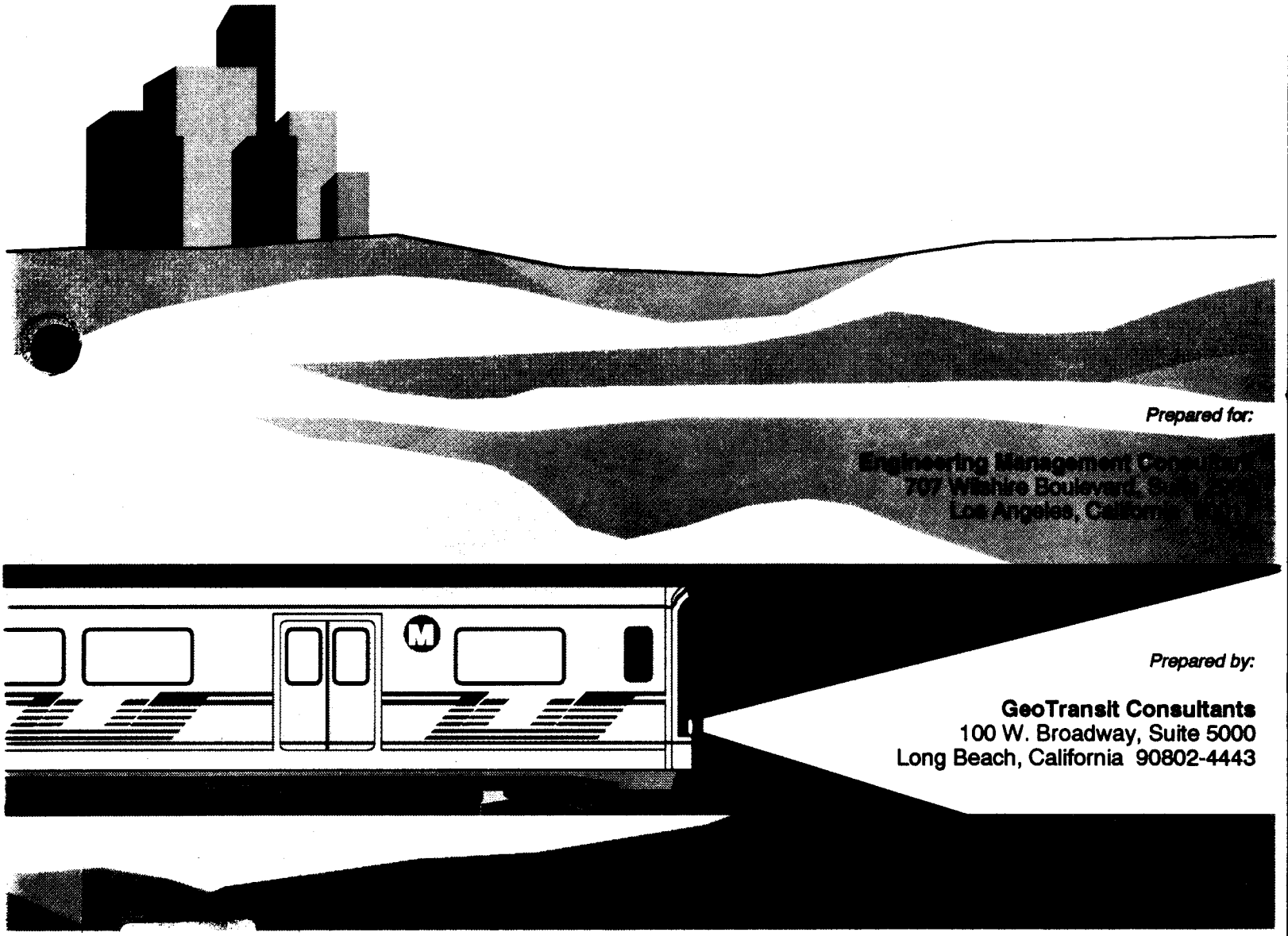
Geotechnical Investigation for:

Preliminary Engineering Program
Eastside Extension
Metro Red Line Project
Volume I of II



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Geotechnical Investigation for:

Preliminary Engineering Program
Eastside Extension
Metro Red Line Project

Volume I of II

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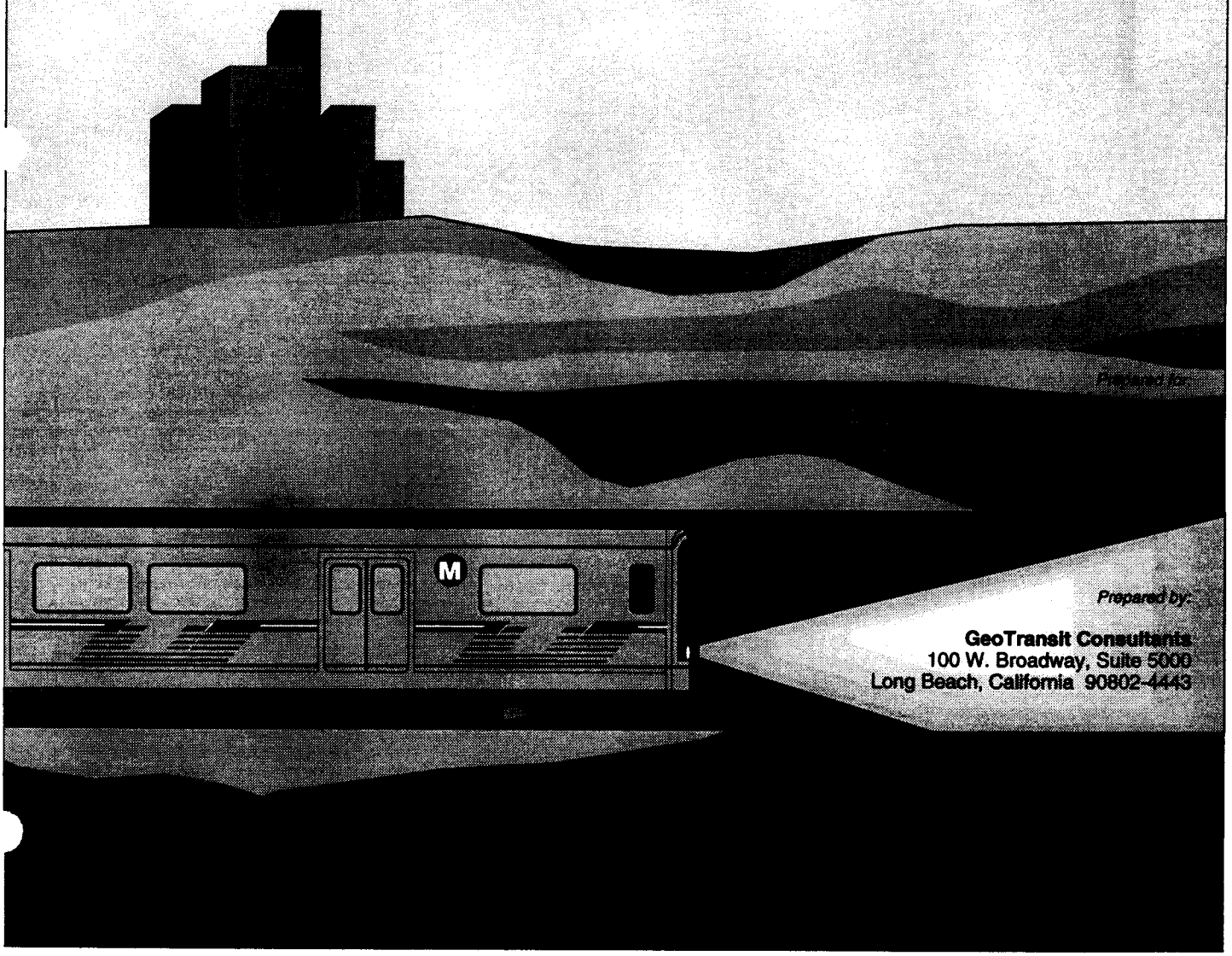

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PRELIMINARY GEOTECHNICAL INVESTIGATION
EASTSIDE EXTENSION
METRO RED LINE

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Section 1.0

1.0 EXECUTIVE SUMMARY

1.1 GENERAL

This report presents the results of a preliminary geotechnical investigation conducted by GeoTransit Consultants for the proposed Eastside Extension of the Los Angeles Metro Red Line. The primary purposes of this investigation were to gain a preliminary understanding of the geologic and geotechnical conditions and associated engineering parameters, and to identify potential constraints that may affect the planned design and construction of tunnel and station facilities along the alignment.

1.2 PROPOSED ALIGNMENT

The proposed alignment is about 6.6 miles long. It begins at Union Station in Los Angeles and ends at the intersection of Whittier Boulevard and Oakford Drive in East Los Angeles. The Eastside Extension will consist of twin 18-foot inside and 20-foot outside diameter tunnels and seven cut-and-cover stations.

1.3 SCOPE

The scope of this investigation consisted of reviewing available literature; conducting a site reconnaissance and preparing a geologic map; performing field explorations including drilling and sampling 31 geotechnical borings and six fault investigation borings, and performing seven CPT (cone penetration testing) soundings; installing 16 piezometers and converting seven selected piezometers to monitoring wells; monitoring groundwater levels; sampling groundwater from monitoring wells; performing limited geotechnical testing on selected soil and bedrock samples and limited chemical testing on selected groundwater samples; conducting an engineering evaluation; and preparing this report.

1.4 GEOLOGIC SETTING

The alignment is located along the southern flank of the Repetto Hills area of the Los Angeles Basin. In this area, the tunnels will be driven through alluvial deposits of Holocene and Pleistocene ages, and Tertiary-aged bedrock units of the Fernando and Puente Formations. Holocene alluvium is present in the Los Angeles River Narrows floodplain area overlying bedrock, and consists of mostly coarse granular deposits with local cobbles and boulders. Both granular and fine-grained deposits represent the Pleistocene-aged Old Alluvium which underlies the majority of the alignment east of the Los Angeles River Narrows area. Bedrock materials will be intermittently encountered within the tunnel envelope along the western half of the alignment. The bedrock consists of siltstone, claystone and occasional sandstone with local hard, well-cemented zones.

The alignment is located in an area having a high seismic potential and has experienced ground shaking from numerous large earthquakes in historical time. The documented active faults closest to the alignment are the east-west trending Hollywood and Raymond faults about 5 miles northwest and 4 miles northeast of the alignment, respectively. The area is underlain by the Elysian Park seismic zone, the postulated source of the 1987 Whittier Narrows earthquake. The seismic zone is postulated to be a concealed, deep thrust fault that in part expresses itself at the surface as the Elysian Hills and Repetto Hills.

A linear topographic scarp and possibly displaced drainages suggest the presence of faults ("Coyote Pass fault" and unnamed faults associated with it) that cross the alignment at least at two locations. An anomalous bedrock high beneath the alluvial deposits in the Los Angeles River Narrows is aligned with the projection of the scarp and may indicate that the Coyote Pass fault crosses the alignment at a third location. Field investigations performed for other purposes show that the Quaternary alluvial deposits in the vicinity of the alignment are being tectonically deformed above the Elysian Park seismic zone. Additional investigations will be necessary to delineate and characterize this fault and to assess its seismic capability. If the "Coyote Pass fault" is active, its potential for movement may become one of the most important concerns in

the design and construction of the proposed tunnel and stations along the proposed alignment. Another linear escarpment, which may be fault related, is located about 1.3 miles to the south of the Coyote Pass escarpment. This escarpment intersects the alignment at one location and trends directly south of and parallel to Whittier Boulevard to the west. Similarly, investigations will be necessary to evaluate this feature and its impact on the alignment.

1.5 GROUNDWATER LEVEL

Based on current plans and profiles and data from this preliminary investigation, groundwater levels are likely to be at or below the planned tunnel and station inverts east of Boring PE-14. Data from the current investigation also indicate that groundwater levels will be within or above the tunnel envelope west of Boring PE-17, except for the portion approximately between Borings PE-18 (Third Street/Santa Fe Avenue intersection) and PE-25 (MTA Railroad Yard on Santa Fe Avenue). In this area, a significant discrepancy exists between the data from the current investigation and the data obtained from a 1983 investigation by others. The 1983 data suggest a groundwater level up to 55 feet higher than the current levels measured in the vicinity of Boring PE-18. Additional investigation will be necessary to evaluate the apparent complex nature of the hydrogeological conditions in the general area that may explain the reported discrepancy.

1.6 SUBSURFACE STRATIGRAPHY

The planned tunnel and station excavations will be within Young and Old Alluvium, and the bedrock units of the Fernando and Puente formations. Young Alluvium within the western portion of the alignment between Union Station and the vicinity of Station 93+00 (referenced as the "western segment" in this report) is heterogeneous and consists of predominantly coarse-grained materials ranging from sand to gravels with local zones of cobbles and boulders (to 4 feet in size). Occasional layers of fine-grained soils consisting predominantly of sandy clay and clayey silt are also present in the western segment.

