1.0 (i.e., the horizontal stress exceeds the vertical stress), the general practice has been to assume that stress relaxation occurs during excavation and is not re-established. Observations have not been reported (Gould, 1970). Current design should be based on the general criteria and loading conditions presented in Figure 9-3.

Surcharge loads on walls from adjacent structures not underpinned should be incorporated as presented on Figure 9-2. Distributed pressures from adjacent structures on the underground station roof can be computed based on elastic solutions (such as Lambe and Whitman, 1969, p. 103). Generally, the effects of adjacent structures on roof loads should be negligible.




(Gould, 1970)

Recommended Earth Pressure Coefficients

| Geologic Unit | Active Coefficient ($K_a$) | At-Rest Coefficient ($K_o$) |
|---|---|---|
| ° Alluvium ($A_1$, $A_2$, $A_3$ and $A_4$) | 0.3 | 0.5 |
| ° Puente and Fernando Formations (C) | 0.2 | 0.4 |
| ° San Pedro sands | | |
| Without significant tar (SP) | 0.3 | 0.5 |
| Tar sands ($SP_p$) | 0.4 | 0.8 |

PRESSURES ON PERMANENT UNDERGROUND STRUCTURES

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Figure No. 9-3

Effects of earthquake loading on permanent underground structures will be addressed in a separate report.

### 9.4.4 Shafts

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

- Fine-Grained Alluvium, Tar Sands, and Claystone ($A_2$, $A_4$, $SP_p$ and C)
  Radial stresses can be assumed equal to the at-rest pressure based on effective stress plus the hydrostatic pressure. Thus,

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

where: $\sigma_r$ = estimated radial stress
$K_o$ = at-rest earth pressure coefficient

- $A_2$, $A_4$ — $K_o = 0.5$
- Tar Sands — $K_o = 0.8$
- Claystone — $K_o = 0.4$

$\sigma_s'$ = effective vertical earth stress at designated location
$\mu$ = anticipated ground water pressure at designated location

- Granular Alluvium, San Pedro sands and and Siltstone/Sandstone ($A_1$, $A_3$, SP, and C)
  Theoretical analyses based on methods developed by Terzaghi (1943) and Szechy (1970) indicate the radial effective stress on shafts in granular soils is nearly equal to the active pressure at shallow depths but approaches a constant pressure at great depths. Radial stresses on shafts can be estimated as:

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

where: $\sigma_r$ = estimated radial stress
$K_a$ = active earth pressure

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

$\sigma_s'$ = effective vertical earth stress at designated location
$\mu$ = anticipated ground water pressure at designated location
R = Reduction Factor based on ratio of depth (z) to shaft diameter (D) where (after Mueser, and others, 1967):

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

## 9.5 UNDERGROUND STRUCTURES CONSTRUCTED IN MINED EXCAVATIONS

### 9.5.1 General

This section presents design criteria for underground structures constructed in mined excavations. This would include bored tunnels and mined stations.

### 9.5.2 Tunnels in Soft-Ground Units

The forces that will develop on the lining of tunnels in the soft-ground units is a function of many complex factors. Various methods have been proposed to compute the theoretical loading, taking into account arching (Terzaghi, 1946), earthpressure coefficients (Hewell and Johannesson, 1922) and soil structure deformation characteristics (Szechy, 1970). These computational approaches have serious shortcomings, including (partially after Golder, 1976):

- Methods of assessing the loads are based on highly simplified and sometimes arbitrary assumptions.
- None of the methods considers the actual behavior of the soil or rock during the tunneling process, nor the method or time of installation of supports.
- Standard practice of designing both the initial (temporary) support and final (permanent) lining for the full load can be overly conservative since, in many cases, very limited loads, if any, will develop on the permanent lining.

It is recommended that the procedures presented by Peck (1969) and Deere and others (1969) be adapted for current design purposes to estimate the forces on the lining. This method is based on field observations and basic geotechnical principles. Specifically:

#### 9.5.2.1 Ring Loads

Designing for full overburden pressure is normally acceptable for tunnels at shallow depths in soil or soft ground. For the alluvium soils ($A_1$, $A_2$, $A_3$ and $A_4$) and the Puente and Fernando formation units, arching of load over the tunnel will be significant at greater depths, and design for full overburden would be overly conservative. This will become particularly significant if something other than a concrete lining is selected. It is recommended the following criteria, patterned to some extent after BARTD, be utilized for current design:

$$P_r = pR$$

where: $P_r$ = ring load (lbs/lin. ft)

$p$ = earth and water pressure on lining, assumed uniform all around (psf)

$R$ = radius of tunnel (ft).

Recommended values of design pressure "p" are as follows (see Figure 9-4 for term definition):

a. Minimum for all cases:

$$p = \gamma D$$

b. Tar sand, $SP_p$:

$$p = \gamma Z$$

[NOTE: This provides allowance for long-term creep effects.]

c. Alluvium ($A_1$, $A_2$, $A_3$ and $A_4$) and San Pedro sands (SP):

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

[NOTE: This provides full support of everything in the first 2 diameters over the tunnel, and 1/2 of everything above that.]

d. Puente and Fernando Formations (C):

i. For $Z_c < D$:

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

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

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

iii. For $Z_c > 2D$:

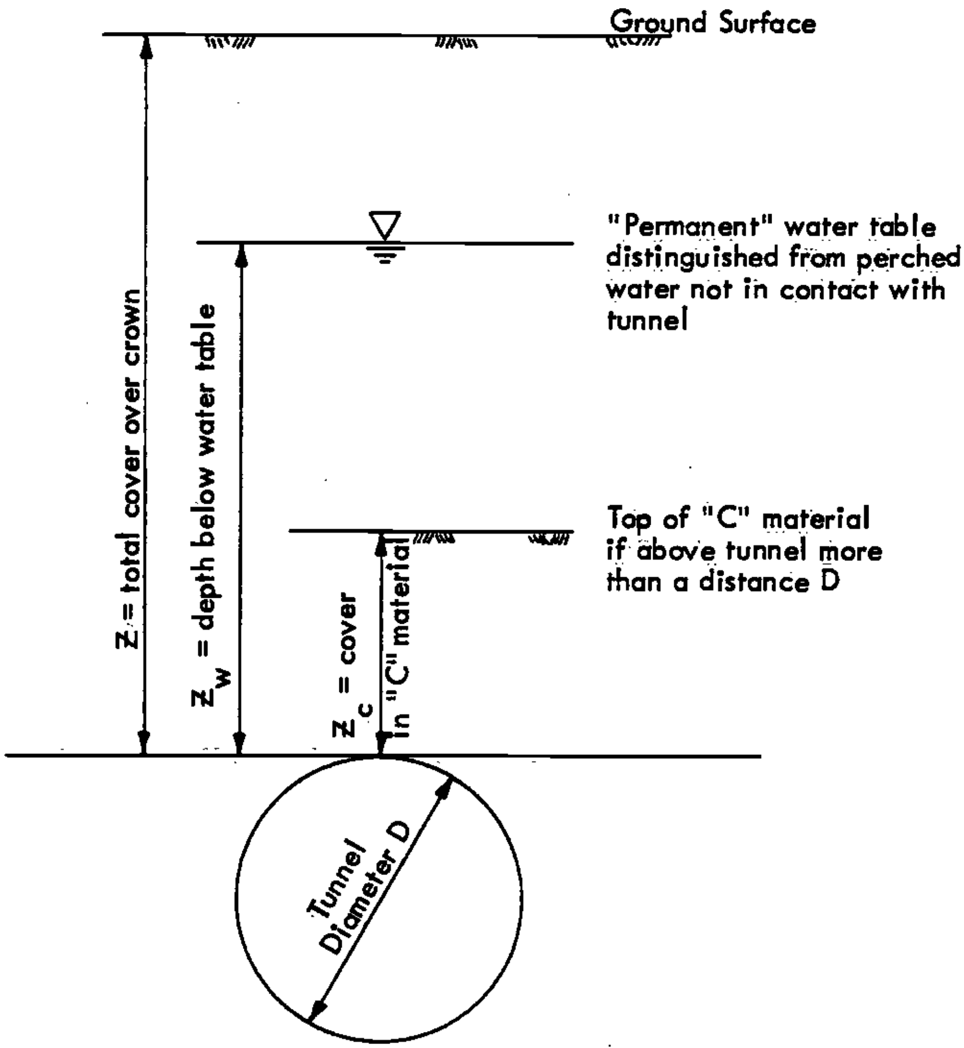
$$p = \gamma Z - q_u$$

[NOTE: Maximum p should not exceed that calculated for case of $Z_c < D$.]

All the relationships above consider full external water pressure on the lining. It is recommended that full hydrostatic pressures be used in $A_1$, $A_3$ and SP materials where the tunnel is below the water table (i.e, in "permeable" ground), but in $A_2$, $A_4$ and C materials (i.e, which are relatively "impermeable" ground), a prudent concept would be to provide permanent drainage in the tunnel to handle the small amount of long-term seepage which may occur. In this case, the hydrostatic pressure on the lining is eliminated.

9.5.2.2 Bending Moment

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




LEGEND:

$\gamma$ = total unit weight of earth if water pressure on lining eliminated by drainage, or total unit weight of earth above water table if permanent drainage not provided (pcf).

$\gamma_b$ = buoyant unit weight of earth below water table if permanent drainage not provided.

$\gamma_w$ = unit weight of water (pcf).

$q_u$ = unconfined compressive strength (psf).

TERM DEFINITION FOR TUNNEL RING LOADS IN SECTION 9.5.2.1

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Figure No. 9-4

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Approved for publication by

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

where:
$M_{max}$ = maximum bending moment (lb-ft)
$E$ = modulus of elasticity of lining (psf)
$I$ = moment of inertia of lining ($ft^4$)
$R_m$ = average radius of lining (ft)
$R$ = radius of lining (ft)
$\Delta R$ = change of radius of lining (ft).