The alluvium in the remaining portion of the alignment (referred to as the "eastern segment" in this report) is Old Alluvium with Young Alluvium occurring locally along the alignment within intermittent drainage courses. Alluvium is very heterogeneous and consists of fine-grained materials (clay, sandy clay, silty clay and silt), and sand and gravel with occasional cobbles and boulders.

Bedrock units of the Fernando and Puente formations underlying the Young and Old Alluvium will be locally encountered within the tunnel envelope west of Boring PE-17. Within the planned tunnel and station excavations, the bedrock materials, when encountered, will consist predominantly of very low strength siltstone, claystone and occasionally sandstone with local layers of hard, well-cemented calcareous interbeds to 4.5 feet thick, and hard concretionary nodules from approximately 2 to 18 inches in size. Except for the hard interbeds and nodules, Fernando Formation and Puente Formation strata are expected to behave similarly to the hard and dense soils.

1.7 GASSY CONDITIONS

Available data from this investigation and several previous investigations performed in the vicinity of the Union Station area suggest the presence of hydrogen sulfide (H_2S) in groundwater and soils, and in intergranular spaces within the unsaturated zone between Union Station and the vicinity between Borings PE-18 and PE-29. At this location, there is a "bedrock high" that may be acting as a geologic barrier to H_2S and groundwater contamination.

The alignment traverses two known oil fields (Union Station and Boyle Heights oil fields). The proximity to oil fields and the permeable (granular) nature of the alluvium along the alignment suggest the likely presence of methane and other oil field related gases along the alignment, especially along the portions traversing the oil fields as evidenced by high OVA (organic vapor analyzer) readings of soil samples from Borings PE-15, (within the Boyle Heights Oil Field) and Borings PE-28 through PE-31 (within the Union Station Oil Field).

1.8 ANTICIPATED GROUND BEHAVIOR AND SUPPORT

Based on the results of this investigation and design and construction experience under similar subsurface conditions, it is anticipated that the subsurface conditions along most of the Eastside Extension are favorable for conventional soft ground/soft rock tunnel construction techniques using mechanical excavation methods within a shield, similar to those used in past and on-going Metro Red line construction. However, there are several conditions that may slow the excavation progress or create poor face stability and excessive ground settlement problems. These conditions include the local presence of cobbles and boulders to 48 and 18 inches in size along the western and eastern segments, respectively; mixed-face conditions (between alluvium and bedrock units); shallow groundwater conditions in alluvium within or above the tunnel envelope; raveling and running/flowing conditions in alluvium of predominantly granular nature; local presence of two to five feet thick, hard, well-cemented calcareous interbeds; and the potential presence of hard concretionary nodules, beds and lenses (to two feet thick) within the bedrock units; and the potential presence of H_2S and methane gases. To enhance face stability and help mitigate potential ground settlements, preconstruction dewatering to lower the groundwater below tunnel inverts, in conjunction with special provisions, such as equipping the shield with moveable hoods (poling plates) and breasting doors, the use of earthpressure balance machines or pre-construction stabilization of granular materials, will be necessary.

It is also anticipated that excavation of the proposed cut-and-cover stations can be accomplished using readily available mechanical excavation equipment and conventional shoring provisions. Preconstruction dewatering is anticipated to be required at the First/Boyle Station and possibly the Little Tokyo Station.

1.9 LIQUEFACTION POTENTIAL

Standard penetration test (SPT) blow counts are widely utilized in geotechnical engineering practice for assessing the liquefaction potential of granular materials that do not contain significant gravel or larger size materials. Very high SPT blow counts were observed in the

granular alluvium which contains varying amounts of gravel, cobbles and boulders along the alignment. These high SPT blow counts are most likely due to the presence of gravel, cobbles and boulders and do not represent the consistency of the overall granular soil matrices. Thus, these high SPT blow counts are not considered good liquefaction potential indicators of the granular alluvium along the alignment. A limited liquefaction potential evaluation performed on alluvial layers free of gravels and cobbles indicated the presence of potentially liquefiable layers of sand, 3 to 11 feet thick, within and below the tunnel zone. Additional studies using Becker Hammer blow count data are recommended to evaluate the liquefaction potential of the gravelly and cobbly layers particularly within the segment between Union Station and the vicinity of Boring PE-29, and the segment within the limits of the First/Boyle Station.

1.10 SOIL AND GROUNDWATER CONTAMINATION

The results of limited chemical testing on selected groundwater samples from this investigation and data from other investigations in the vicinity of Union Station suggest potential soil and groundwater contamination with hydrocarbons and H_2S between Union Station and the vicinity of Borings PE-18 and PE-29, where a barrier to H_2S and groundwater contamination appears to exist. Data from this investigation also indicate potential soil and local perched groundwater contamination with hydrocarbons and/or other metals or chemical compounds in the immediate vicinity of the known oil fields, LUST (Leaking Underground Storage Tank list by Regional Water Quality Control Board) sites and other sites having present or past activities that possibly resulted in adverse environmental conditions. The potential for soil and groundwater contamination for the alignment has been evaluated and is discussed in a separate Stage II Environmental Assessment Report.

1.11 CORROSION POTENTIAL

Subsurface soils are moderately to extremely corrosive to metals. Very corrosive soil samples were encountered within the fine- and coarse-grained alluvium at shallow depths and within the tunnel zone and bedrock. For the most part, soils are mildly to moderately corrosive to

concrete. Type II cement should be adequate for most of the alignment. However, occasional corrosive zones that may require Type V cement exist along the alignment.

1.12 RECOMMENDATIONS

The results of this investigation have provided a needed database for a preliminary understanding of the geologic and geotechnical conditions, and a preliminary characterization of geotechnical engineering parameters and potential ground behavior along the Eastside Extension. This investigation has also identified a number of concerns and data gaps. Further investigations will be necessary to refine the results of this investigation and to address the identified concerns and data gaps so that sufficient site- and structure-specific data are obtained for final design and construction purposes.

In addition to a detailed environmental assessment currently underway to characterize soil and groundwater contamination and gassy conditions along the alignment, we recommend that the following programs be considered:

- A detailed geotechnical investigation program with closely-spaced borings. Most of the borings should be drilled using large diameter bucket auger and/or Becker drill rigs so that the extent and size distribution of cobbles and boulders can be characterized, and the consistency of gravelly soils in alluvium can be evaluated, especially for liquefaction potential assessment purposes.

- A detailed fault study program including geologic mapping, borings, trenches, geophysical surveys and age-dating to further evaluate the presence and configuration, and to characterize the seismic capability of the "Coyote Pass fault" and an inferred fault located approximately one mile to the south.

- A geophysical program to further evaluate the alluvium/bedrock contact and to obtain seismic velocity data for alluvium in areas of shallow groundwater to support the liquefaction potential assessment.

- A detailed hydrogeological investigation consisting of a series of single and multi-staged monitoring wells in the area between Boring PE-29 and the vicinity of Boring PE-18, where a barrier to H_2S and groundwater contamination appears to exist, and where there is a discrepancy between the groundwater level data from the current investigation and a 1983 investigation conducted by others.

- Performance of pump tests and water quality characterization in the areas that require pre-construction dewatering for tunnel and station construction.

Section 2.0

2.0 INTRODUCTION

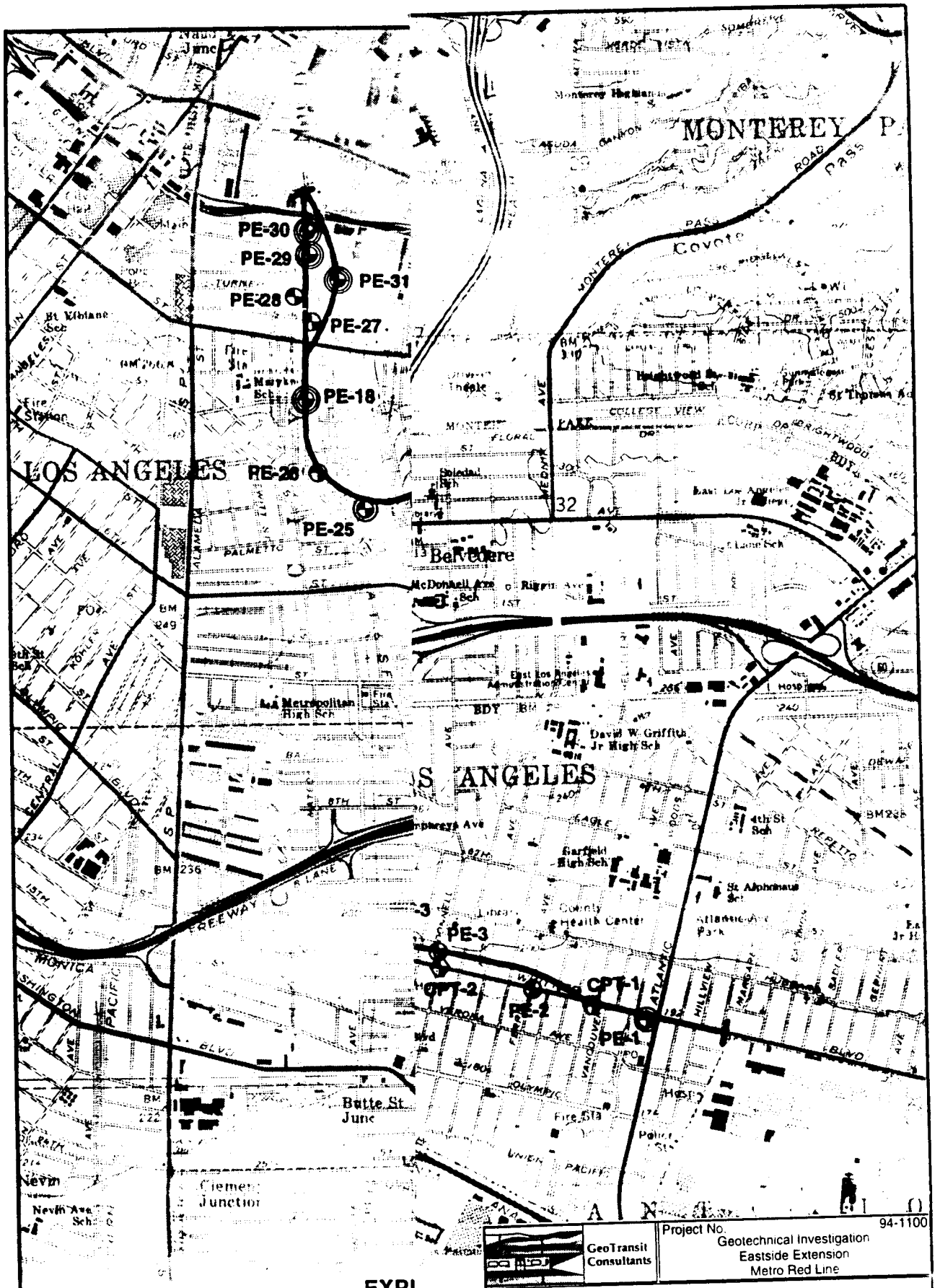
2.1 GENERAL

This report by GeoTransit Consultants presents the results of a preliminary geotechnical investigation for the proposed Eastside Extension of the Los Angeles Metro Red Line. The investigation was performed to support the preliminary engineering effort being undertaken by Engineering Management Consultant (EMC) for the Los Angeles County Metropolitan Transit Authority (MTA). The work was performed under contract to EMC in accordance with our proposal dated September 7, 1993 and our proposal for supplementary investigation dated December 6, 1993.

The primary purposes of this investigation were to gain a preliminary understanding of the geologic and geotechnical conditions and engineering parameters, and to identify potential geotechnical constraints along the alignment. In addition to providing geotechnical data for the preliminary design of the alignment, the results will be used for the development of a more detailed geotechnical investigation program to better define the subsurface conditions and geotechnical parameters along the alignment in support of the final design to be undertaken by EMC.

2.2 PROJECT DESCRIPTION

Figure 2-1 shows the location of the approximately 6.6-mile long Eastside Extension and vicinity. The western terminus is at the south end of Union Station in Los Angeles at about Station 13+20. The eastern terminus is the Whittier Boulevard/Oakford Drive Intersection (Station 363 + 92.39), about 800 feet east of the Whittier/Atlantic Station in the vicinity of the Whittier Boulevard/Atlantic Boulevard intersection in East Los Angeles.



EXPI

PE-31 ⊖ Rotary Wash Boring Drilled for

PE-1 ◎ Rotary Wash Boring with Piezc
Piezometers Converted to Mc

CPT-6 ⊕ Cone Penetration Test Condu

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d 1981

GeoTransit Consultants

Project No. 94-1100
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location Map and Exploration Plan

2-94

Figure 2-1

From Union Station the alignment runs along two southerly branches, with one branch (CR Track) running approximately southerly to South Santa Fe Avenue, and the other (CL Track) curving southeast and then southwest until merging with the CR track at the intersection of South Santa Fe Avenue and First Street. The alignment then proceeds southerly along South Santa Fe Avenue past the Little Tokyo Station, curves southeasterly and crosses the Los Angeles River north of the Fourth Street Bridge. The alignment continues in a northeasterly direction to the First/Boyle Station; southeasterly parallel to, and about 200 feet southwest of, Brooklyn Avenue and along First Street; southerly along Indiana Street and easterly along Whittier Boulevard to the eastern terminus (Plates 1A and 1B).