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

9.5.2.3 Buckling

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

9.5.2.4 Parallel Tunnels

When a second tunnel is driven parallel and close to an existing tunnel, the resulting stresses on both tunnels can be greater than for a single tunnel. We recommend a minimum pillar width of one tunnel diameter except in special cases such as at portals, transition into station, braced cut, double-track tunnel, etc. In such special cases, a minimum of one radius pillar width is recommended. Recommended also that ground loading "p" (Section 9.5.2.1) for design should be increased by 20 percent where pillar width is less than one tunnel diameter. This increase applies only to the earth component of "p", not to the water component.

9.5.2.5 Surcharge from Adjacent/Overhead Buildings

According to Peck (1969), bored tunnels adjacent to or underlying existing buildings may experience marginally more radial deformation than normal during excavation and prior to installation of lining. This increased deformation apparently causes increased earth arching, with the net result that little if any increased ring load has been observed. However, for current design purposes and to accommodate the alternatives involving shallow bored tunnels, it is recommended that the methods recommended in Sections 9.4.3 to compute surcharge loads be applied to bored tunnels.

9.5.2.6 Jacking Forces

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

### 9.5.3 Tunnels in Rock Units

Reach 7 is classified as a "rock" tunneling reach. Sedimentary units, as well as the basalt unit, of the Topanga Formation will be penetrated. Suitable temporary support for these rocks may consist of steel sets and lagging. However, it is judged that the rock can be reinforced with rock bolts or shotcrete in a significant length of the tunnel. It is recommended that steel support placed on 4-foot centers be assumed in design for 35 percent of the tunnel and that the remaining length of tunnel be assumed to have rock bolts, shotcrete or any combination of steel support, bolts and shotcrete. The method of mining this reach will affect, to some extent, the requirements for support and/or reinforcement of the rock. Paragraph 7.7.7.2 indicates that a tunnel boring machine (TBM) or a drill and blast method can be used. Characteristically a TBM cuts a smooth, circular section, causing far less disturbance to the periphery rock as compared to a drill and blast tunnel, wherein secondary stresses are formed due to the blasting effect. Support and rock reinforcements, therefore, are normally less for a TBM tunnel than for a drill and blast tunnel. Controlled blasting procedures can minimize the damage to the rock surface, but these procedures can be time consuming and costly if indiscriminately specified for construction.

Design loadings for temporary support and permanent linings for hard rock tunnels are not easily quantified. A method developed by Terzaghi (1946) is commonly used (Table 9-3). This method involves estimating the rock loads based on rock conditions and tunnel dimensions. The rock conditions are estimated based on Terzaghi's rock classification which has been used throughout this report. The permanent lining in complex geometry situations should be designed for vertical loosening loads following the Terzaghi criteria. For simple circular or horseshoe cross-sections, where cast-in-place concrete lining is to be used, the design should provide a permanent concrete lining thickness of 1/2 inch per foot of tunnel diameter, or a minimum concrete thickness of 10 inches, for diameters up to 40 feet. Tunnels with diameters larger than 40 feet require special consideration. Use of the Terzaghi method results in selection of steel support sizes considered conservative, and this procedure is recommended for current design.

Table 9-4 (O'Neill, 1966) gives generalized relationships between the rock classifications and requirements for rock reinforcement and support in drill and blast underground excavations. Rock bolt reinforcement can be done by spotting bolts to pin individual blocks of rock or by pattern bolting which creates a compressive stress in the rock normal to the free surface. Pattern bolting is recommended in Reach 7 and can be effectively done when jumbos are designed for production bolting. When close jointing of rock creates a hazard of rock fall from between bolts, chain link fabric can be used in conjunction with the bolts (Table 9-4). The following criteria are recommended for pattern rock bolt design:

° Length of bolt is determined on basis of dimensions of the excavation as well as the geologic factors. Minimum bolt lengths range from 0.25 to 0.40 of the tunnel width.

TABLE 9-3

# ROCK LOADS

# TERZAGHI METHOD

**Rock load $H_p$ in feet of rock on roof of support in tunnel with width B (ft) and height $H_t$ (ft) at depth of more than $1.5\ (B + H_t)$.[1]**

| *Rock Condition* | *Rock Load $H_p$ in feet* | *Remarks* |
|---|---|---|
| 1. Hard and intact | zero | Light lining, required only if spalling or popping occurs. |
| 2. Hard stratified or schistose[2] | 0 to 0.5 B | Light support. |
| 3. Massive, moderately jointed | 0 to 0.25 B | Load may change erratically from point to point. |
| 4. Moderately blocky and seamy | 0.25 B to 0.35 $(B+H_t)$ | No side pressure. |
| 5. Very blocky and seamy | (0.35 to 1.10) $(B+H_t)$ | Little or no side pressure. |
| 6. Completely crushed but chemically intact | 1.10 $(B+H_t)$ | Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs |
| 7. Squeezing rock, moderate depth | (1.10 to 2.10) $(B+H_t)$ | Heavy side pressure, invert struts required. Circular ribs are recommended. |
| 8. Squeezing rock, great depth | (2.10 to 4.50) $(B+H_t)$ | |
| 9. Swelling rock | Up to 250 ft. irrespective of value of $(B+H_t)$ | Circular ribs required. In extreme cases use yielding support. |

1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

(After Terzaghi, 1946)

# TABLE 9-4

## CHARACTERISTICS OF COMPETENT ROCK AND GENERALIZED APPLICABILITY OF ROCK REINFORCEMENT AND SUPPORT OF UNDERGROUND OPENINGS

| ROCK CONDITIONS | ROCK CHARACTERISTICS | CONTROLLED BLASTING | ROCK REINFORCEMENT | DIRECT SUPPORT |
|---|---|---|---|---|
| HARD AND INTACT; MASSIVE | UNJOINTED, TIGHTLT CEMENTED JOINTS OR WIDE-SPACED JOINTS (6 FEET OR MORE APART) OF SUCH ORIENTATION THAT ROCK IS NOT BLOCKY. USUALLY DRY. | ROCK BREAKS ACROSS GRAIN AND DRILL HOLE TRACES USUALLT REMAIN WITH MINIMUM EFFORT. | OPTIONAL IN SMALL EXCAVATIONS. DESIRABLE IN LARGE EXCAVATIONS. GOOD ANCHORAGE FOR ANY TYPE BOLT. PATTERN BOLTS SHOULD BE LONG ENOUGH TO CROSS WIDE-SPACED JOINTS. MESH ORDINARILY NOT REQUIRED. | ORDINARILT NOT REQUIRED. |
| HARD, STRATIFIED OR SCHISTOSE | UNJOINTED, TIGHTLY CEMENTED JOINTS OR WIDE-SPACED JOINTS (6 FEET OR MORE APART). DEPENDING ON ORIENTATION OF EXCAVATION, COMBINATION OF JOINTS AND STRATIFICATION OR SCHISTOSITY MAY CAUSE SLIGHT BLOCKY CONDITION. USUALLT DRY. | GOOD RESULTS CAN BE EXPECTED WITH HIGH PERCENTAGE OF DRILL TRACES REMAINING. DIRECTION OF SCHISTOSITY WILL HAVE SOME EFFECT ON UNIFORMITY OF EXCAVATION. | BOLT PATTERN NORMAL TO SURFACE ORDINARILT SATISFACTORY BUT OCCASIONALLT SCHISTOSITY OR FOLIATION MAY REQUIRE OTHER ORIENTATION. GOOD ANCHORAGE EXPECTED FOR ANY TYPE BOLT. ADEQUATE LENGTH BOLTS SHOULD BE SELECTED WITH REFERENCE TO JOINTS AND OTHER WEAK PLANES. MESH ORDINARILY NOT REQUIRED. | IF REQUIRED, SUP PORT SYSTEMS WOULD ORDINARIL BE LIGHT. |
| MODERATELY JOINTED, MASSIVE, STRATIFIED OR SCHISTOSE | JOINT PLANES SPACED FROM 2 TO 6 FEET APART. ONE OR MORE PREDOMINENT SETS. ROCK ORDINARILT HARD AND COMPETENT HOWEVER STRATIFIED ROCK MAT HAVE SOFTER BEDS COMBINATION OF JOINTS, STRATIFICATION OR SCHISTOSITY MAY CAUSE BLOCKY CONDITION. VERY FEW SEAMS. DRY OR WET. | WITH PROPER RELATIONSHIP OF PERIMETER HOLE ALIGNMENT AND SPACING, POWDER TYPE, LOADING FACTOR AND BURDEN TO PERIMETER HOLE SPACING, GOOD RESULTS WITH SMALL PERCENT OVERBREAK CAN BE EXPECTED. DIRECTION OF JOINTS, SCHISTOSITY AND STRATIFICATION WILL AFFECT RESULTS. | ROOF CONTROL ORDINARILY GOOD. BOLT PATTERN NORMAL TO SURFACE ORDINARILY SATISFACTORY, BUT JOINT SYSTEMS AND STRATIFICATION SHOULD BE REVIEWED AS WELL AS BOLT LENGTH. GOOD ANCHORAGE EXPECTED FOR ANY TYPE BOLT EXCEPT WEDGE MAY BE DIFFICULT TO SET IN SOFT BEDS. MESR OPTIONAL BUT DESIRABLE, PARTICULARLY IF MUCH WATER. | WHERE ROCK REIN FORCEMENT NOT USED, SUPPORT RE QUIREMENTS WOUL ORDINARILY BE LIGHT. |
| CLOSELY JOINTED | JOINT PLANES SPACED LESS THAN 2 FEET APART. ONE OR MORE PREDOMINENT SETS. ROCK MAINLT HARD. COMBINATION OF JOINTS, STRATIFICATION OR SCHISTOSITY MAY CAUSE BLOCKY CONDITION. VERY FEW SEAMS. WET OR DRY. | RELATIONSHIPS LISTED ABOVE IMPORTANT. PERIMETER HOLE SPACINGS MAY HAVE TO BE REDUCED. GOOD RESULTS CAN BE OBTAINED. ROCK MAY HAVE TENDENCY TO BREAK TO JOINT SURFACES REDUCING UNIFORMITY OF SURFACE. OVERBREAK CAN BE HELD TO SMALL PERCENT. | ROOF CONTROL ORDINARILY GOOD. GOOD ANCHORAGE WITH ANY TYPE BOLT ALTHOUGH SEAMS MAY CAUSE DIFFICULTY IN SETTING WEDGE BOLTS. CARE NECESSARY TO MAKE SURE SEAMS ARE ADEQUATELY BOLTED. BOLTING AS SOON AFTER BLASTING AS POSSIBLE IMPORTANT. MESH DESIRABLE. HEADERS TO REINFORCE SEAMY OR BLOCKT AREAS MAT BE REQUIRED. WET CONDITIONS COMPOUND PROBLEMS. | WHERE ROCK REIN FORCEMENT NOT USED, SUPPORT RE QUIREMENTS LIGHT TO MODERATE. LOC AREAS OF BLOCKY AND SEAMY ROCK MAY REQUIRE SUPPORT, PARTICULARL WHERE WET. |
| MODERATELY BLOCKY AND SEAMY | MODERATELY OR CLOSELY JOINTED WITH 2 OR MORE MAJOR SETS. RANDOM FRACTURES OR JOINTS ALSO PRESENT. MANY JOINT PLANES WEAKLY CEMENTED OR OPEN. ROCK MAINLY HARD BUT SEAMS CONTAINING CLAY AND/OR CRUSHED ROCK MAY CUT ROCK IN ANY DIRECTION. WET OR DRY. | PROPER RELATIONSHIPS LISTED ABOVE BECOME MORE IMPORTANT. PERIMETER HOLE SPACING MAY HAVE TO BE REDUCED. GOOD RESULTS CAN BE EXPECTED. ORIENTATION OF JOINTS AND SEAMS MAY CAUSE INCREASED OVERBREAK. | ROOF CONTROL ORDINARILY GOOD. GOOD ANCHORAGE WITH ANY TYPE BOLT ALTHOUGH SEAMS MAY CAUSE DIFFICULTY IN SETTING WEDGE BOLTS. CARE NECESSARY TO MAKE SURE SEAMS ARE ADEQUATELY BOLTED. BOLTING AS SOON AFTER BLASTING IMPORTANT. MESH DESIRABLE. HEADERS TO REINFORCE SEAMT OR BLOCKT AREAS MAY BE REQUIRED. WET CONDITIONS COMPOUND PROBLEMS. | WHERE ROCK REI FORCEMENT NOT USED, SUPPORT R QUIREMENTS LIGH TO MODERATE. LOCAL AREAS OF BLOCKY AND SEAM ROCK MAY REQUIR SUPPORT, PARTICU LARLT WHERE WE |
| VERY BLOCKY OR SEAMY | MODERATELY OR CLOSELY JOINTED WITH 2 OR MORE MAJOR SETS. RANDOM NUMEROUS RANDOM FRACTURES AND JOINTS. JOINTS WEAKLY CEMENTED OR OPEN. MANY SEAMS CUT ROCK ALONG OR ACROSS JOINTS. CONCENTRATION OF JOINTS AND SEAMS MAY CAUSE SOFT ZONES IN OTHERWISE HARD ROCK. WET OR DRY. | ROCK HAS TENDENCY TO BREAK TO JOINTS AND SEAMS. UNIFORMITY DIFFICULT TO ACHIEVE. WITH PROPER PROCEDURES, SOUNDNESS CAN BE IMPROVED AND OVERBREAK CAN BE HELD TO FRACTION OF THAT EXPECTED WITH NORMAL BLASTING TECHNIQUES. | POSSIBLE IF ROCK NOT TOO SEAMY OR WET. EXPANSION SHELL ANCHORS PREFERRED. BOLTING SOON AFTER BLASTING AND CLOSE TO FACE IMPORTANT. WORKMEN MAY REQUIRE PROTECTION DURING INSTALLATION PROCEDURES. MESH DESIRABLE AS CLOSE TO HEADING AS POSSIBLE. FREQUENT USE OF HEADERS AND MINE TIES ANTICIPATED. | WHERE ROCK REIN FORCEMENT NOT USED, SUPPORT RE QUIREMENTS ORDINARILY MODERATE TO HEAVY. EXCEPT IONALLY SEAMY, BLOCKY AND WET AREAS WITH SHOR STAND-UP TIME SHOULD BE SUPPO ED. |