The Eastside Extension will consist of twin tunnels and seven stations (Little Tokyo, First/Boyle, Brooklyn/Soto, First/Lorena, Whittier/Rowan, Whittier/Arizona, and Whittier/Atlantic). The tunnels will consist of two single-track 18-foot finished diameter (about 20 feet outside diameter) openings in a double-line configuration. Based on the tunnel profile provided by EMC, the current planned depths of tunnel inverts range from about 42 feet to about 117 feet below existing grade. The tunnels are expected to be constructed using soft-ground tunneling methods. Tunnel support will consist of either a one-pass bolt and gasket liner, or a permanent concrete liner preceded by initial support during excavation. The stations are proposed to range from about 600 feet to 1,000 feet in length, 50 feet to 60 feet in width and are to be constructed by cut-and-cover methods. The current planned station inverts range from about 42 feet to about 118 feet below existing grade.

For ease of presentation and discussion purposes the proposed alignment was broken into two segments:

- Western tunnel segment covering the Los Angeles River Narrows from Station 13+20 (western terminus) to approximately Station 93+00.

- Eastern tunnel segment from approximately Station 93+00 to Station 363+92.39 (eastern terminus).

The approximate limits of these segments are shown in Figure 2-1. The station numbers are based on the most recent plan and profile drawings for the Eastside Extension Alignment (Alternatives 6A and 9B) provided by EMC in December 1993.

2.3 OBJECTIVES AND SCOPE

2.3.1 Objectives

This geotechnical investigation was limited in scope and was performed for preliminary engineering purposes only. The objectives of this investigation were as follows:

- To provide preliminary evaluations of key geologic and geotechnical issues and engineering parameters that may affect design and construction of the proposed tunnels and stations.

- To gain an initial general understanding of the groundwater and seismic conditions along the proposed alignment.

- To obtain limited information on potential subsurface contamination levels by monitoring soil samples with the organic vapor analyzer (OVA), and by limited chemical analyses of selected groundwater samples.

- To identify potential areas that require further evaluation and make preliminary recommendations for more detailed geotechnical investigation programs needed for future design.

2.3.2 Scope

The scope of this investigation consisted of the following:

1. Review of available literature and reports regarding the geologic, geotechnical, groundwater and seismic conditions along the alignment.

2. Planning and coordination of field work, including:
   - Development of field procedures and manuals
   - Planning of the field investigation program
   - Procurement of necessary permits and licenses
   - Coordination with government agencies and utility companies prior to, during, and after the field work
   - Development and implementation of a project-specific Health and Safety Plan.

3. Performance of a field exploration program, including:
   - Drilling and sampling of 31 test borings (PE-1 through PE-31)
   - Drilling 6 borings (FL-1 through FL-6) for a preliminary evaluation of a suspected fault trace (Coyote Pass fault)
   - Performing 7 cone penetrometer tests (CPT-1 through CPT-6 and CPT-6A)
   - Obtaining OVA readings on soil samples and background environments
   - Installing 16 piezometers at selected boring locations and converting seven selected piezometers into monitoring wells.
   - Obtaining groundwater samples from the monitoring wells.
   - Monitoring groundwater levels at all piezometer and monitoring well locations and taking water samples from 7 selected monitoring wells for chemical testing

4. Performance of a laboratory testing program on selected representative soil and water samples to assess the index and engineering properties of soils, and to evaluate general chemical characteristics of the encountered groundwater.

5. Preparation of this report documenting the preliminary findings, conclusions, and recommendations.

2.4 PREVIOUS INVESTIGATIONS AND AVAILABLE DATA

Results of previous geologic, geotechnical and environmental investigations performed in the vicinity of the proposed alignment were reviewed in order to obtain an early understanding of the subsurface conditions along the alignment. The approximate locations of previous work (borings) reviewed are indicated in Plates 1A and 1B, and summarized in Table 2-1.

The distribution of geologic units and geologic structural data for the study area were obtained from the geologic map of the Elysian and Repetto Hills area prepared by Lamar (1970), and the geologic map of the Los Angeles Quadrangle compiled by Dibblee (1989). Differentiation of the alluvial deposits into separate units was based on the work of Bullard and Lettis (1993) from their study of Quarternary fold deformation in the East Los Angeles area. Research by Dr. James Dolan and Dr. Kerry Sieh (personal communication, 1993 and 1994; Sieh 1993; Dolan and Sieh, 1992a, b, c) have identified topographic features which they infer to be fault related. Geologic information from these sources was compiled onto a 1 inch equals 1,000 feet scale topographic base map (Plate 1A - Geology and Exploration Location Map). The map illustrates the distribution of general geologic units and structural features such as folds, faults and bedding planes.

Previous geotechnical investigations have been conducted by various consultants for the Metro Rail project. Converse Consultants, in association with Earth Sciences Associates and Geo/Resource Consultants (1981), conducted geotechnical investigations along the original 18-mile Metro Rail project route to provide comprehensive information on subsurface soil, bedrock

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
CD-DH-1	Converse, Davis and Associates, 1972	411.2	93	-	-	-	0-1	1-10	10-93+	
CD-DH-2	Converse, Davis and Associates, 1972	435.8	112.4	30	+405.8	11-1-72	0-1	1-8	8-112.4+	
CD-BH-4	Converse, Davis and Associates, 1972	325.6	20	-	-	-	0-1	1-20+	-	
CD-BH-5	Converse, Davis and Associates, 1972	355.3	48	-	-	-	0-1	1-24	24-48+	
CD-BH-6	Converse, Davis and Associates, 1972	384.4	73	-	-	-	0-1	1-22	22-73+	
CD-BH-7	Converse, Davis and Associates, 1972	379.4	48	45.3	+334.1	10-25-72	0-1	-	1-48+	
CD-BH-8	Converse, Davis and Associates, 1972	386	48	-	-	-	0-11	11-13	13-48+	
CC:CEG-3	Converse and others, 1981	281	150.6	38	+243	12-15-83	0-5.5	5.5-88.8	88.8-150.6+	Cobbles and boulders at 42'; piezometer installed
CC:CEG-4	Converse and others, 1981	279	150.0	29	+250	12-15-83	0-14	14-101.5	101.5-150+	Hydrocarbon odor, caving at 31'; cobbly and bouldery from 21' to 101.5'; piezometer installed
CC:3-1	Converse and others, 1984	279	21.2	-	-	-	0-12	12-21.2+	-	Lost circulation at 18' and 20.5'
CC:3-1A	Converse and others, 1984	279	49.5	25	+254	9-2-83	0-6	6-49.5+	-	Lost circulation at 21', piezometer installed
CC:3-2	Converse and others, 1984	276	49.5	-	-	-	0-10.5	10.5+49.5+	-	Lost circulation at 20', and 29'; possible groundwater at 37'
CC:3-3	Converse and others, 1984	269	51	-	-	-	0-4	4-51+	-	Possible groundwater at 34'
CC:3-4	Converse and others, 1984	269	46.5	-	-	-	0-3	3-46.5+	-	Caving from 10' to 46.5'

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
CC:3-5	Converse and others, 1984	268	40.5	-	-	-	0-3	3-40.5+	-	
CC:3-6	Converse and others, 1984	268	40.8	-	-	-	0-4	4-40.8+	-	
CC:3-7	Converse and others, 1984	266	30.5	25	+241	12-15-83	0-3	3-30.5+	-	Piezometer installed
CC:3-8	Converse and others, 1984	263	30.7	-	-	-	0-2	2-30.7+	-	
CC:3-9	Converse and others, 1984	266	30	22	+244	12-15-83	0-2	2-30+	-	
CC:3-10	Converse and others, 1984	266	29.5	-	-	-	0-2	2-29.5+	-	
CC:3-11	Converse and others, 1984	264	40.5	-	-	-	0-2	2-40.5+	-	Possible groundwater at 39'
CC:3-12	Converse and others, 1984	265	50.0	-	-	-	0-3	3-50+	-	Possible groundwater at 35'
CC:3-13	Converse and others, 1984	265	39.2	-	-	-	0-3	3-39.2+	-	Possible groundwater at 34'
CC:3-14	Converse and others, 1984	266	45.1	-	-	-	0-3	3-45.1+	-	Possible groundwater at 34'
CC:3-15	Converse and others, 1984	264	30.5	23	+241	12-15-83	0-3	3-30.5+	-	Cobbles at 19'; piezometer installed
CC:3-16	Converse and others, 1984	262	20	-	-	-	1-3	3-20+	-	
CC:3-17	Converse and others, 1984	262	20	-	-	-	0-4	4-20+	-	
CC:3-18	Converse and others, 1984	262	30	-	-	-	0-5	5-30+	-	
CC:3-19	Converse and others, 1984	261	29.5	-	-	-	0-2	2-29.5+	-	
CC:3-20	Converse and others, 1984	261	30	-	-	-	0-2	2-30+	-	
CC:3-21	Converse and others, 1984	260	30	-	-	-	0-4	4-30+	-	
CC:3-22	Converse and others, 1984	260	30	-	-	-	0-3	3-30+	-	Caving at 12'
CC:3-23	Converse and others, 1984	260	31	-	-	-	0-3	3-31+	-	

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
CC:3-24	Converse and others, 1984	259	29.7	24	+235	12-15-83	0-9	9-29.7+	-	Cobbles throughout; piezometer installed
CC:3-25	Converse and others, 1984	261	10	-	-	-	0-3	3-10+	-	
CC:3-26	Converse and others, 1984	261	10	-	-	-	0-2	2-10+	-	
CC:3-27	Converse and others, 1984	259	10	-	-	-	0-2	2-10+	-	
CC:3-28	Converse and others, 1984	257	10	-	-	-	0-2	2-10+	-	
CC:3-29	Converse and others, 1984	256	10	-	-	-	0-2	2-10+	-	
CC:3-30	Converse and others, 1984	254	10	-	-	-	0-2.5	2.5-10+	-	Hydrocarbon odor at 4'
CC:3-31	Converse and others, 1984	260.4	33	-	-	-	0-8	8-33+	-	Cobbles and boulders at 15', 18 and 26'
CC:3-32	Converse and others, 1984	261.5	20	-	-	-	0-0.7	0.7-20+	-	Caving from 7' to 8' and at 10.5', 13' and 16'; boulders (16") at 11' and 14' to 15'
CC:3-33	Converse and others, 1984	365.7	35.0	-	-	-	0-5.8	5.8-35+	-	Caving and belling from 9' to 17'; sand with gravel and cobbles from 10' to 26'
CC:3-34	Converse and others, 1984	-	122.5	-	-	-	0-15.4	15.4-95.5	95.5-122.5+	Piezometer installed; hydrocarbon odor; oily from 19' to 24.5' and 87.0' to 95.5'; oily sandstone inclusions from 95.5 to 122.5; cobbles from 34.5' to 35.5'; cobbles and boulders at 38'; gravel and cobbles at 43'

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
CC:3-35	Converse and others, 1984	-	123	-	-	-	0-9.5	9.5-91.5	91.5-123+	Piezometer installed; boulders at 17'; 34.5', 42.5' to 44.5', 77' to 78', 85.5' to 86.5'; hydrocarbon odor, oily
CC:B-10	Converse and others, 1984	279	107.0	-	-	-	0-9	9-95	95-107+	Hydrocarbon odor at 15', 20', 50' and 75'. H_2S odor at 60'; caving at 24' and 70'
CC:B-11	Converse and others, 1984	271	107	-	-	-	0-29	29-98	98-107+	Hydrocarbon odor at 55', tar sands at 68' and 81', H_2S odor at 70'
53-1305:B-1	Caltrans, 1959	300.7	60	-	-	-	-	0-60+	-	
53-101:B-1	Caltrans, 1963	309.7	52	-	-	-	-	0-52+	-	
53-101:B-4	Caltrans, 1963	305.2	52	-	-	-	-	0-52+	-	
53-1314:B-3	Caltrans, 1957	358.8	60	22	+336.8	9-56	-	0-60+	-	Groundwater possibly perched. See 53-1314:B4
53-1314:B-4	Caltrans, 1957	355.1	50	-	-	-	-	0-50+	-	Dry at elevation 330.6
53-1150:B-1	Caltrans, 1964	192.9	45	-	-	-	-	0-45+	-	
53-2673:B-1	Caltrans, 1985	276.3	76	25.4	+250.9	7-14-80	0-6	6-76+	-	Cobbles to 10" at 36'
53-2673:B-2	Caltrans, 1985	275.5	35.7	-	-	-	0-6	6-35.7+	-	Cobbles to 8"; caving below 25'; landfill
53-2673:B-3	Caltrans, 1985	292.1	65	-	-	-	0-25	25-65+	-	Cobbles 8" to 12"; caving; landfill
53-2673:B-4	Caltrans, 1985	279.3	72	22.4	+256.9	7-14-80	0-28	28-35	35-72+	Organic odor in bedrock
53-2673:B-5	Caltrans, 1985	277.9	70	-	-	-	0-7	7-58	58-70+	Organic odor at 38'

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
53-2673:B-6	Caltrans, 1985	281.6	65	-	-	-	0-14	14-47	47-65+	Bedding inclined from 60° to nearly vertical
53-2673:B-7	Caltrans, 1985	292.5	44.5	-	-	-	0-27	27-44.5+	-	Cobbles to 10"; caving; landfill
53-2673:B-14	Caltrans, 1985	277.5	50	-	-	-	0-3	3-50+	-	Boulders estimated to 15"
53-2673:B-17	Caltrans, 1985	277.2	102.2	-	-	-	0-2	2-87	87-102.2+	Scattered cobbles
53-2673:B-18	Caltrans, 1985	278.3	51.8	23.0	+255.3	7-14-80	0-3	3-51.8+	-	Free hydrocarbons; large cobbles reported
53-2673:B-19	Caltrans, 1985	278.9	38.9	32.4	+246.5	9-29-54	0-20	20-38.9+	-	Cobbles reported; size unknown
53-2673:B-20	Caltrans, 1985	278.6	20.6	-	-	-	0-10	10-20.6+	-	Free hydrocarbons; cobbles reported
53-2673:B-21	Caltrans, 1985	278.3	38.3	-	-	-	0-12	12-38.3+	-	Free hydrocarbons; cobbles reported
53-2673:B-22	Caltrans, 1985	277.4	30	-	-	-	-	0-30+	-	
53-2673:B-23	Caltrans, 1985	246.0	61	11.8	+234.2	2-23-53	-	0-61+	-	Free hydrocarbons; refusal on cobbles
53-2673:B-24	Caltrans, 1985	246.0	53	12.0	+234.0	2-23-53	0-9	9-53+	-	Free hydrocarbons: H_2S odor; refusal on cobbles
53-2673:B-25	Caltrans, 1985	279.5	40	-	-	-	0-3	3-40+	-	
53-2673:B-27	Caltrans, 1985	274.0	30	-	-	-	-	0-30+	-	Cobbles to 10"
53-2673:B-28	Caltrans, 1985	274.0	28	-	-	-	-	0-28+	-	Boulders to 15"; caved from 13' to 26'
B-301	Earth Technology, 1987a	275.8	60	32	+243.8	6-1-87	-	0-60+	-	Monitoring well installed; cobbly from 18' to 20' and at 40'

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
B-302	Earth Technology, 1987a	276.6	41	28	+248.6	6-5-87	-	0.41+	-	Cobbles at 15', 19', 32' and 38' OVA >1,000 ppm at 85'
B-302A	Earth Technology, 1987a	276.6	113	-	-	-	-	0-97	97-113+	
B-303	Earth Technology, 1987a	275.1	40	27	+248.1	6-15-87	-	0-40+	-	Cobbles at 8.5' and 19'
B-303A	Earth Technology, 1987a	275.1	93	-	-	-	-	0-84	84-93+	Cobbles at 44' to 47' and 60'; OVA >1,000 ppm at 86.5'
B-304	Earth Technology, 1987a	276.4	35	27	+249.4	6-17-87	0-12(?)	12(?)-35+	-	Cobbles at 17' and 24'; monitoring well installed; hydrocarbon and H_2S odor; heaving sands from 27.5 to 35'
B-304A	Earth Technology, 1987a	276.4	60	-	-	-	0-12(?)	12(?)-60+	-	
B-305	Earth Technology, 1987a	276.2	110.5	-	-	-	0-17	17-102.5	102.5-110.5+	Cobbles at 40' and 45'
B-305A	Earth Technology, 1987a	276.2	36	27.8	+248.4	7-22-87	0-17	17-36+	-	Cobbles at 15 and 23'
B-306	Earth Technology, 1987a	278.0	98.7	26.5	+251.5	7-14-87	-	0-89	89-98.7+	Cobbles at 20'; H_2S odor at 55'; heaving sand at 75'; high OVA readings below 75'
B-201	Earth Technology, 1987b	277.4	46.5	29	+248.4	1/8/87	0-7	7-46.5+	-	
B-202	Earth Technology, 1987b	277.3	50	29	+248.3	1/8/87	0-2.75	2.75-50+	-	
B-203	Earth Technology, 1987b	276.5	60	30	+246.5	1/14/87	0-8	8-60+	-	
B-204	Earth Technology, 1987b	275.5	60	30	+245.4	1/12/87	0-5	5-60+	-	160 ppm on OVA
B-205	Earth Technology, 1987b	274.7	60	30	+244.7	1/13/87	0-5.5	5.5-60+	-	
B-206	Earth Technology, 1987b	276.8	4	-	-	-	0-4	-	-	

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
B-206A	Earth Technology, 1987b	276.5	40	29.5	+247	1/9/87	-	0-40+	-	
B-207	Earth Technology, 1987b	276.9	60	30	+246.9	1/12/87	0-10	10-60+	-	Heaving sand 45-60'
B-208	Earth Technology, 1987b	270.6	60	25	+245.6	1/13/87	-	0-60+	-	Creosote odor; cobbles at 20'
B-209	Earth Technology, 1987b	273.6	50	30	+243.6	1/21/87	0-5	5-50+	-	H_2S odor
B-112	Earth Technology, 1987c	227	45.5	30	+197	1/25/87	0-0.5	0.5-45.5+	-	Petroleum odor
B-113	Earth Technology, 1987c	226	40.5	30	+196	11/25/87	0-0.5	0.5-40.5+	-	
B-114	Earth Technology, 1987c	278	55.5	30	+248	11/26/87	0-7.5	7.5-55.5+	-	Cobbles at 14'
B-115	Earth Technology, 1987c	278	60.5	30	+248	12/1/87	0-0.5	0.5-60.5+	-	Cobbles from 32' to 51'
B-116	Earth Technology, 1987c	278	30.5	30	+248	12/2/87	0-3.5	3.5-30.5+	-	
B-117	Earth Technology, 1987c	278	60.5	30	+248	12/2/87	0-3.5	3.5-60.5+	-	Cobbles at 19'
B-1	Earth Technology, 1986	-	44	-	-	-	0-4	4-44+	-	Cobbles at 16'
B-2	Earth Technology, 1986	-	35	-	-	25.7	0-4	4-35+	-	Piezometer; cobbles at 13'
B-4	Earth Technology, 1986	-	57	-	-	-	0-4.5	4.5-57+	-	
B-5	Earth Technology, 1986	-	5	-	-	-	0-5	-	-	Rebar and bricks
B-5E	Earth Technology, 1986	-	45	-	-	-	0-4	4-45+	-	
B-6	Earth Technology, 1986	-	55	-	-	-	0-4	4-55+	-	Petroleum (?)
B-6A	Earth Technology, 1986	-	35.5	-	-	-	0-4.5	4.5-35.5+	-	
B-7	Earth Technology, 1986	-	45	-	-	-	0-5	5-45+	-	Petroleum (?)
B-8A	Earth Technology, 1986	-	15	-	-	-	0-4.5	4.5-15+	-	Hit underground tank (?)

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
B-8D	Earth Technology, 1986	-	60	-	-	-	0-4.5	4.5-60+	-	Tar
B-9A	Earth Technology, 1986	-	50	-	-	-	0-3.5	3.5-50+	-	Petroleum found in H_20
B-10	Earth Technology, 1986	-	55	-	-	-	0-2	2-55+	-	Petroleum found in H_20; heaving sand at 47
B-11	Earth Technology, 1986	-	60	23.6	-	11/14/86	0-3	3-60+	-	Piezometer installed; OVA goes off scale
2765	Los Angeles County Department of Public Works	259.0	-	109.1 90.0 113.6	+149.9 +169.0 +145.4	3-79 10-38 4-72	-	-	-	Water well
2765D	Los Angeles County Department of Public Works	-	-	-	-	-				
2766	Los Angeles County Department of Public Works	-	300	-	-	-	-	0-169	169-300+	
2766A	Los Angeles County Department of Public Works	-	300	-	-	-	-	0-185	185-300+	
2775	Los Angeles County Department of Public Works	268.0	100	52.6 40.2 70.8	+215.4 +227.8 +197.8	10-75 11-34 6-40	-	0-76	76-100+	Water well
2776A	Los Angeles County Department of Public Works	-	225	-	-	-	-	-	-	
2807	Los Angeles County Department of Public Works	200.0	556	156.2 126.6 286.2	+43.8 +73.4 -86.2	11-90 3-35 4-60	-	0-556+	-	Water well

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2808A	Los Angeles County Department of Public Works	185.0	744	196.2 192.8 220.7	-1.2 -8.5 -77.8	11-77 4-71 12-72	-	0-744+	-	
2808C	Los Angeles County Department of Public Works	197.9	-	241.7 239.8 284.0	-43.8 -41.9 -86.1	4-78 4-70 10-60	-	-	-	Water well
2818B	Los Angeles County Department of Public Works	188.6	-	162.0 131.5 388.0	+26.6 +57.1 -199.4	4-92 2-43 5-59	-	-	-	Water well
2818C	Los Angeles County Department of Public Works	187.2	-	191.0 155.0 338.0	-3.8 +32.2 -150.8	10-88 1-47 7-57	-	-	-	Water well
2818D	Los Angeles County Department of Public Works	184.5	-	351.0 146.0 363.0	-166.5 +38.5 -178.5	7-78 7-44 6-78	-	-	-	Water well
2827A	Los Angeles County Department of Public Works	227.0	680	196.2 192.8 220.9	+30.8 +34.2 +6.3	11-60 4-59 5-51	-	0-680	680+	Destroyed in 1961
2827D	Los Angeles County Department of Public Works	-	296	-	-	-	-	0.296+	-	Destroyed in 1945
2827F	Los Angeles County Department of Public Works	211.1	-	186.8 185.8 352.0	+24.3 +25.3 -140.9	5-83 11-82 4-66	-	-	-	Water well
2827G	Los Angeles County Department of Public Works	200.4	-	184.0 209.0 348.0	+16.4 -8.6 -147.6	4-92 2-51 9-60	-	-	-	Water well

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2827J	Los Angeles County Department of Public Works	210.5	-	201.3 133.2 262.8	+9.2 +77.3 -52.3	11-79 11-74 1-62	-	-	-	Water well
2828	Los Angeles County Department of Public Works	192.9	306	182 119.9 228.0	+10.9 +73.4 -35.1	9-51 3-35 3-50	-	0-306+	-	Destroyed in 1957
2828A	Los Angeles County Department of Public Works	193.0	-	189.9 120.0 206.9	+3.1 +7.3 -13.9	9-51 1-35 7-49	-	-	-	
2828D	Los Angeles County Department of Public Works	196.0	-	244.0 202.0 378.0	-48.0 -6.0 -182	2-77 2-51 9-60	-	-	-	Water well
2828E	Los Angeles County Department of Public Works	195.0	-	236.5 210.5 346.5	-41.5 -15.5 -151.5	11-80 4-51 5-61	-	-	-	Water well
2828F	Los Angeles County Department of Public Works	185.8	-	175.0 170.0 323.0	+10.8 +15.8 -137.2	4-92 4-85 9-57	-	-	-	Water well
2828G	Los Angeles County Department of Public Works	193.1	-	170.0 194.0 342.0	+23.1 -0.9 -148.9	4-92 4-77 9-60	-	-	-	Water well
2837A	Los Angeles County Department of Public Works	196.0	407	203.0 107.0 352.5	-7 +89 -156.5	1-68 1-46 9-60	-	0-407+	-	
2837B	Los Angeles County Department of Public Works	196.5	-	158.0 123.2 368.0	+38.5 +73.3 -171.5	4-92 1-35 7-74	-	-	-	Water well

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
2838A	Los Angeles County Department of Public Works	174.2	500	131.0 131.0 277.0	+43.2 +43.2 -129.8	4-92 4-92 7-78	-	0-500+	-	Water well
2838B	Los Angeles County Department of Public Works	161.6	-	142.0 138.4 263.4	+19.6 +23.2 -101.8	4-92 1-50 9-57	-	-	-	Water well
2847	Los Angeles County Department of Public Works	190.0	534	201.5 116.0 360.5	-11.5 +74.0 -170.5	11-80 2-42 5-61	-	0-534+	-	Water well
2847B	Los Angeles County Department of Public Works	203.7	470	145.0 145.0 323.0	+58.7 +58.7 -119.3	4-92 4-92 7-57	-	0-470+	-	Water well
2847C	Los Angeles County Department of Public Works	228.3	-	200.0 150.8 322.0	+28.3 +77.5 -93.7	4-92 4-35 9-60	-	-	-	Water well
2857C	Los Angeles County Department of Public Works	228.0	-	212.0 212.0 336.0	+16.0 +16.0 -108.0	11-80 2-70 8-60	-	-	-	Water well
MW-1	Thrifty Oil Company, 1993	Not reported	41.5	-	-	-	0-1	1-41.5+	-	Petroleum odor from 15-20'
MW-2	Thrifty Oil Company, 1993	Not reported	51.5	-	-	-	0-1.5	1.5-51.5+	-	Petroleum odor throughout
MW-3	Thrifty Oil Company, 1993	Not reported	46.5	-	-	-	0-0.3	0.3-41.5+	-	Petroleum odor throughout
MW-4	Thrifty Oil Company, 1993	Not reported	51.5	-	-	-	0-1.5	1.5-51.5+	-	Petroleum odor throughout
B-5	Thrifty Oil Company, 1993	Not reported	20.5	-	-	-	-	0-20.5+	-	
B-6	Thrifty Oil Company, 1993	Not reported	20	-	-	-	-	0-20+	-	
B-7	Thrifty Oil Company, 1993	Not reported	20	-	-	-	-	0-20+	-	