(After O'Neill, 1966)

- Length to spacing ratio for bolts should be no less than 1.5 to 2.

- Tensional bolts should be installed at a load of between 1/2 and 2/3 of the yield strength; alternatively, untensioned bolts, grouted or resin encapsulated for their full length, may be used.

- Bolts should be installed within 5 feet of the face or at the tail of the TBM and as soon as possible after excavation.

For example, for a 20-foot-diameter tunnel it is judged that a pattern of 6- to 8-foot-long bolts on a 4- to 5-foot spacing across the arch and above the spring line would serve as adequate rock reinforcement in Reach 7. Estimates should include a liberal quantity of mine ties, straps and mesh for areas where small rocks could fall from between bolts.

Deere and others (1970) have proposed guidelines, based primarily on Rock Quality Designations (RQD), for typical bolting patterns for different methods of excavation in hard rock tunnels. The subject bolting guidelines should be used only as a rough guide because the design of rock bolt systems depends on many factors in addition to RQD.

Shotcrete can be pneumatically applied to the arch and walls of the tunnel and can be effective in supporting and controlling rock. It can be used in conjunction with rock bolts, mesh reinforcement and/or steel sets where additional strength is required to resist loads. Empirical relationships are used for temporary support design criteria since analytical methods result in design thickness significantly thicker than experience proves necessary. Table 9-5 (Heuer, 1974) relates typical thickness of shotcrete for different ground conditions. The following temporary support criteria are recommended for shotcrete design:

- Initial layer of shotcrete should not be less than 5 cm (2 inches).

- Shotcrete should be applied to freshly exposed rock surfaces as soon as possible after blasting (no later than one hour after blasting) and should be completed as rapidly as possible.

- An appropriate accelerator, matched to the cement and aggregate, must be added to the mix just prior to application to achieve high early strength.

- Shotcrete aggregate should contain particle sizes up to 1/2 inch.

- Minimum compressive strength requirements for shotcrete should be 750 psi in 8 hours; 1,500 psi in 24 hours; and 4,000 psi in 28 days.

A typical shotcrete reinforced 20-foot-diameter tunnel in hard rock similar to that expected in Reach 7 would call for an initial layer no less than 2 inches thick to be applied at the heading within one hour of excavation. A subsequent layer should be applied further behind the heading where the shotcrete operation does not interfere with activities at the heading. Total typical thickness of shotcrete is anticipated to range from 4 to 6 inches. In heavy ground load areas, such as fault zones or crushed rock zones, shotcrete thicknesses would be increased and the shotcrete could be supplemented with rock bolts, mesh or steel sets. If shotcrete is planned as the permanent lining the thickness may be increased. Shotcrete is generally not practical for use with a TBM.

TABLE 9-5 Temporary Support Under Loosening Loads*

| Ground Condition | Shotcrete Application |
|---|---|
| Good Ground<br>Where support problems are minimal, but a "bald-headed" tunnel is considered unacceptable | 5 cm (2 inches) shotcrete in arch above springline |
| Fair Ground<br>Where the rock is more closely jointed or broken and the arch definitely needs some support, but sidewalls are stable. Ground allows good bond of shotcrete to excavation perimeter. | 7 to 8 cm (3 inches ±) shotcrete in arch, feather out below springline |
| Poor Ground:<br>Where tunnel walls tend to ravel some, or where a good shotcrete bond to rock cannot be achieved. | 8 to 10 cm (3 to 4 inches) shotcrete in arch.<br>7 to 8 cm (3± inches) shotcrete on walls.<br>Carry to base of walls in horseshoe tunnels. On circular tunnels, wall coverage required depends on quality of shotcrete bond to rock.<br>• If bond is of fair quality, may need only upper 270° shotcrete coverage.<br>• If bond is very poor, need full 360° coverage. |
| Very Poor Ground | If support problems are truly loosening loads, treat as above for poor ground but add another 2 to 3 cm (1 inch) of shotcrete, giving 10 to 12 cm (4 to 5 inches) in arch, 10± cm (4± inches) on walls. |

*For tunnel diameters of 4 to 6 m (15 to 20 ft); after Heuer, 1974.

### 9.5.4 Mined Stations

The behavior and design of mined stations depends upon, but is not necessarily limited to, ground conditions, geometry, mining and support method, and general construction sequencing and procedures. It is not feasible at this time to present specific design criteria to apply to all the conceivable station geometry and mining procedures which might be considered. However, for current design purposes it is recommended the loadings be in accordance with procedures outlined in Section 9.5.2 and 9.5.3. Construction of mined stations for the Metro Rail alignment is judged feasible and the estimates of soft-ground and rock loadings given above are considered an adequate basis from which to make current cost estimates and design.

## 9.6 GROUND WATER

Ground water conditions along the alignment are complex and variable. In particular, the extent of perched ground water and the occurrence of unsaturated materials in the zone between perched ground water and the underlying permanent ground water is not well defined. Detailed information and data on the ground water conditions were discussed in previous Sections 4.5, 5.2, 6.0 and 7.2, and a summary of ground water monitoring readings for all borings drilled as a part of this study is presented in Table A-5, Appendix A, Volume II.

For current design purposes, it is recommended that ground water pressures be compiled on the following basis:

- Perched ground water within the alluvium materials ($A_1$, $A_2$, $A_3$ and $A_4$) does not have hydraulic continuity with the permanent ground water which is primarly in the Fernando and Puente formations (C).
- Structures within the petroliferous San Pedro sands ($SP_p$), Fernando and Puente formations (C) and Topanga Formation materials will not have to be designed for ground water pressures or large continued water inflows, for either the short-term (during construction) or the long-term loading cases, if a permanent drainage system is provided.
- Structures within the $A_2$ and $A_4$ alluvium should be designed, for the long-term loading case, for the full pressure of either a perched or permanent ground water level. For the short-term loading case, ground water pressures and water inflows should be totally controllable by temporary dewatering systems.
- Structures within the $A_1$ and $A_3$ alluvium and San Pedro sand (SP) materials should be designed, for either the short-term or the long-term loading cases, for the full pressure of either a perched or permanent ground water level.

From the above general guidelines and Drawing 2, Table 9-6 presents recommended qualitative ground water design criteria.