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TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
B-8	Thrifty Oil Company, 1993	Not reported	20	-	-	-	-	0-20+	-	
B-9	Thrifty Oil Company, 1993	Not reported	20	-	-	-	-	0-20+	-	
B-10	Thrifty Oil Company, 1993	Not reported	20	-	-	-	-	0-20+	-	
B-11	Thrifty Oil Company, 1993	Not reported	40	-	-	-	-	0-40+	-	Petroleum odor to 25'
B-12	Thrifty Oil Company, 1993	Not reported	40	-	-	-	-	0-40+	-	
BC-7	Brown and Caldwell Consultants, 1993	192	40	-	-	-	0-12	12-40+	-	Petroleum odor to 20'
BC-8	Brown and Caldwell Consultants, 1993	192	65	-	-	-	0-10	10-65+	-	Hard to drill from 45 to 65'
BC-9	Brown and Caldwell Consultants, 1993	192	60	-	-	-	-	0(?)-60+	-	Conductor casing 0-20'
E-2	EMCON, 1993	approx. 290	95.5	-	-	-	-	0-95.5+	-	
BH-3	EMCON, 1993	approx. 290	10	-	-	-	0-3	3-10+	-	
E-3	EMCON, 1993	approx. 290	91	-	-	-	-	0-91+	-	
E-4	EMCON, 1993	approx. 290	86	-	-	-	-	0-86+	-	
E-6	EMCON, 1993	approx. 290	90.5	-	-	-	-	0-90.5+	-	
E-7	EMCON, 1993	approx. 290	90	-	-	-	-	0-90+	-	
BH-7	EMCON, 1993	approx. 290	45	-	-	-	0-9	9-45+	-	
BH-8	EMCON, 1993	approx. 290	20	-	-	-	-	0-20+	-	
E-9	EMCON, 1993	approx. 290	95.5	-	-	-	-	0-95.5-	-	

TABLE 2-1. EXISTING SUBSURFACE INFORMATION

Boring/ Water Well	Source	Ground Surface Elevation (feet above MSL)	Total Depth (feet)	Groundwater[1]			Geologic Unit (depth in feet)			Comments
				Depth (feet)	Elevation (feet above or below MSL)	Date	Fill	Alluvium	Bedrock	
E-13	EMCON, 1993	approx. 290	76	-	-	-	-	0-76+	-	
E-14	EMCON, 1993	approx. 290	70	-	-	-	0-15	15-70+	-	
B-1	CTL, 1992	280	50.5	45	+235	4-10-92	0-15	15-50.5+	-	Petroleum odor to 45'
B-2	CTL, 1992	280	46.5	45	+235	4-10-92	0-15	15-46.5+	-	Petroleum odor to 25'
B-3	CTL, 1992	280	46.5	45	+235	4-10-92	0-15	15-46.5+	-	Petroleum odor to 35'

Note: 1. Most recent, historic high and historic low groundwater measurements are indicated for Los Angeles County Department of Public Works monitored water wells.

and groundwater conditions. Several of the borings completed for that investigation were located in the vicinity of the Los Angeles River near Union Station. The Converse Consultants team (1984) also conducted geotechnical investigations for Design Unit A100, for which a total of 33 borings were drilled and 5 monitoring wells were installed. In a geotechnical investigation for the Metro Pasadena Line, a number of borings were drilled by Law/Crandall, Inc. (1993) near the Union Station area.

Earth Technology (1986, and 1987c and 1987d) carried out a series of environmental investigations for the original A-130 corridor to evaluate the nature of contamination in that area. Subsequent investigations by Earth Technology (1987a and 1987b) focused on assessing the environmental and geotechnical conditions of the realigned A-130 corridor.

Subsurface data from other environmental studies for which geologic and groundwater data are available were also incorporated into the data base presented in Table 2-1 (Brown and Caldwell Consultants, 1993; Emcon Southwest, 1993; Levin-Fricke, 1993; Thrifty Oil Company, 1993; CTL Environmental Services, 1992 and Woodward Clyde Consultants, 1986).

The hydrogeologic conditions of the area are discussed in a report by the California Division of Water Resources (1961). That report provides a description of the geology of the water-bearing sediments within the Los Angeles Basin and includes historical data on the groundwater conditions. More recent groundwater information was obtained from the Flood Control District of the Los Angeles County Department of Public Works, the local agency responsible for maintaining records of groundwater levels and logs for wells in the county.

Logs of test borings from investigations performed by the California Department of Transportation (Caltrans, 1985a, 1985b, 1964, 1963, 1957a, 1957b and 1953) for seven selected freeway bridge structures near the alignment were obtained and reviewed.

Subsurface geologic and geotechnical data obtained from preconstruction and postconstruction investigations for the City Terrace trunk sewer tunnel were reviewed. The sewer tunnel trends

generally southward for about 4,600 feet as a 6.5-foot diameter tunnel driven through old alluvium and Fernando Formation and Puente Formation bedrock beneath the heights of City Terrace (Plate 1A). A report by Converse, Davis and Associates (1972) includes exploratory subsurface data from eight bucket auger borings and two NX core holes along the alignment. After completion of the tunnel by a mechanical boring machine, as-built logging of the tunnel was conducted by both Converse, Davis and Associates (1975) and LeRoy Crandall and Associates (1979) in response to a claim of changed conditions by the tunneling contractor, who asserted that hard cemented zones in the Fernando Formation had not been properly evaluated in the initial geotechnical exploration. As-built geologic logging found that less than two percent of the strata that were tunneled, contained hard well cemented, calcareous and/or siliceous nodules, lenses and discontinuous beds in otherwise uncemented sandstone, siltstone and fine silty sandstone of low to moderate hardness. Irregular well-cemented zones ranging in size from 2-inch nodules to lenses and beds up to 12 feet in length were noted in the course of logging.

Section 3.0

Section 3.0

3.0 FIELD EXPLORATION AND LABORATORY TESTING

This section provides a description of the subsurface exploration and laboratory testing performed in this program.

3.1 FIELD EXPLORATION

Field exploration consisted of drilling and sampling 31 borings along the alignment and six fault investigation borings off the alignment on the order of 1,000 feet east of CPT-5; performing seven cone penetrometer test soundings (CPT); installing standpipe piezometers in 16 borings, monitoring groundwater levels; developing seven selected piezometers into monitoring wells, and sampling groundwater from these monitoring wells. The approximate locations of the borings, piezometers, monitoring wells, and CPTs are shown in Plates 1A and 1B. Detailed location maps of borings and CPTs accompany the logs presented in Appendix A.

3.1.1 Borings

Exploratory borings for the geotechnical subsurface exploration program were drilled using Mayhew 1,000 mud rotary drill rigs with 4-7/8-inch diameter tricone drill bits producing nominal 5- to 6-inch diameter boreholes. Borings were generally drilled to depths of about 20 feet or more below the tunnel inverts, and about 30 feet below the station inverts as determined from the plan and profile drawings for the Eastside Extension alignment (Alternatives 6A and 9B) provided by EMC in September 1993. However, the plan and profile were subsequently revised by EMC in December 1993, after completion of the drilling program. Tunnel invert and station bottom elevations have been lowered within certain sections of the alignment. Due to the revision, the penetration depths of 11 borings did not reach the targeted depths of 20 and 30 feet below the tunnel invert and station bottom elevations, respectively. Soil samples were obtained at 5-foot depth intervals or at changes in stratigraphy, whichever occurred first, by

alternately using a split-spoon sampler (Standard Penetration Test Method) and a California drive sampler lined with 2.4-inch diameter by 1-inch-high brass rings.

Six exploratory borings for the evaluation of a suspected fault trace (Coyote Pass fault) were drilled using CME-75 and CME-85 hollow-stem auger drill rigs with 6-inch and 8-inch diameter drill bits, respectively. These borings were drilled to depths ranging from 27 to 96.5 feet below ground surface (BGS). Soil samples were obtained continuously from these borings using an unlined California drive sampler.

The borings were logged by a geologist or soils engineer under the direct supervision of a Certified Engineering Geologist (CEG). The materials were classified in accordance with the Unified Soil Classification System (USCS). Locations and depths of the borings are shown in Table 3-1a. Boring logs are presented in Appendix A.

3.1.2 Cone Penetration Testing (CPT)

CPT soundings were performed at seven locations to depths ranging from 8 feet to 49 feet BGS using a 1.4-inch diameter cone assembly mounted at the end of a series of sounding rods. The CPTs provide a continuous log of cone tip resistance and shaft resistance which is then used to interpret subsurface soil types and material properties based on established correlations. The CPT soundings were planned to depths of about 20 feet below the proposed tunnel inverts. At all the locations, however, the CPT probe encountered refusal prior to reaching the planned depths. The locations and penetration depths of the CPT soundings are presented in Table 3-1b. The CPT logs are presented in Appendix A.

3.1.3 Piezometer Installation

Sixteen, 2-inch diameter piezometers were installed in Borings PE-1, PE-3, PE-7, PE-8, PE-11, PE-13, PE-16, PE-17, PE-18, PE-19, PE-21, PE-23, PE-25, PE-29, PE-30 and PE-31, to monitor groundwater levels. Seven of these piezometers (PE-8, PE-16, PE-18, PE-23, PE-29,

TABLE 3-1a. FIELD EXPLORATION PROGRAM – BORINGS AND PIEZOMETERS

BORING #	APPROXIMATE STATIONING [1]	LOCATION	PURPOSE	APPROXIMATE GROUND SURFACE ELEVATION (FEET)	APPROX. TUNNEL INVERT/ STATION BOTTOM DEPTH (FEET)[1]	TOTAL PENETRATION DEPTH (FEET)[2]	PIEZOMETER INSTALLATION
PE-1	352+80	Whittier/Atlantic	Station	192	70	91	Piezometer
PE-2	335+25	Whittier/Ferris	Tunnel	192	64	70.8	
PE-3	320+25	Whittier/Mc Donnell	Station	195	63	106	Piezometer
PE-4	313+90	Whittier/Duncan	Tunnel	196	60	85.7	
PE-5	299+65	Whittier/Eastern	Tunnel	200	55	80.4	
PE-6	292+45	Whittier/Brannick	Tunnel	209	60	80	
PE-7	280+20	Whittier/Bonnie Beach	Tunnel	212	58	70.5	Piezometer
PE-8	263+55	Whittier/Eastman	Station	222	63	90.6	Piezometer
PE-9	254+00	Whittier/Ditman	Tunnel	222	55	80.8	
PE-10	241+30	Indiana/Hubbard	Tunnel	255	78	80.3	
PE-11	230+40	Indiana/Lanfranco	Tunnel	290	102	85.2	Piezometer
PE-12	219+74	Indiana/Fourth	Tunnel	315	115	95.4	
PE-13	195+40	First/Lorena	Station	292	70	90.5	Piezometer
PE-14	180+20	First/Fresno	Tunnel	320	83	101.5	
PE-15	172+00	First/Julien	Tunnel	300	57	91	
PE-16	156+94	Michigan/Saratoga	Tunnel	313	55	111.5	Piezometer
PE-17	144+00	Brooklyn/Mathews	Station	345	68	91.5	Piezometer
PE-18	42+90	Third/Santa Fe	Station	265	70	86	Piezometer
PE-19	126+96	St. Louis/New Jersey	Tunnel	310	46	72.5	Piezometer
PE-20	114+70	N. State/New Jersey	Tunnel	350	105	76	
PE-21	100+40	First/Boyle	Station	313	88	110.4	Piezometer
PE-22	91+04	Pecan/Third	Tunnel	260	45	85.5	
PE-23	83+12	Third/Clarence	Tunnel	257	70	91	Piezometer
PE-24	70+64	Fourth/Mission	Tunnel	260	83	85.3	
PE-25	62+36	MTA Railroad Yard at Santa Fe	Tunnel	263	66	86	Piezometer
PE-26	52+14	MTA Railroad Yard at Santa Fe	Tunnel	262	60	85.4	
PE-27	32+00	Center/Banning	Tunnel	269	74	81.5	
PE-28	29+00	Center/Temple	Tunnel	270	75	80.9	
PE-29	22+54	Center/Ducommun	Tunnel	270	60	82	Piezometer
PE-30	14+24	Center/Commercial	Tunnel	275	56	80.8	Piezometer
PE-31	28+50	Center/Jackson	Tunnel	270	75	83	Piezometer
FL-1		Evergreen/New Jersey	Coyotes Pass Fault Investigation	326		60	
FL-2		Evergreen/New Jersey	Coyotes Pass Fault Investigation	321		96.5	
FL-3		Evergreen/New Jersey	Coyotes Pass Fault Investigation	331.5		27	
FL-4		Evergreen/New Jersey	Coyotes Pass Fault Investigation	316		62	
FL-5		Evergreen/New Jersey	Coyotes Pass Fault Investigation	314		54.5	
FL-6		Evergreen/New Jersey	Coyotes Pass Fault Investigation	342.5		52	

NOTES:

[1] Stationing and Tunnel Invert Depths Based on Eastside Extension Tunnel Line Section Drawings Provided by EMC in December 1993

[2] Borings were planned to depths of about 20 feet below proposed tunnel invert and about 30 feet below proposed bottom slab elevations for underground stations. However, portions of the tunnel profile were revised after completion of the field program. As a result, some of the borings do not extend to the target depths.