TABLE 9-6 Ground Water Design Criteria

| Reach | Geologic Unit | Ground Water Level* (ft) | | Remarks in Reference to "Water Problems" |
|---|---|---|---|---|
| | | Permanent | Perched | |
| 1 | $A_1$ and C | 20 - 30 | - | Major in or in $A_{1w}$ materials |
| 2 | $A_1$ and C | 100 | - | Minimal |
| 3 | C | 120 - 150 | - | Minimal |
| | $A_4$ | - | 20 - 30 | Minor |
| 4 | C and SP | 150 | - | Major in SPw materials |
| | $A_3$ and $A_4$ | - | 20 - 40 | Minor in $A_{3w}$ materials |
| 5 | C and $SP_p$ | 150 | - | Minimal |
| | $A_2$ and $A_4$ | - | 10 - 35 | Minor |
| 6 | C | Below 200 | - | Minimal |
| | $A_2$ and $A_4$ | - | 20 - 30 | Minor |
| 7 | ** | ** | - | Minimal |
| 8 | $A_1$ and $A_3$ | 120 | - | Major at L.A. River in $A_{1w}$ and minor in $A_{3w}$ |

*Refers to level below the existing ground surface (see Drawing 2).
**"Nonwater-bearing" Topanga Formation.

# Section 10.0
# Specific Subsurface Problems in Design

# Section 10.0: Specific Subsurface Problems in Design

## 10.1 GENERAL

This section identifies certain subsurface problems and delineates specific areas which, in our opinion, deserve special design considerations.

## 10.2 GROUND WATER

Ground water conditions may influence the choice of alignment and the methods of excavation and support. Perhaps the most succinct statement of justification for ground water evaluation is the following from the text Ground Water (Freeze and Cherry, 1979):

> The tunneling literature contains references to many case histories in a wide variety of geologic environments, but while lithology, stratigraphy and structure vary from case to case, there is one feature that is remarkedly common. In case after case, the primary geotechnical problem encountered during tunnel construction involved the inflow of ground water (emphasis added). Some of the most disastrous experiences in tunneling have been the result of interception of large flows of water from highly fractured, water-saturated rocks. Tunnelers the world over know that in planning for a tunnel, it is essential to make every attempt to identify the nature of the ground water conditions that are likely to be encountered.

In some areas, particularly through Reach 1, dewatering may be difficult, requiring extensive pumping and/or use of cutoff walls/trenches. In addition to the impact on construction, ground water conditions can have a profound influence on permanent design pressures and waterproofing/drainage requirements. The difference in design pressures between a line segment/station located 60 feet below the static ground water table, and one located in an unsaturated zone below a perched ground water table, can be substantial.

Ground water conditions along the proposed alignment are believed to be complex for several reasons, including:

- With the possible exception of the Young Alluvium ($A_1$) through Reach 1, the non-tar-impregnated San Pedro sands (SPw) through Reach 4, and the Young Alluvium ($A_1$) and Old Alluvium ($A_3$) through Reach 8, the majority of the subsurface materials are either nonwater-bearing or are composed of interbedded water and nonwater-bearing zones. Thus, it is believed that the ground water regime is generally not continuous.

- A relatively shallow perched ground water table is believed to exist within the alluvium throughout much of the alignment. The occurrence of ground water within and below this zone, as well as seasonal variations, is difficult to quantify.

- In many non-water-bearing formations, water is stored in isolated lenses/pockets (granular soil in alluvium) or structural discontinuities (joints, faults, etc. in rock).

We recommend the data and procedures outlined throughout this report, particularly in Section 9.6, be implemented in regard to ground water considerations for current design. If, however, final line segment and station locations are selected in areas of major ground water uncertainties, then it may be necessary in the future to collect more detailed information. Such information might be obtained from installing additional piezometers, performing insitu permeability tests, full-scale pumping tests or model studies.

## 10.3 TAR SANDS

As discussed in Section 8.10, the proposed alignment through Reach 5 may penetrate oil and tar impregnated sands of the San Pedro Formation (commonly referred to as "tar sands"). The engineering behavior of tar sands is not well understood although some building excavations in the Los Angeles area have penetrated them (an example is discussed in Section 8.4.4). To put the problem in perspective, the extreme conditions that could conceivably occur are:

- Optimum Behavior
  The tar sands may, even at depth, behave as a medium sand with temperature sensitive cohesion. In this case, other than the determination of relative density with depth and provisions to accommodate infiltrating gas and oil, excavations into the tar sands may not involve special problems. It is understood that a tunnel has been bored through the Athabasca Oil Sands in Canada without any significant problems; however, the area was well-drained, which may or may not be the case along the Metro Rail alignment, and gas pressure reportedly was not a problem.

- Adverse Performance
  As dicussed in Section 8.10.1, the presence of gas within the tar sands may have a significant effect on the mechanical behavior of the tar sands. During rapid unloading, such as caused by an advancing excavation, tar sands with high gas contents could lose much of their strength while undergoing a large expansion. There is also some evidence that tar sands may exhibit considerable creep, particularly at elevated temperatures. These conditions could result in loss of ground, high shoring/lining pressures, loss of passive resistance, and other problems.

The differences in costs and design criteria associated with the two extreme conditions could be significant, particularly for deep excavations into the tar sands.

If design of the Metro Rail alignment indicates that tunnel grades and/or deep excavations into the tar sands are desired, it is recommended that additional studies be performed on the behavior of the tar sands. These studies might include:

- <u>Laboratory Tests</u>
  Additional testing should be performed to provide information on the strength and deformation characteristics of the tar sands at different confining pressures, strain rates and stress levels. The three-phase nature of the pore fluid (water, oil or tar, and gas) will require special testing procedures and equipment. Appropriate laboratory tests might include a series of unconsolidated-undrained triaxial tests performed at different temperatures and strain rates.

- <u>Borings and Borehole Measurements</u>
  Additional borings (which we recommend) will better define the occurrence of tar sands along the proposed alignment. Boring CEG 19, located some 2,000 feet from the proposed Fairfax Avenue Station, encountered tar sands at a depth of 14 feet, while at Hancock Park the tar sands are known to be virtually at the ground surface. The occurrence of shallow tar sands may have significant cost implications on the optimum location of the Fairfax Avenue Station and line segment depth. These additional borings should include measurements of in situ gas content and provisions for evaluating the potential for soil expansion during sampling. Sample expansion, such as observed in Boring CEG 23, may represent a key observation in identifying potential problem areas. In addition, it is recommended that consideration be given to performing in situ borehole measurements such as pressuremeter tests. The pressuremeter test (particularly the self-boring pressuremeter test which is, in effect, a miniature tunnel boring machine) is well suited to obtain strength, deformability and limited time-dependent characteristic information.

- <u>Pilot Excavation</u>
  Depending on the implications of information obtained from additional laboratory tests, additional borings and borehole measurements, it may be appropriate to perform a small pilot excavation. This might consist of constructing a small shored excavation in an area of shallow tar sands which contain a high gas content and exhibited pronounced soil expansion during sampling. A less expensive alternative might involve drilling a 6- to 10-foot diameter shaft to the depths being considered for a line segment/station. The excavations should be instrumented to measure the appropriate behavior of the tar sands. It is believed that a pilot excavation may prove to be essential in reducing the uncertainties involved in the proposed construction through Reach 5.

## 10.4 SEISMIC CONSIDERATIONS

Seismic studies were not a part of this investigation, but will be the subject of a separate study which is under preparation.

## 10.5 LIQUEFACTION/VIBRATION DENSIFICATION

In general, the granular alluvium deposits encountered along the alignment appear to be dense to very dense with standard penetration resistances generally in excess of 40 to 50 blows per foot. However, there are exceptions to this general condition, e.g., some of the granular alluvium encountered in Reaches 1, 2, and 8 was only loose to medium. Loose granular alluvium in the Los Angeles area is susceptible to liquefaction where it is below the ground water table and/or to densification due to construction vibrations. These conditions might require in situ densification, more conservative design earth pressures, piles (uplift and/or compression), additional underpinning, special construction procedures to minimize vibrations, permanent lowering of the ground water table and/or other procedures.

If current design studies locate line segment or station structures in zones of loose and saturated granular alluvium deposits, then consideration should be given to investigate such deposits in more detail.

## 10.6 CORROSION POTENTIAL

### 10.6.1 General

Results of the water analyses, from perched or permanent, shallow or deep, ground water along the proposed Metro Rail alignment, indicate reaches containing very poor to poor water quality, e.g., high in Total Dissolved Solids (TDS). For example, Boring CEG 6 (Reach 1), Boring CEG 11, (Reach 3), Boring CEG 19 (Reach 5) contain TDS exceeding 15,000 parts per million (ppm). By comparison, the U.S. Public Health Service Standard for potable domestic drinking water is a maximum of 500 ppm.

### 10.6.2 Sulfate ($SO_4$) Content

Ground water along the proposed Metro Rail alignment is high in sulfate content. The sulfate content in 19 of the 31 CEG borings sampled exceeded 150 ppm (see Appendix G, Volume II). Sulfate content above 150 ppm is generally regarded to be deleterious to concrete. For example, Type II cement is appropriate for sulfate concentrations from 150 to 1,000 ppm. Therefore, the reaches presented in Table 10-1 would appear to require Type II cement based on water samples removed from borings.

TABLE 10-1 Reaches Requiring Type II Cement*

| Boring CEG No. | Reach | Groundwater Sulfate Content (ppm) |
|---|---|---|
| 1 | 1 | 475 |
| 3 | 1 | 152 |
| 16 | 4 | 231 |
| 19 | 5 | 240 |
| 21 | 5 | 263 |
| 22 | 5 | 149 |
| 23A | 5 | 154 |
| 26 | 6 | 161 |
| 27 | 6 | 245 |
| 28A | 6 | 272 |
| 30 | 7 | 202 |
| 31 | 7 | 161 |
| 32A | 7 | 434 |
| 33 | 7 | 693 |
| 36 | 8 | 253 |
| 37 | 8 | 418 |
| 38 | 8 | 463 |

*Based on water samples removed from borings.

Borings where sulfate content of groundwater exceeds 1,000 ppm, possibly requiring special cement, are presented in Table 10-2.

TABLE 10-2 Reaches Requiring Special Cement*

| Boring CEG No. | Reach | Groundwater Sulfate Content (ppm) |
|---|---|---|
| 10 | 2 | 2,200 |
| 29 | 7 | 2,600 |

*Based on water samples removed from borings.

### 10.6.3 Sodium Chloride (NaCl) Content

Chemical analyses of ground water samples exhibit very high sodium chloride (salt) content. These "salty" waters are judged to originate from oil field brines. About 10 miles of the alignment is located near existing oil fields. Water easily corrodes metals used in construction when its sodium content exceeds 1,000 ppm. The borings presented in Table 10-3 are examples:

TABLE 10-3 Reaches Encountering Metal-Corrosive Ground Water*

| Boring CEG No. | Reach | Groundwater Sodium Chloride Content (ppm) |
|---|---|---|
| 3 | 1 | 3,000 |
| 4 | 1 | 4,600 |
| 6 | 1 | 19,000 |
| 10 | 3 | 1,400 |
| 11 | 3 | 18,000 |
| 12 | 3 | 5,340 |
| 16 | 4 | 5,700 |
| 19 | 5 | 13,600 |
| 29 | 7 | 3,090 |
| 35 | 8 | 2,218 |

*Based on water samples removed from borings.

The poor quality ground water, however, is not limited to the reaches near oil fields, as testimony from high TDS in Borings CEG 29, 33, 35 and 38 (Cahuenga Pass to north Portal).

Based on available water analysis data, and knowing much of the ground water is highly mineralized, additional corrosion studies are recommended to assist the designer in selecting proper materials for construction.

## 10.7 FAULTS

The proposed Metro Rail alignment traverses 12 known faults (Drawings 1 and 2) and will also traverse many unknown faults. In rock tunneling conditions, faults generally contain steeply dipping (near-vertical) zones of crushed or slickensided rock that may be up to several tens of feet wide. In soft-ground tunneling conditions, effects of faulting are more diffuse, e.g., instead of forming discrete ruptures that cut the rock materials, the fabric of grains and fragments is deformed by rearrangement. In both cases, fault zones could form:

° significant traps for oil and gas, e.g., 3rd Street, 6th Street, San Vicente and Santa Monica faults in the La Brea Tar Pit (Salt Lake Oil Field) area;

° barriers (or conduits) along which impounded ground water may enter the tunnel bore as a result of impervious clay and/or broken rock formed along the fault rupture;

- weak, plastic zones at ground water barriers, especially in claystone unit $C_w$, may clog an excavator or cutterhead as well as reduce the heading stability;

- very blocky and seamy rock zones (crushed rock and fault gouge containing altered materials) several tens of feet to possibly several hundred feet in width, as at the Hollywood, Hollywood Bowl and Benedict Canyon fault crossings;

- possible squeezing and/or swelling ground in major fault crossings as well as in undetected fault zones in the soft claystone unit.

The proposed alignment crosses all known fault traces at nearly right angles, thus the fault zones anticipated to be intersected will be roughly equal to the width of the fault zone.

## 10.8 GAS

It is recognized that the proposed Metro Rail alignment will pass over or near six major oil fields (Section 4.7 and Drawing 1) and over 50 percent of the alignment has been classified as either potentially gassy or gassy ground (Section 4.8 and Drawing 2). In addition to being a potential construction hazard, the presence of gas can reduce construction excavation rates substantially (Section 7.6), require special lining provisions for certain portions of the alignment (Section 7.7.5.2), and mandate adequate collection and ventilation systems for the finished project.

We recommend the data, conclusions and cautions outlined throughout this report, particularly for Reaches 1 through 5, be implemented in regard to considerations for gas for current designs. If, however, more detail than is presented in this report is required for design, it will be necessary to collect more field data. Such additional data might include locating and monitoring specific sources and pressures of gases at selected alignment locations.

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# Drawings

SILT
CLAY
SANDY SILT
SANDY CLAY
CLAYEY SILT
SILTY CLAY
SILTY SAND
CLAYEY SAND
SAND
GRAVELLY SAND
SANDY GRAVEL
GRAVEL
SILTSTONE
CLAYSTONE

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

CEG 38 .... CONVERSE WARD DAVIS DIXON EARTH SCIENCES ASSOCIATES AND GEO/RESOURCE CONSULTANTS BORINGS (1981)

NRC 4630 ...... NUCLEAR REGULATORY COMMISSION (1980); SHOWING STREET ADDRESS ON WILSHIRE

WC-7 .... WOODWARD-CLYDE BORING (1977)

K-54 ... KAISER ENGINEERING BORING (1962)

Qal ...... YOUNG ALLUVIUM
Qalo .... OLD ALLUVIUM
SP ...... SAN PEDRO FORMATION
Tf ....... FERNANDO FORMATION
Tp ...... PUENTE FORMATION
Tt ...... TOPANGA FORMATION
Tb ..... BASALT
p ...... PETROLEUM/GAS

?—? ...... GEOLOGIC CONTACT: APPROXIMATELY LOCATED; QUERIED WHERE INFERRED

? ...... FAULT: APPROXIMATELY LOCATED; QUERIED WHERE INFERRED; ARROWS INDICATE PROBABLE RELATIVE MOVEMENT

?.?...... PERCHED GROUND WATER LEVEL: QUERIED WHERE INFERRED; SOURCE CHIEFLY FROM 1981 BORINGS

?—?...... PERMANENT GROUND WATER LEVEL: QUERIED WHERE INFERRED; SOURCE CHIEFLY FROM 1981 BORINGS

GROUND SURFACE

MOISTURE CONTENT (%) / DRY DENSITY (pcf) ... 23/114
UNCONFINED COMPRESSION (psi) ..... Qu
STATIC TRIAXIAL COMPRESSION ..... ST
CYCLIC TRIAXIAL ............. CT
DYNAMIC TRIAXIAL ............. DT
RESONANT COLUMN ............ RC
PETROLEUM SAMPLE .......... P

35, 88, >100 } STANDARD PENETRATION TEST (N-VALUES, bpf)

3/10/81 .... WATER LEVEL & DATE MEASURED

40 — DIP OF BEDDING; FROM UNORIENTED CORE SAMPLES; STRIKE IS NOT ORIENTED

DOWNHOLE COMPRESSIVE WAVE VELOCITY (Vp), fps ... 5270
DOWNHOLE SHEAR WAVE VELOCITY (Vs), fps ........ 960

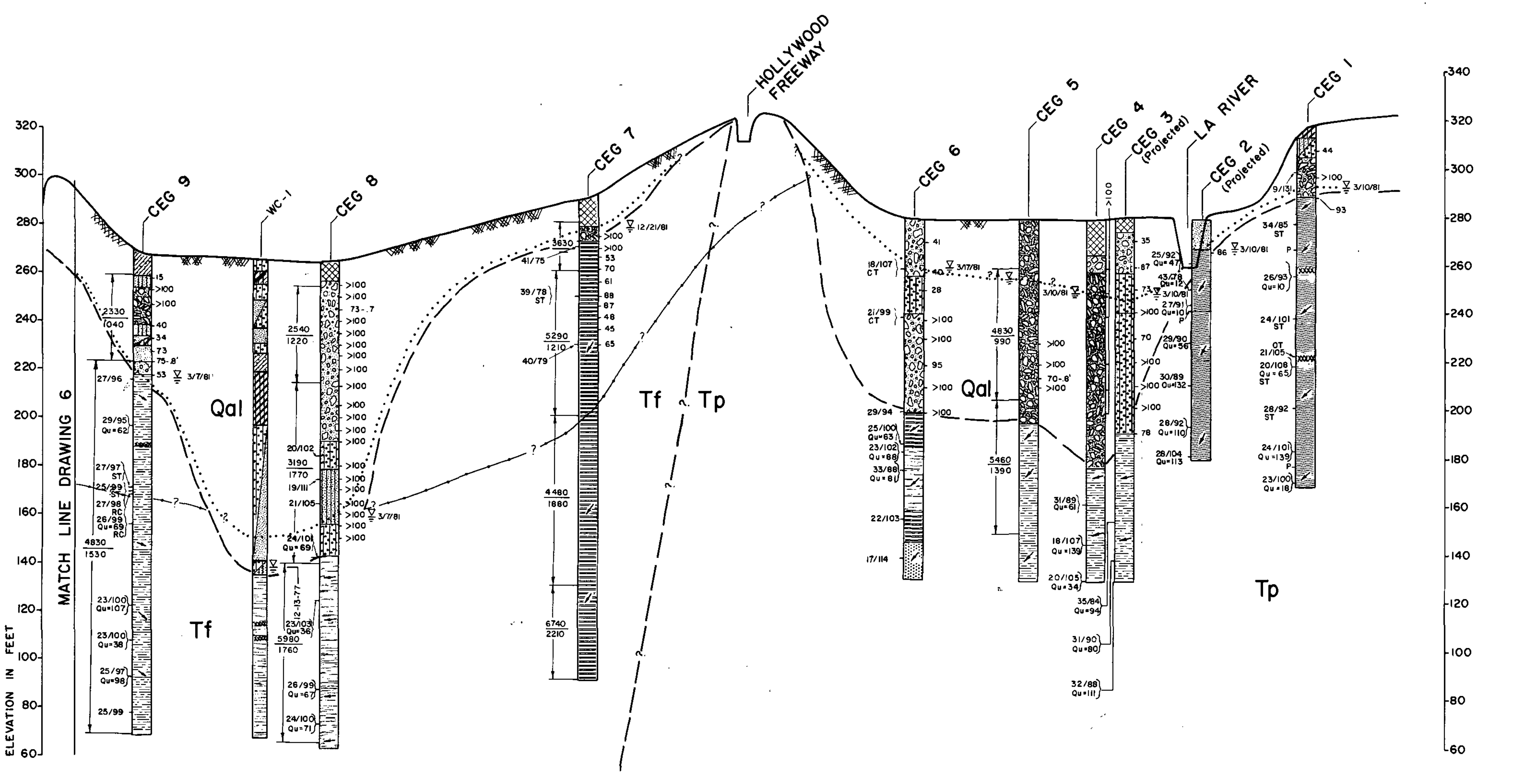
NOTES:
1) These Engineering Geology Profiles are based on interpolation between borings and were prepared as an aid in developing design recommendations. Actual conditions encountered during construction may be different.
2) The location of these profiles is the presently Proposed Metro Rail Project alignment shown on Drawing 1.
3) Borings projected more than 300' to the profile line were considered in the interpretation of subsurface conditions, and are noted on Drawing 2. However, final interpretation is based on numerous factors and may not reflect the boring logs as presented in Appendix A.
4) Refer to report for specific description of geologic units.
5) Refer to Drawing 2 for expanded classification and description of geologic symbols.
6) Refer to appendices for engineering properties of materials.