TABLE 3–1b. FIELD EXPLORATION PROGRAM – CONE PENETRATION TESTING (CPT)

CPT #	APPROXIMATE STATIONING[1]	LOCATION	PURPOSE	APPROXIMATE GROUND SURFACE ELEVATION (FEET)	APPROXIMATE TUNNEL INVERT DEPTH (FEET)[1]	TOTAL PENETRATION DEPTH (FEET)[2]
CPT–1	345 + 80	Whittier/Clela	Tunnel	190.0	67	27.1
CPT–2	320 + 30	Whittier/Mc Donnell (Close to Boring PE–3)	Tunnel	195.0	63	41.8
CPT–3	313 + 95	Whittier/Duncan (Close to Boring PE–4)	Tunnel	196.0	60	38.9
CPT–4	180 + 25	First/Fresno (Close to Boring PE–14)	Tunnel	320.0	83	35.4
CPT–5	157 + 00	Saratoga/Michigan (Close to Boring PE–16)	Tunnel	313.0	55	49.2
CPT–6	144 + 05	N. Mathews/Brooklyn (Close to Boring PE–17)	Tunnel	345.0	68	9.8

NOTES:

[1] Tunnel Invert Depth Based on Eastside Extension Tunnel Line Section Drawings Provided by EMC in December 1993

[2] Encountered refusal

PE-30, and PE-31) were converted to monitoring wells for groundwater sampling. Within the Los Angeles River Narrows (western segment), the piezometers (PE-18, PE-23, PE-25, PE-29, PE-30 and PE-31) were generally screened from about 5 to 10 feet above the anticipated groundwater level (estimated at about 30 feet BGS from previous data) to the bottom of each boring. In the eastern segment, the piezometers (PE-1, PE-3, PE-7, PE-8, PE-11, PE-13, PE-16, PE-17, PE-19 and PE-21) were screened over a 30-foot zone encompassing the originally proposed tunnel envelope. However, due to the December 1993 revisions, the screened intervals do not completely cover the tunnel envelopes in Borings PE-1, PE-3, PE-7, PE-11, PE-13, and PE-16. Piezometer installation diagrams are presented in Appendix A.

3.1.4 Groundwater Level Monitoring and Sampling

Groundwater levels were monitored in the piezometers and monitoring wells using an electronic water-level indicator. Groundwater level readings taken periodically after the piezometer installation, are summarized in Table 3-2. Piezometers PE-8, PE-16, PE-18, PE-23, PE-29, PE-30 and PE-31 were developed and groundwater samples obtained to evaluate the potential extent of groundwater contamination.

3.2 LABORATORY TESTING PROGRAM

Laboratory testing (geotechnical and chemical testing) was performed on selected soil and groundwater samples obtained in this investigation. The geotechnical laboratory test program was intended to aid in soil classifications, provide preliminary indications of subsurface conditions and evaluation of engineering parameters of soils and bedrock. The chemical testing was limited in scope and was performed for a preliminary evaluation of the potential extent of groundwater contamination. The following sections provide a general description of the test program.

TABLE 3–2. SUMMARY OF GROUNDWATER LEVEL READINGS

PIEZOMETER	LOCATION	APPROXIMATE STATIONING [1]	APPROXIMATE GROUND SURFACE ELEVATION (FEET)	TOTAL DEPTH (FEET)	WELL SCREEN [4] INTERVAL (FEET)	APPROXIMATE DEPTH TO TUNNEL		GROUNDWATER [2]			
						CROWN (FEET)	INVERT (FEET)	DEPTH (FEET)	ELEVATION (FEET)	DEPTH (FEET)	ELEVATION (FEET)
								12/8/1993 TO 12/15/1993		01/11/1994	
PE–1	Whittier/Atlantic	352 + 80	192	91.0	34.8 – 64.8	50	70	DRY	–	DRY	–
PE–3	Whittier/Mc Donnell	320 + 25	195	106.0	45.0 – 75.0	40	63	DRY	–	DRY	–
PE–7	Whittier/Bonnie Beach	280 +20	212	70.5	25.0 – 55.0	38	58	DRY	–	DRY	–
PE–8	Whittier/Eastman	263 +55	222	90.6	35.0 – 65.0	40	63	62.5 [3]	159.5	DRY	–
PE–11	Indiana/Lanfranco	230 + 40	290	85.2	40.0 – 70.0	82	102	DRY	–	DRY	–
PE–13	First/Lorena	195 + 40	292	90.5	35.0 – 65.0	50	70	DRY	–	DRY	–
PE–16	Michigan/Saratoga	156 + 94	313	111.5	64.5 – 94.5	35	55	52.3	260.7	51.8	261.2
PE–17	Brooklyn/Mathews	144 + 00	345	91.5	35.0 – 65.0	37	61	57.3	287.7	57.7	287.3
PE–18	Third/Santa Fe	42 + 90	265	86.0	24.0 – 84.0	45	70	78.7	186.3	79.2	185.8
PE–19	St. Louis/New Jersey	126 + 96	310	72.5	24.1 – 69.1	36	56	31.9	278.1	32.1	277.9
PE–21	First/Boyle	100 + 40	313	110.4	25.0– 90.0	64	88	58.2	254.8	58.4	254.6
PE–23	Third/Clarence	83 + 12	257	91.0	25.0 – 85.0	50	70	53.8	203.2	54.2	202.8
PE–25	MTA Railroad Yard	62 + 36	263	86.0	25.0 – 85.0	46	66	72.2	190.8	72.3	190.7
PE–29	Center/Ducommun	22 + 54	270	82.0	20.0 – 80.0	40	60	33.8	236.2	34.9	235.1
PE–30	Center/Commercial	14 + 24	275	80.8	17.5 – 78.0	36	56	36.0	239.0	36.5	238.5
PE–31	Center/Jackson	28 + 50	270	75.0	20.0 – 80.0	55	75	37.5	232.5	38.3	231.7

NOTES:

[1] Stationing and Tunnel Invert Depths Based on Eastside Extension Tunnel Line Section Drawings Provided by EMC in December 1993

[2] Groundwater depths measured between December 8 and 15, 1993 and on January 11, 1994

[3] On December 2, 1993, the groundwater level rteading was 55.0 feet below surface in piezometer PE–8

[4] The screened interval does not cover the entire tunnel envelope in monitoring wells PE–1, PE–3, PE–7, PE–11, PE–13, and PE–16 due to revisions in the proposed tunnel profile after completion of field exploration program

3.2.1 Geotechnical Laboratory Testing

All drive, split spoon and bulk samples obtained during the subsurface exploration were brought to Earth Technology's Huntington Beach laboratory where they were visually examined to verify field classification. Samples of the various material types encountered were selected for laboratory testing. The laboratory test program was designed to classify the predominant soil types encountered in the borings and to evaluate the insitu moisture and density, gradation, shear strength, unconfined compressive strength, consolidation characteristics, and corrosion potential. The tests were performed in accordance with applicable standard test methods specified by the American Society for Testing Materials (ASTM), the Environmental Protection Agency (EPA), or the California Department of Transportation (Caltrans).

The test program and applicable test standards are summarized in Table 3-3. Laboratory test results are summarized in Table 3-4 and are included in Appendix B. Insitu density and moisture content are also shown on the boring logs included in Appendix A. A discussion of the engineering properties of subsurface materials is presented in Section 5.2.

3.2.2 Analytical (Chemical) Testing of Groundwater

A limited analytical (chemical) testing program was performed on groundwater samples obtained from Monitoring Wells PE-8, PE-16, PE-18, PE-23, PE-29, PE-30 and PE-31. Laboratory analyses were performed by Pace, Incorporated, and CKY, Incorporated, both state certified hazardous waste testing laboratories.

The test program and relevant test standards are summarized in Table 3-5. The results of the analytical testing of groundwater are summarized in Table 3-6 and presented in Appendix C. An evaluation of the results and discussions of potential impacts on construction are presented in Sections 5.4 and 6.5, respectively.

TABLE 3-3. GEOTECHNICAL LABORATORY TEST PROGRAM

TEST TYPE	NUMBER OF TESTS	TEST PROCEDURE
Visual Soil Classification	Every Sample	ASTM D2487 / D2488
Moisture Content	257	ASTM D 2216
Dry Density	211	ASTM D 2937
Grain Size Distribution	79	ASTM D 422
Grain Size Distribution (With Hydrometer Analysis)	22	ASTM D 422
Percent Passing #200 Sieve	82	ASTM D 1140
Atterberg Limits	72	ASTM D 4318
Specific Gravity	6	ASTM D 854
Direct Shear (3 Points)	24	ASTM D 3080
Unconfined Compression	7	ASTM D 2166
One Dimensional Consolidation	6	ASTM D 2435
pH	31	EPA Method 9045
Chloride Content	31	CALTRANS Test 422
Sulphate Content	31	CALTRANS Test 417-B
Electrical Resistivity	31	CALTRANS Test 532

TABLE 3-4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 1 OF 16)

Boring No	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Ce	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-1	D-1	6.0	CL	Qya	(13)	20.1	107.6												7.25	102	ND	84	1250
	S-2	10.0	CL		31																		
	D-3	15.0	SC		(10)	14.0	115.3		44														
	S-4	20.0	CL		25																		
	D-5	25.0	SC		(18)	15.4	105.2																
	S-6	30.0	SM		24	21.3			40														
	D-7	35.0	CL		(18)	18.5	105.7	0:28:72		37,16,21		28	900		0.13	0.02	0.0028	+0.1					
	S-8	40.0	CL		39	21.1				44, 16, 28													
	D-9	41.5	CL		(17)																		
	D-10	45.0	CL		(18)	12.8	120.9		53										8.1	256		92	4000
	S-11	50.0	SC/SM		41																		
	D-12	55.0	ML		(25)	20.6	97.2		55	32, 24, 8													
	S-13	60.0	SP-SM		>100				8														
	D-14	65.0	SP-SM		(40)	10.3	107.8		11														
	S-15	70.0	SM		105																		
	D-16	75.0	SP-SM		(40)	13.4	93.4		6														
	S-17	80.0	ML		48																		
	D-18	85.0	SM		(46)	17.4	91.1																
	S-19	90.0	SP-SM		100																		
PE-2	S-1	5.0	CL	Qya	9																		
	D-2	11.0	CL	Qoa	(23)	21.6	108.8																
	S-3	15.0	CL		51																		
	D-4	20.0	CL		(34)	16.0	114.0		69														
	S-5	25.0	CL		51																		
	D-6	30.0	CL		(27)	22.3	104.9	0:6:94		45, 22, 23									8.3	167		2071	1538
	S-7	35.0	SM		43																		
	D-8	40.0	CL		(27)	21.3	92.1			29, 22, 7													
	S-9	45.0	CL		40																		
	D-10	50.5	CL		(85)	12.6	124.2																
	S-11	55.0	SM		97				14														
	D-12	60.0	SW-SM		(92)	15.9	113.9		9														
	S-13	65.3	SM		70				29														
	D-14	70.3	SC		(92)	10.5	110.7																