# ENGINEERING GEOLOGY PROFILE EXPLANATION

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC — Geotechnical Consultants

| | |
|---|---|
| Scale | AS SHOWN |
| Date | NOV., 1981 |
| Prepared by | T.J.M. |
| Checked by | H.A.S. |
| Approved by | J.W.B. |
| Project No. | 80-1280 |
| Drawing No. | 4 |




1000 0 500 1000
HORIZONTAL SCALE IN FEET

NOTE: SEE DRAWING 4 FOR EXPLANATION

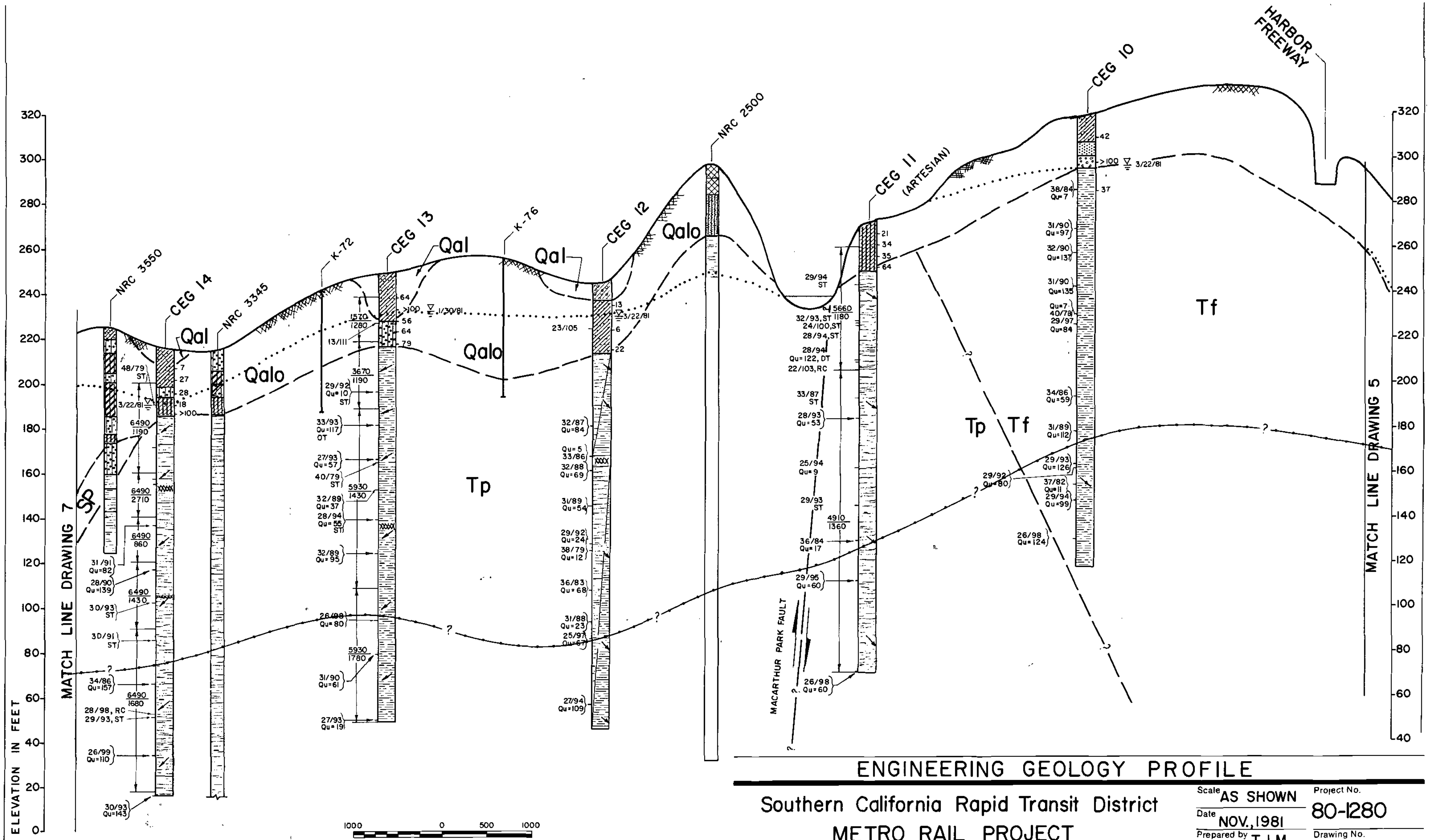
# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC

Geotechnical Consultants

| | |
|---|---|
| Scale AS SHOWN | Project No. 80-1280 |
| Date NOV., 1981 | Drawing No. |
| Prepared by T.J.M. | |
| Checked by H.A.S. | 5 |
| Approved by J.W.B. | |




NOTE: SEE DRAWING 4 FOR EXPLANATION.

# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

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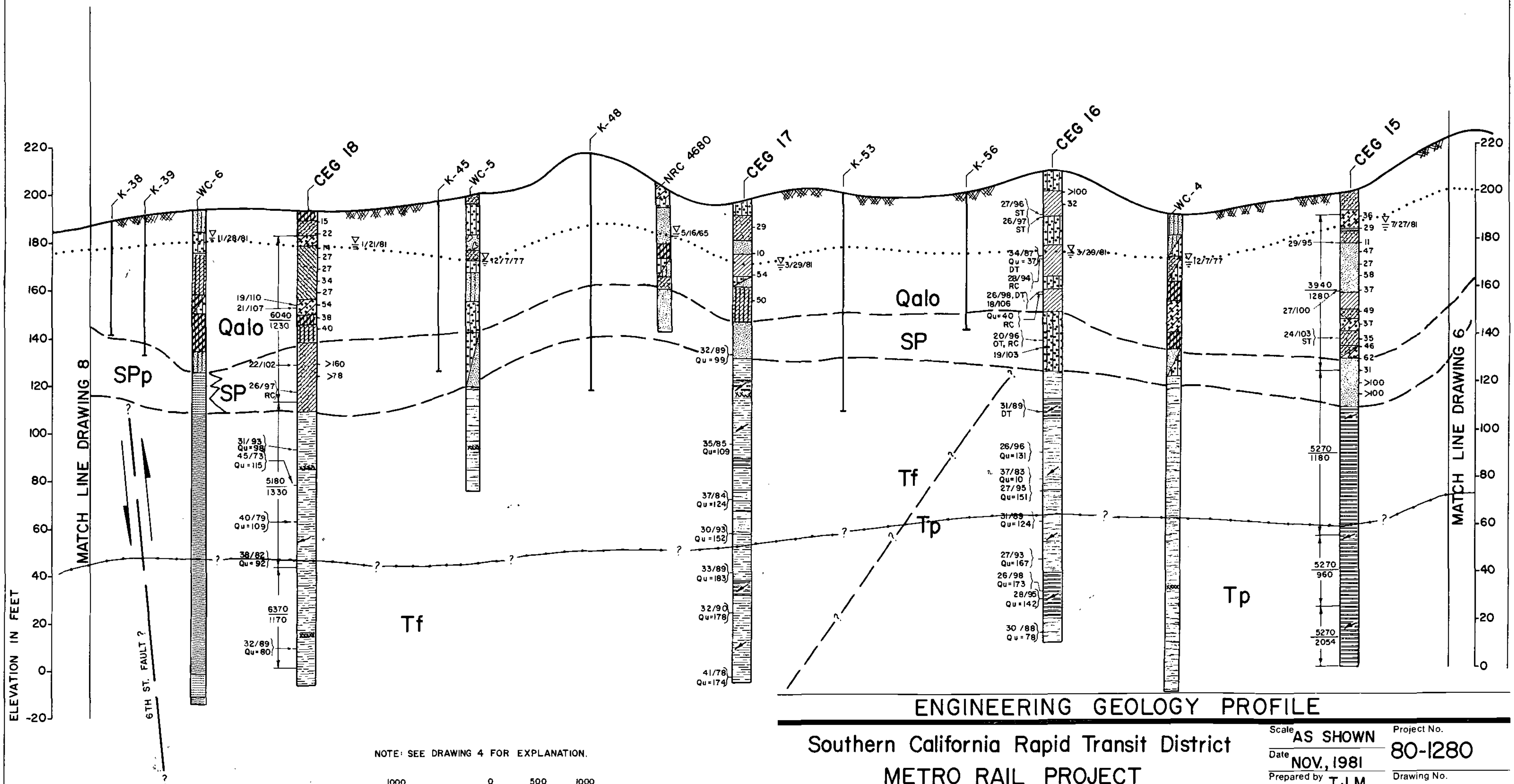
|---|---|
| Scale AS SHOWN | Project No. 80-1280 |
| Date NOV., 1981 | Drawing No. 6 |
| Prepared by T.J.M. | |
| Checked by H.A.S. | |
| Approved by J.W.B. | |




# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

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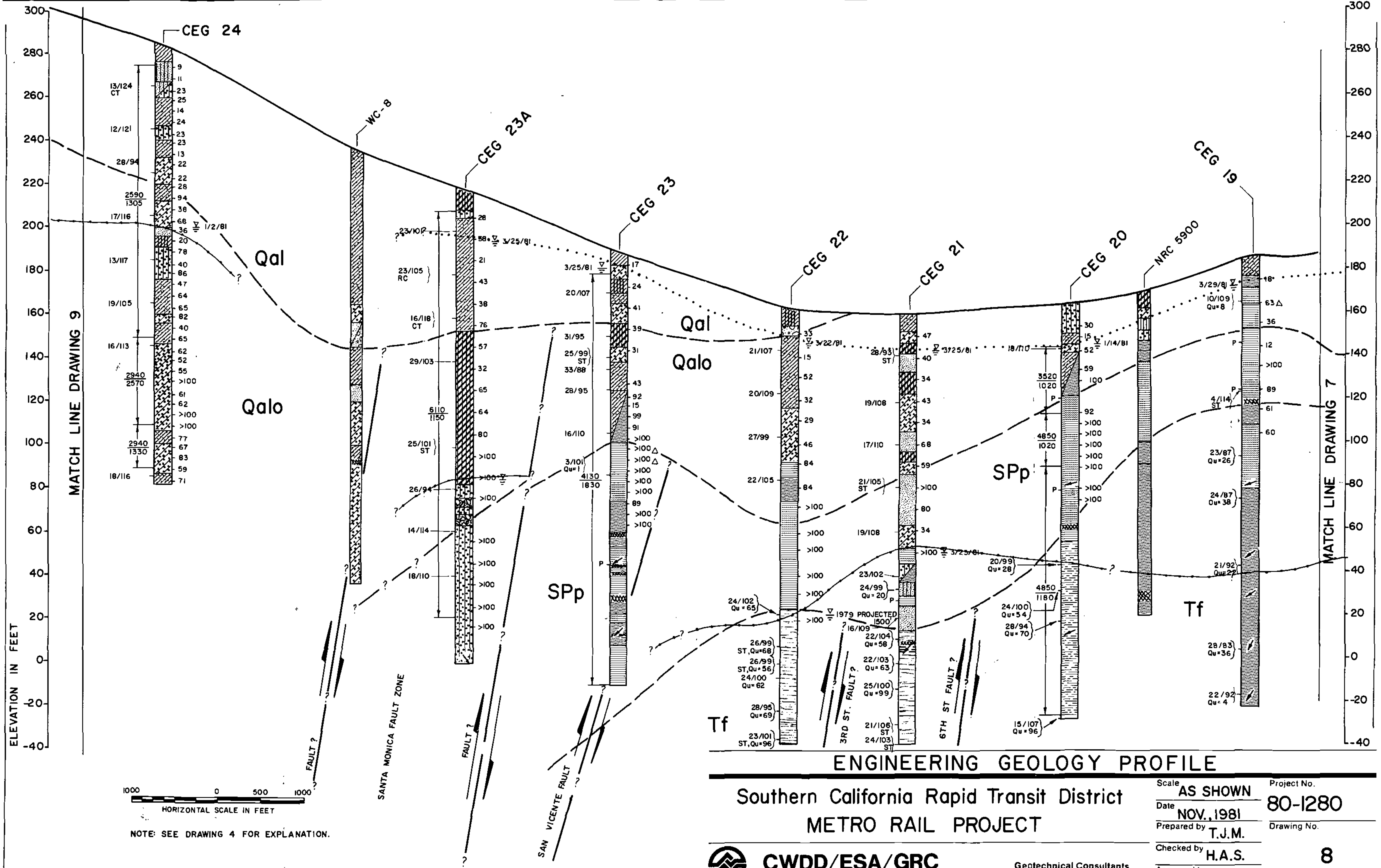
| Scale AS SHOWN | Project No. 80-1280 | |
| Date NOV., 1981 | Drawing No. 7 | |
| Prepared by T.J.M. | | |
| Checked by H.A.S. | | |
| Approved by J.W.B. | | |




NOTE: SEE DRAWING 4 FOR EXPLANATION.

# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District

METRO RAIL PROJECT

CWDD/ESA/GRC

Geotechnical Consultants

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| Scale AS SHOWN | Project No. 80-1280 |
| Date NOV., 1981 | |
| Prepared by T.J.M. | Drawing No. |
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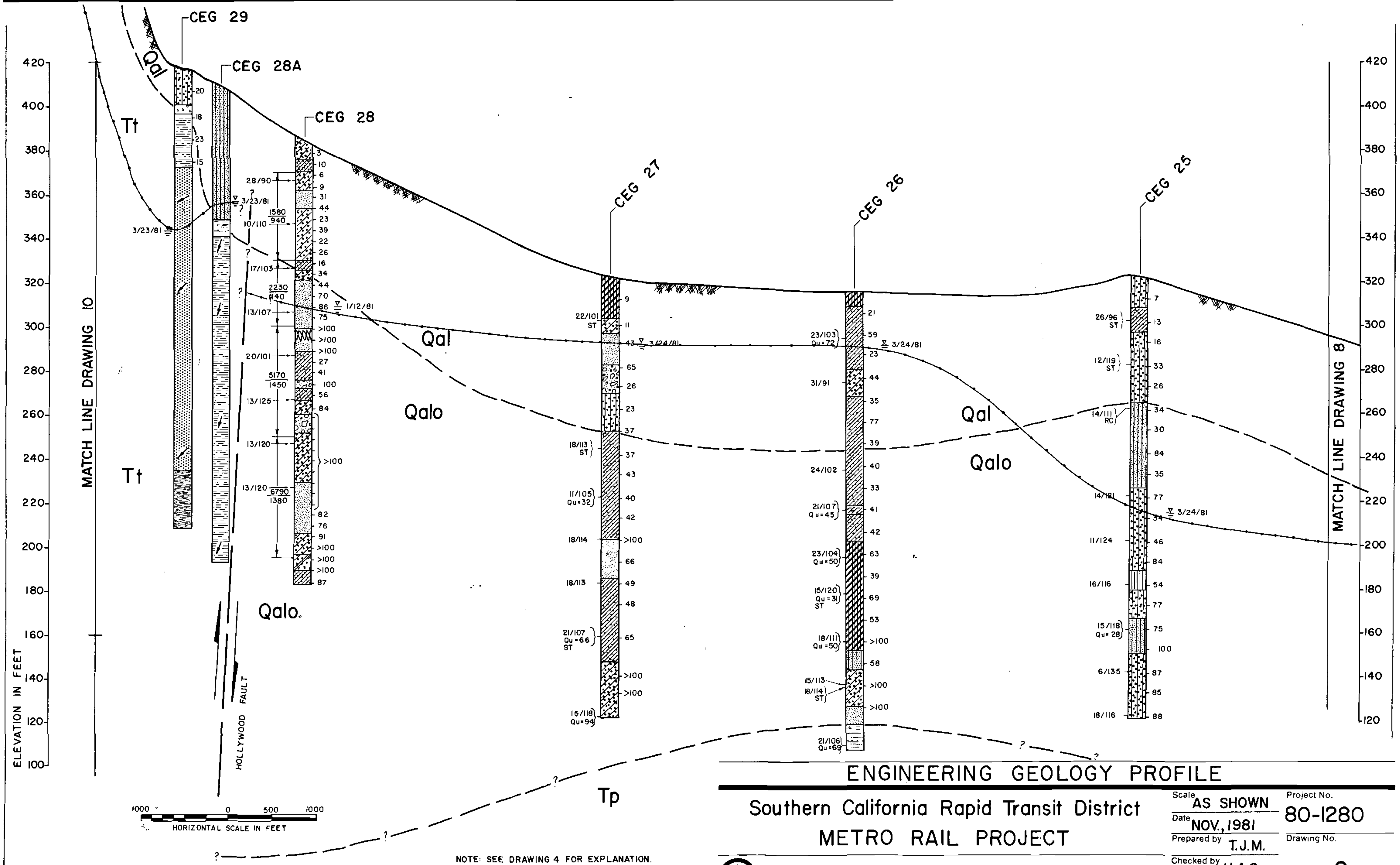
| Approved by J.W.B. | |




# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC
Geotechnical Consultants

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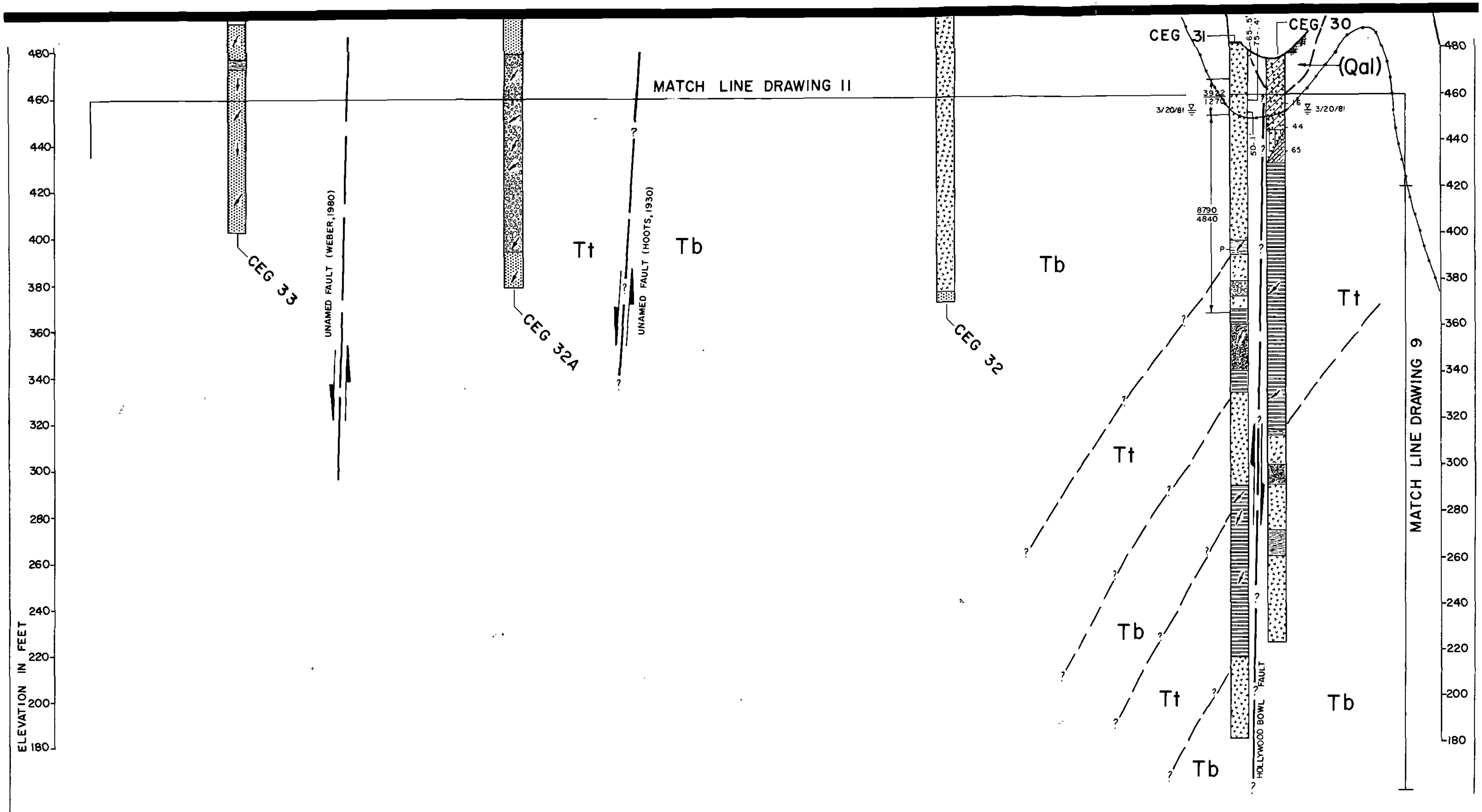
| Scale | AS SHOWN |
| Date | NOV., 1981 |
| Prepared by | T.J.M. |
| Checked by | H.A.S |
| Approved by | J.W.B. |
| Project No. | 80-1280 |
| Drawing No. | 9 |




1000 0 500 1000
HORIZONTAL SCALE IN FEET

NOTE: SEE DRAWING 4 FOR EXPLANATION.

# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

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| Scale AS SHOWN | Project No. 80-1280 |
| Date NOV., 1981 | |
| Prepared by T.J.M. | Drawing No. |
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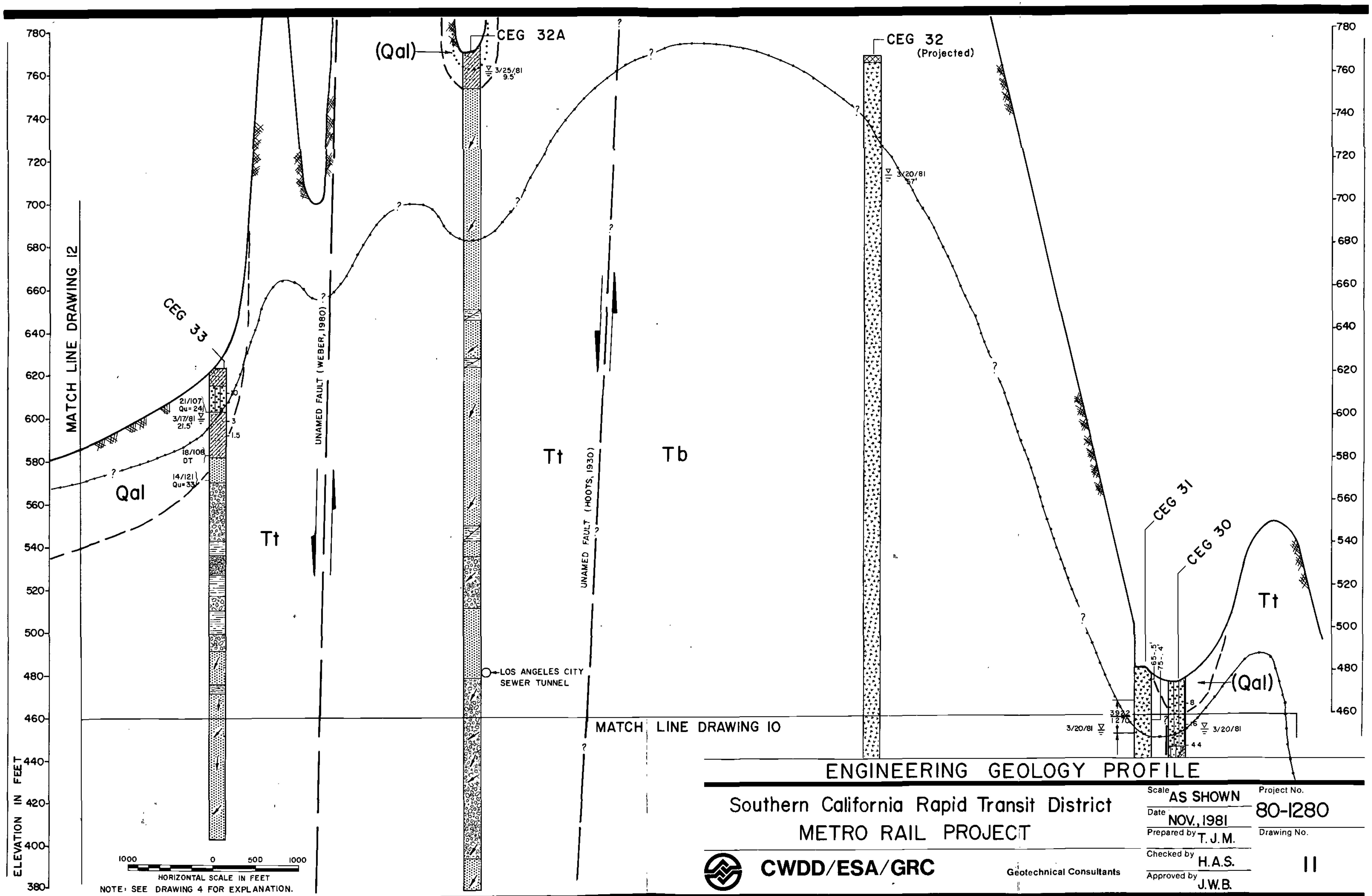
| Approved by J.W.B. | |




# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

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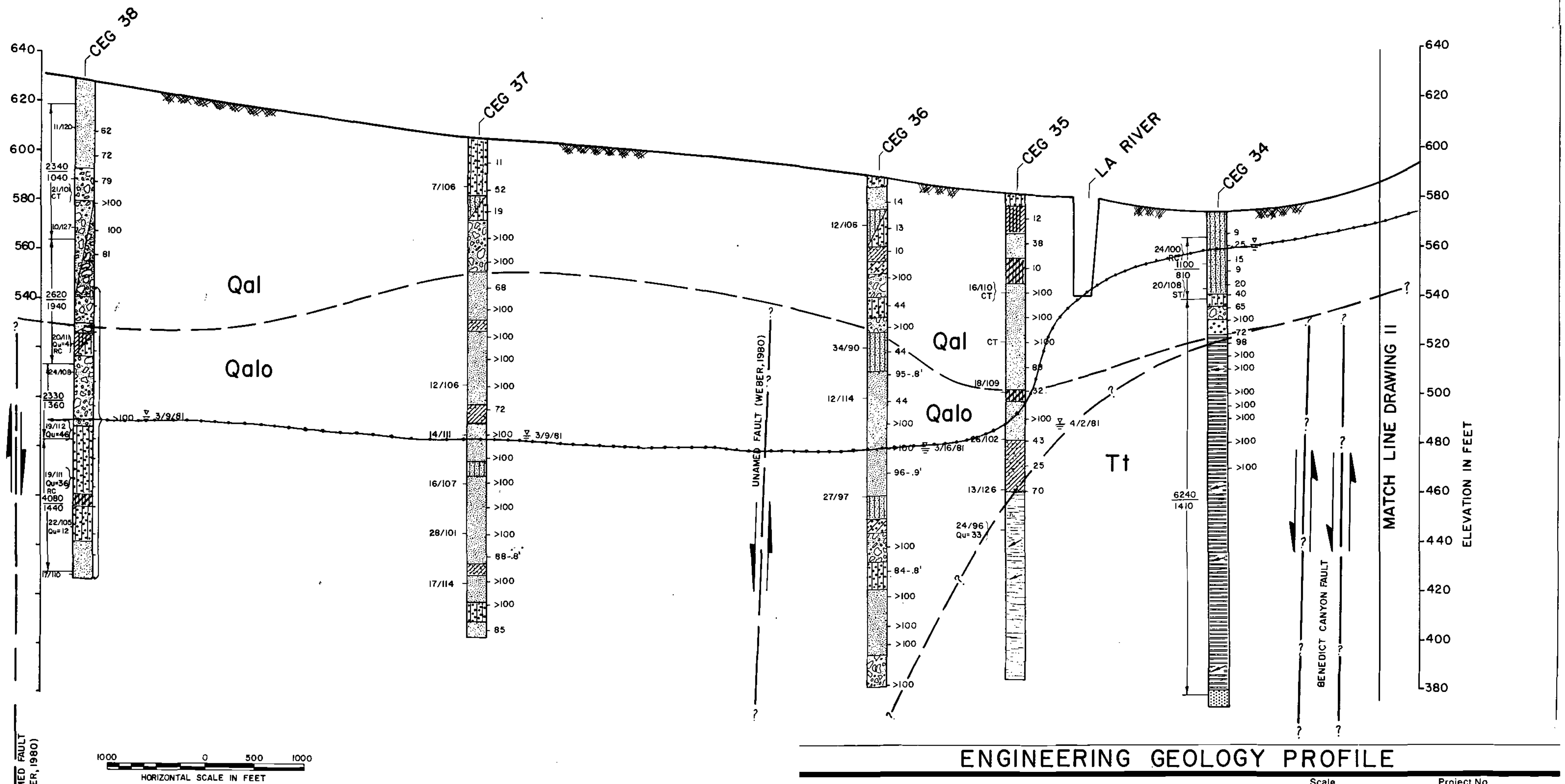
| Scale AS SHOWN | Project No. 80-1280 |
| Date NOV., 1981 | |
| Prepared by T. J. M. | Drawing No. |
| Checked by H.A.S. | 11 |
| Approved by J.W.B. | |




NOTE: SEE DRAWING 4 FOR EXPLANATION.

# ENGINEERING GEOLOGY PROFILE

Southern California Rapid Transit District
METRO RAIL PROJECT

CWDD/ESA/GRC Geotechnical Consultants

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|---|---|
| Scale | AS SHOWN |
| Date | NOV., 1981 |
| Prepared by | T.J.M. |
| Checked by | H.A.S. |
| Approved by | J.W.B. |
| Project No. | 80-1280 |
| Drawing No. | 12 |