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 2 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–3	D–1	5.0	CL	Qoa	(12)	18.1	115.1												6.88	98	1.8	25	2500
	S–2	10.0	CL		25																		
	D–3	15.0	CL		(9)	23.1	101.6																
	S–4	20.0	SC/CL		14																		
	D–5	25.0	CL		(29)	15.0	118.5																
	S–6	30.0	CL		32																		
	D–7	35.0	CL		(14)	20.0	101.2																
	S–8	40.0	SM		53				17														
	D–9	45.0	CL		(13)	18.7	103.7		54	24,16,8													
	S–10	50.0	SM		44				26														
	D–11	55.0	CL		(20)	13.1	116.3																
	S–12	60.0	SM		41														5.5	92		24	4167
	D–13	65.0	SM		(24)	14.6	111.1	0:67:33		Nonplastic		34	600										
	S–14	70.0	CL/SC		48																		
	D–15	75.0	SW–SC		(61)	13.6	113.1	6:80:14															
	S–16	80.0	SC		>100																		
	D–17	85.0	SW		(>100)	5.3																	
	S–18	90.0	SW–SM/SM		>100																		
	D–19	95.0	SW–SM/SM		(64)	15.4	115.4																
	S–20	100.0	SM/ML		>100																		
	D–21	105.0	SM		(41)	21.4	97.4																
PE–4	S–1	5.0	CL	Qya	26																		
	D–2	11.0	CL		(27)	23.8	103.0																
	S–3	15.0	CL	Qoa	31																		
	D–4	20.0	CL		(34)	16.7	114.0																
	S–5	25.0	SM		64																		
	D–6	30.0	SM		(32)	16.9	110.8																
	S–7	35.3	SM		91				21														
	D–8	40.2	CL/ML		(60)	26.3	100.7	0:16:84		20,13,7		31	150										
	S–9	45.0	CL		65																		
	D–10	50.0	SC		(83)	10.7	124.6	1:85:14															
	S–11	55.0	SW–SC		>100																		
	D–12	60.0	SW–SC		(>100)	10.0																	

TABLE 3-4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 3 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2168 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-4	S-13	65.0	SP-SM	Qoa	>100																		
	D-14	70.0	SP-SM		(>100)	11.1																	
	D-15	80.0	SP-SM		(>100)	5.8																	
	D-16	85.5	SP-SM		(97)	5.2																	
PE-5	D-1	5.0	CL	Qoa	(15)	15.3	109.3																
	S-2	10.0	SC		49																		
	D-3	15.0	SC		(16)	10.2	115.9																
	S-4	20.0	ML		30																		
	D-5	25.0	CL		(20)	15.6	109.8																
	S-6	30.0	SM		20				46														
	S-7	40.0	SP-SM		>100			23:68:9															
	D-8	45.0	GP		(92)	11.4																	
	S-9	50.0	SM		65																		
	D-10	55.0	SM/GP		(78)	12.7			18										7.26	89		50	2128
	D-11	60.0	SM		(24)	20.7	105.3		45	Nonplastic													
	S-12	65.0	SM		>100				16														
	D-13	70.0	SP/SW		(92)	11.6	115.2																
	S-14	75.0	SP/SW		>100																		
	D-15	80.0	SP/SW		(>100)	19.7	100.9																
PE-6	D-1	5.0	CL	Qoa	(62)	12.0	115.4																
	S-2	10.0	SW		50																		
	D-3	15.0	SM		(16)	16.7	110.1																
	S-4	20.0	SM		75																		
	D-5	25.0	GW-GM		(63)	14.4	122.9	10:83:7															
	D-7	40.0	GW-GM		(>100)	5.8																	
	D-10	58.0	ML		(>100)	23.5	100.7	0:36:64											6.70	243		11	1351
	D-11	60.0	ML		(>100)	27.5	95.9		60	Nonplastic													
	D-12	65.0	SP-SM		(>100)	11.7		16:79:5															
	D-13	70.0	SM		(>100)	23.6	100.8																
	D-14	75.0	SP-SM		(>100)	11.6			5														
PE-7	S-1	6.5	CL	Qya	6																		
	D-2	10.0	SM		(13)	5.7	119.5																
	S-3	15.0	CL		28																		

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 4 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-7	D-4	20.0	CL	Qya	14	16.6	108.5		57	27,15,12													
	S-5	25.0	SW-SM		>100																		
	D-6	30.0	SW-SM		(55)	16.4	113.7	18:76:6											7.37	230		136	4545
	S-7	35.0	ML		35				64														
	D-8	40.0	SM		(60)	12.6	102.0		16														
	S-9	45.0	SP/GP						3														
	D-10	50.0	SM		(>100)	15.2	111.3																
	D-11	55.0	SP/GP		(>100)	10.2	112.4		4														
	B-12	60.0	SP/GP		>100																		
	D-14	70.0	CL	Qoa	(>100)	20.0	111.3																
PE-8	D-1	5.0	CL	Qoa	(22)	13.3	118.9												7.64	94	1.27	43	3703
	S-2	10.0	ML		33																		
	D-3	15.0	SM		(23)	22.2	93.3		26														
	S-4	20.0	SW-SM		75			7:85:8															
	D-5	25.0	CL		(21)	20.4	108.1																
	S-6	30.0	CL		32																		
	D-7	35.0	CL		(32)	19.6	108.7																
	S-8	40.0	CL/ML		24																		
	D-9	45.0	CL/ML		(20)	15.5	116.2	0:34:66		18,13,5	2.73	38	500		0.09	0.01	0.0023	+0.21					
	S-10	50.0	SC/CL		26				49														
	D-11	55.0	SC/CL		(18)	19.3	107.7																
	S-12	60.0	SC/CL		26				53.1										7.35	149		52	3191
	D-13	65.0	ML/SM		(29)	19.2	109.8		51														
	S-14	70.0	SC/CL		85																		
	D-15	75.0	SM		(21)	18.5	111.3																
	S-16	80.0	SP-SM		>100				12														
	D-17	85.0	SM		(>100)	10.8	107.9																
	D-18	90.0	SP		(>100)	13.0																	
PE-9	D-1	5.0	CL	Qoa	(31)	13.1	116.2																
	S-2	10.0	SW-SM		37																		
	D-3	15.0	SM/ML		(17)	18.3	96.4																
	S-4	20.0	SW-SM		47																		
	D-5	25.0	SW-SM		(29)	11.9	109.1																

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 5 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-9	S-6	30.0	SC	Qoa	19				47	30,22,8													
	D-7	35.0	SP-SM		(43)	12.7	98.7		8														
	S-8	40.0	SM		95				21										7.10	98		45	6667
	D-9	45.0	SP-SM		(78)	11.9		7:85:8															
	S-10	50.0	SW-SM		>100			16:77:7															
	D-11	55.0	SW-SM		(64)	15.1	104.4																
	S-12	60.0	SW-SM		>100																		
	D-13	65.0	SC		(34)	10.1	127.1		27														
	S-14	70.0	SP-SM		>100																		
	D-15	75.0	SP-SM		(>100)	17.2	98.9																
	D-16	80.0	SP-SM		(>100)	14.7	103.8																
PE-10	D-1	5.0	SP-SM	Qoa	(14)	7.7	110.2																
	S-2	10.0	ML		39																		
	D-3	15.0	SM		(21)	17.4	93.3																
	S-4	20.0	SP-SM		54																		
	D-5	25.0	ML		(31)	38.6	79.6																
	S-6	30.0	SM		38																		
	D-7	35.0	SM		(>100)	18.9			18														
	D-8	45.0	ML		(>100)			0:4:96		34,28,6		25	1000						6.52	104		22	2000
	S-9	50.0	GW/SW		>100																		
	D-10	61.0	SC		(>100)	18.1	116.4		40														
	D-11	65.0	GP/SP		(>100)	13.4																	
PE-11	S-1	5.0	CL	Qya	54																		
	D-2	10.0	SM		(11)	16.8	96.3																
	S-3	15.0	SW-SM	Qoa	51																		
	D-4	20.3	SW-SM		(64)	16.8	113.3																
	S-5	25.3	SM		34																		
	D-6	30.0	SM		(32)	16.6	93.1		37														
	S-7	35.2	SM		52																		
	D-8	40.5	SW/GW		(>100)																		
	S-9	47.0	CL		42					43,20,23									7.46	215		207	990
	D-10	50.2	SP-SM		(46)	13.6																	
	S-11	55.0	SW/GW		>100			3:79:18															

TABLE 3-4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 6 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-11	D-12	60.4	SM/ML	Qoa	(55)	13.7	119.1		45														
	S-13	65.0	CL		80				61	30,18,12													
	D-14	70.5	SM		(62)	16.8	107.8		44														
	D-16	85.0	GW/SW		(>100)	4.8																	
PE-12	D-1	5.0	CL	Qoa	(36)	16.1	114.4																
	S-2	10.0	SW-SM		85																		
	D-3	15.0	GP		(32)	7.8	135.5																
	S-4	20.0	GP-GM		70																		
	D-5	25.0	SP		(34)	14.7	111.1																
	S-6	30.0	SM		36																		
	D-7	35.0	SP/SP-SM		(35)	11.6	100.8																
	S-8	40.0	CL		32				69	47,22,25													
	D-9	45.0	CL/SC		(59)	9.3	126.4																
	S-10	50.0	SM		>100				14														
	D-11	55.0	SW		(98)	16.9		2:87:11															
	S-12	60.0	SW		100																		
	D-13	65.0	SW		(92)	12.8	112.4		6														
	S-14	70.0	SW		>100			23:69:8											8.88	90		87	2833
	D-15	75.0	SM		(27)	24.8	89.3																
	S-16	80.0	GP-GM		>100			28:62:10															
	D-17	85.0	GP-GM		(>100)	10.6	96.6																
	S-18	90.0	GP-GM		>100				10														
	D-19	95.0	SM		(>100)	7.6	113.4		14														
PE-13	S-1	5.0	CL	Qoa	6														7.8	95	2.8	54	1923
	D-2	10.0	CL		(8)	26.6																	
	S-3	15.0	CL		5			0:33:67		24,14,10													
	D-4	20.0	SC		(12)	15.5			38	25,14,11													
	S-5	25.0	SM/ML		24																		
	D-6	30.0	SM		(55)	16.4	114.3		13														
	S-7	35.0	SM		95			3:83:14															
	D-8	40.2	SM		(46)	19.2	94.4																

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 7 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH USEPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–13	S–9	46.0	GP–GM/SP–SM	Qoa	100			0:72:28															
	D–10	55.5	GP–GM/SP–SM		(120)	12.6	119.0	24:69:7											8.68	92		71	3378
	S–11	65.0	GP–GM/SP–SM		>100																		
	D–12	75.0	SP–SM		(98)	21.2																	
	S–13	80.0	CL		52																		
	D–14	85.0	ML/CL		60	21.8	105.1																
PE–14	D–1	5.0	CL	Qoa	(31)	14.9	117.7																
	S–2	10.0	SM		13																		
	D–3	15.0	SM		(16)	21.8	95.2																
	S–4	20.0	SM		45																		
	D–5	25.0	SM		(22)	16.6	102.8		25														
	S–6	30.0	SM		42				47														
	D–7	35.0	SM		(46)	10.7	113.7																
	S–8	40.0	SM		41																		
	D–9	45.0	SM		(31)	12.6	91.2		38														
	S–10	50.0	SM		90			0:87:13															
	D–11	55.0	SM		(52)	11.8	104.0		35														
	S–12	60.0	SM		45				50										5.75	227		36	1395
	D–13	65.0	CL		(36)	21.5	105.3	0:21:79		29,20,9		26	1600										
	S–14	70.0	CL		37					39,19,20													
	D–15	75.0	SM		(36)	19.0	112.1																
	S–16	80.0	SM		80				25														
	D–17	85.0	SM		(76)	15.4	111.2																
	S–18	90.0	CL		37					47,19,28													
	D–19	95.0	SP–SM		(73)	17.5	109.5																
	S–20	100.0	ML	Tf/Tp	79				89														
PE–15	S–1	5.5	ML/CL	Qya	4																		
	D–2	10.5	CL		(5)	34.9	85.7																
	S–3	15.5	ML		49																		
	D–4	20.0	ML		(21)	28.8	84.1																
	S–5	25.0	ML		68																		

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 8 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2168 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Ce	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-15	D-6	30.0	SP-SM TO ML/SM	Qya	(13)	26.9	91.1																
	S-7	35.0	SP-SM TO ML/SM		60																		
	D-8	40.0	ML	Qoa	(16)	21.1	105.2		68														
	S-9	45.5	ML		67				58	nonplastic													
	D-10	50.0	SM		(23)	25.6	99.1	0:67:33				29	700										
	S-11	54.5	SM		80																		
	D-12	60.0	SP-SM		(23)	27.6	95.8	0:90:10															
	S-13	65.0	CL		63					39,18,21									8.80	99		112	958
	D-14	69.5	CL		(22)	19.4	108.6	0:41:59		47,20,27		24	1700										
	S-15	75.0	SM		100				22														
	D-16	80.0	SW		(23)	15.2	114.5																
	S-17	85.0	SM		73																		
	D-18	90.0	MH	Tf/Tp	(22)	26.3	96.4																
PE-16	D-1	5.0	CL	Qoa	(20)	16.2	114.5																
	S-2	10.0	SC		12																		
	D-3	15.0	ML		(20)	17.8	109.6																
	S-4	20.0	SM/ML		22																		
	D-5	25.0	CL		(20)	25.5	98.8																
	S-6	30.0	SM		26				42														
	D-7	35.0	CL		(25)	18.0	109.3																
	S-8	40.0	CL		38																		
	D-9	45.0	CL/ML		(23)	23.4	101.4																
	S-10	50.0	SM		48				31														
	D-11	55.0	SP-SM		(40)	21.3	103.7		6														
	S-12	60.0	SM		31																		
	D-13	65.0	ML/CL		(25)	33.5	88.4		83														
	S-14	70.0	SM		100				20														
	D-15	75.0	CL		(29)	16.8	114.9	0:41:59		29,15,14		29	1100										
	S-16	80.0	CL		44				79	42,20,22													
	D-17	85.0	ML		(28)	27.9	95.1		99										7.44	113		122	1024
	S-18	90.0	ML/SM		66																		
	D-19	95.0	SM		(55)	25.8	99.6																
	S-20	100.0	CL/SC		38																		

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 9 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–16	D–21	105.0	CL/SC	Qoa	(29)	14.7	119.2																
	S–22	110.0	CL/SC		49																		
PE–17	D–1	5.0	SM/GP–GM	Qoa	(32)	10.9	121.0	5:76:19											7.21	210	1.8	144	2941
	S–2	10.0	SP–SC		71																		
	D–3	15.0	SP–SM		(58)	12.9	115.6	11:77:12															
	S–4	20.0	CL		27																		
	D–5	25.0	ML		(26)	27.1	93.1	0:5:95		34,30,4		30	800										
	S–6	30.0	ML		40				60														
	D–7	35.0	CL/ML		(16)	34.5	87.3																
	S–8	40.0	SM		97				23														
	D–9	45.0	SP		(52)	8.4	96.5	0:95:5				34	650										
	S–10	50.0	SP		105																		
	D–11	55.0	SM		(73)	17.5	93.4		20		2.79								7.03	33		37	9837
	S–12	60.0	SM		72																		
	D–13	65.0	CL	Tf/Tp	(30)	26.4	98.0	0:4:96		48,24,24		30	750		0.11	0.02	0.0017	–0.14					
	S–14	70.0	ML/CL		35																		
	D–15	75.0	ML/CL		(38)	25.8	100.9																
	S–16	80.0	ML/CL		38																		
	D–17	85.0	ML/CL		(29)	25.7	99.6																
	S–18	90.0	ML		44																		
PE–18	S–1	5.0	ML	Qya	2														7.74	109	1.8	73	1538
	D–2	10.0	GW/SW		(22)	8.2	128.9																
	S–3	15.0	GW/SW		24			1:92:7															
	D–4	20.0	GW		(27)	11.0	121.3																
	S–5	25.0	GW/SW		58																		
	D–6	30.0	GW		(55)	6.7	123.4																
	S–7	35.0	GW		>100			20:72:8															
	D–8	40.0	GW		(>100)	11.9	129.3																
	S–9	45.0	GW		>100			59:37:4															
	D–10	50.0	GW		(>100)	7.4																	
	S–11	55.0	SM		>100				20														
	D–12	60.0	SM		(43)	17.0	110.3	0:75:25		28,22,6	2.72				0.07	0.01	0.0014	–0.01					
	S–13	65.0	SM		>100			2:77:21											7.82	97		115	4000

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 10 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2168 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH USEPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–18	D–14	70.0	SW–SM/GW–GM	Qya	(>100)	12.6	123.8																
	S–15	78.5	SM		>100			0:84:16															
	D–16	85.0	SM/ML		(33)	20.1	107.0																
PE–19	S–1	5.0	CL/ML	Qya	44																		
	D–2	10.0	SM		(34)	11.2	119.0																
	S–3	16.0	SW–SM		80			2:88:10															
	D–4	20.0	SW–SM	Qoa	(48)	17.6	109.8																
	S–5	25.0	SM		88				19														
	D–6	29.5	SM		(29)	22.8	95.6																
	S–7	35.0	SM		87			0:73:27															
	D–8	40.0	ML		(31)	30.2	93.9	0:20:80				32	650										
	S–9	45.0	CL	Tf/Tp	38														5.96	120		43	638
	D–10	49.0	CL		(25)	26.2	97.2	0:2:98		38,25,13		32	1200										
	S–11	55.0	CL		49																		
	D–12	59.5	CL		(25)	26.1	96.62																
	S–13	65.0	CL		83				90	37,24,13													
	D–14	70.0	CL		(31)	23.3	101.8																
PE–20	D–1	5.0	CH	Qoa	(3)	34.0	90.1																
	S–2	10.0	CH		5					55,22,33													
	D–3	15.0	CH		(6)	25.4	101.1																
	S–4	20.0	CL		10				79	26,17,9													
	D–5	25.0	ML	Tf/Tp	(20)	28.3	96.5			43,27,16													
	S–6	30.0	ML		26					42,33,9													
	D–7	35.0	ML		(28)	25.9	99.3	0:3:97		42,26,16	2.74	32	600										
	S–8	40.0	ML		27					44,26,18									7.29	113		29	1081
	D–9	45.0	ML		(28)	27.6	95.8																
	S–10	50.0	ML		23																		
	D–11	55.0	ML		(22)	27.1	96.6			46,28,18				64									
	S–12	60.0	ML/MH		24					51,30,21									7.18	113		99	1053
	D–13	65.0	ML/MH		(28)	25.0	98.8			49,31,18		23	2750										
	S–14	70.0	ML		43																		
	D–15	75.0	ML		(28)	24.2	101.0			45,29,16				96									

TABLE 3-4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 11 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2168 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH USEPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-21	D-1	5.0	CL	Qoa	(27)	14.8	117.7												7.6	36	2.3	45	2083
	S-2	10.0	CL		35																		
	D-3	15.0	CL		(19)	14.9	114.9																
	S-4	20.0	SC		>100																		
	D-5	25.0	SM		(29)	18.4	107.8																
	S-6	30.0	SM		83			0:73:27															
	D-7	35.0	CL		(26)	18.6	114.0	0:26:74		24,15,8	2.71	34	450		0.09	0.01	0.0028	+0.02					
	S-8	40.0	CL		44				57	33,15,18									8.44	158		210	12987
	D-9	45.0	SW/GW		(>100)	9.0	130.2																
	S-10	50.2	CL		100				53	38,23,15													
	D-11	55.0	SW		(>100)	12.1	108.6																
	S-12	60.0	SW-SM		>100				12														
	D-13	65.0	SW-SM		(>100)	11.1																	
	S-14	70.0	SW-SM		98			4:85:11															
	D-15	75.0	SW-SM		(>100)	16.5																	
	S-16	80.0	GW/SW		>100																		
	D-17	85.5	SP-SM		(>100)	20.9																	
	S-18	90.0	CH	Tf/Tp	100					55,24,31													
	D-19	95.0	CH		(50)	34.2	88.6		99	61,24,37	2.72				0.16	0.04	0.0021	-0.53					
	S-20	100.0	SW/ML		>100																		
	D-21	105.0	SW/ML		(87)	17.0	104.9																
	D-22	110.0	SW/ML		(>100)	18.4																	
PE-22	D-1	5.0	ML	Qya	(3)	13.1	105.0																
	S-2	10.0	SM		18																		
	D-4	15.0	SW		(25)	8.6																	
	S-5	20.0	SW-SM		63			33:59:8															
	D-6	25.0	SW-SM		(41)	7.4	117.7	6:88:6											6.98	152		62	3571
	S-7	30.0	GP-GM		>100			49:46:5											8.77	200		128	408
	D-8	35.0	GP-GM		(49)	13.7	116.7																
	S-10	40.0	SW-SM		>100			30:57:13															
	D-11	45.0	SW-SM		(76)	11.9	119.7	4:89:7															
	S-12	50.0	CH		38					57,20,37													
	D-13	55.0	SP-SM		(52)	20.0	102.7																

TABLE 3-4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 12 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-22	S-14	60.0	SW-SM	Qya	>100			10:82:8															
	D-15	65.0	SW-SM		(44)	13.8	120.6																
	S-16	70.0	SW-SM		>100			18:74:8															
	D-17	75.0	SP-SM		(>100)	22.3	102.3	1:93:6															
	D-19	85.0	SM		(>100)	11.9																	
PE-23	S-1	5.0	SM	Qya	5			0:62:38															
	D-2	10.5	SM		(14)	8.8	99.7																
	S-3	15.0	SM		50			12:75:13															
	D-4	20.0	SP-SM/GP-GM		(24)	10.2	124.6																
	S-5	25.0	SP-SM/GP-GM		68			10:82:8															
	D-6	30.0	GW		(>100)	4.1																	
	S-7	35.0	SP-SM		>100			16:76:8											9.09	98		61	266
	D-8	40.0	GP/SW		(46)	10.5	127.5	21:74:5															
	S-9	45.5	GP		>100																		
	D-10	50.0	SP		(64)	20.9	100.2																
	S-11	55.0	ML		>100			0:31:69		29,23,6													
	D-12	60.0	SP-SM		(63)	24.2	101.9	0:90:10															
	S-13	65.0	SM		>100			2:82:16															
	D-14	71.0	SW-SM		(93)	15.0	117.2																
	S-15	75.0	SW-SM		>100																		
	D-16	80.0	SW-SM		(90)	14.4	117.2																
	S-17	85.0	GW		>100																		
PE-24	D-1	5.0	SP-SM	Qya	(4)	19.9	90.7		7														
	S-2	10.0	SP		20																		
	D-3	15.0	SP		(23)	10.7	118.6																
	S-4	20.0	SP-SM		68			8:83:9															
	D-5	25.0	SM		(32)	18.2	112.0																
	S-6	30.0	GW		100			20:72:8															
	D-7	35.0	GW		(68)	11.3	117.9																
	S-8	40.0	GW		>100			35:60:5															
	D-9	45.0	GW		(76)	7.7																	
	S-10	50.0	SP-SM		>100			8:81:11															
	D-11	55.0	SP-SM		(98)	20.9	108.7																

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 13 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH USEPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–24	S–12	60.0	SM	Qya	100			0:84:16											8.17	96		76	4000
	D–13	65.0	SM		(78)	19.1	109.1																
	S–14	69.0	ML/SM		68				50	26,23,3													
	D–15	75.0	ML/SM		(46)	25.2	100.6	0:43:57		Nonplastic		33	1250										
	S–16	80.0	SM		>100			19:64:17															
	D–17	85.0	GP		(>100)	5.0																	
PE–25	D–1	5.0	SM	Qya	(10)	18.4	95.6																
	S–2	10.0	ML		4					Nonplastic													
	D–3	15.0	SW		(14)	12.8	118.5																
	S–4	20.0	GP–GM/SP–SM		31			7:84:9															
	D–5	25.0	GP–GM/SP–SM		(24)	9.8																	
	S–6	30.0	GP–GM/SP–SM		63			20:70:10															
	D–7	35.0	GP–GM		(68)	10.9	124.7																
	S–8	41.0	GP–GM		>100			37:56:7															
	D–9	45.0	GP		(>100)	8.8																	
	S–10	50.0	GP		>100			52:43:5															
	D–11	56.0	GP		(61)	8.6	130.5																
	S–12	59.0	CL		34					43,23,20									8.59	112		289	3333
	D–13	65.0	SM/ML		(23)	16.8	115.3	0:51:49		Nonplastic		27	1600										
	S–14	70.0	SP–SM		>100			0:90:10															
	D–15	75.0	SP		(73)	18.3	111.0																
	S–16	80.0	ML		72			0:25:75															
	D–17	85.0	ML		(59)	21.3	104.7																
PE–26	D–2	10.0	ML	Qya	(26)	13.6																	
	S–3	15.0	SP–SM		9																		
	D–4	20.0	SP–SM		(46)	18.0		14:81:5															
	S–5	25.0	GP–GM		>100			55:37:8															
	D–6	30.0	SM		(60)	12.1	115.0																
	D–9	57.0	CL		(58)	23.2	102.1	0:24:76		34,21,13		30	900						7.89	172		87	1887
	S–10	64.0	CL		>100				72	28,20,8													
	D–11	70.0	CL		(46)	26.7	99.6																
	D–12	80.0	SM		(>100)	18.8	104.5	0:85:15															
	D–13	85.0	CL	Tf/Tp	(>100)	21.7	106.2		84	36,24,12													

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 14 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417–B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms–cm)
PE–27	D–1	5.0	SP–SM	Qya	(4)	9.5	98.5		12														
	S–2	10.0	GP–GM/SP–SM		22			33:62:5															
	D–3	15.0	GP/GW		(29)	2.4																	
	S–4	20.0	GP/GW		40																		
	D–5	25.0	GP/GW		(34)	9.9	121.4																
	S–6	30.0	GP/GW		56			23:71:6															
	D–7	35.0	GP/GW		(46)	9.5	122.7																
	S–8	40.0	GP/GW		>100			80:17:3															
	D–9	46.0	SW/GW		(>100)	10.7																	
	S–10	50.0	ML	Tf/Tp	32																		
	D–11	55.0	ML		(33)	25.6		0:3:97		36,29,7		31	1150										
	S–12	60.0	ML		29					47,30,17									7.52	119		183	7692
	D–13	65.0	ML		(36)	24.6				29,26,1				61									
	S–14	70.0	ML		51																		
	D–15	75.0	ML		(30)	24.6				47,27,20				74									
PE–28	D–1	5.0	SM/SC	Qya	(2)	22.0	100.6																
	S–2	10.0	GW		46			30:60:10															
	D–3	15.0	SP		(30)	9.2	129.5	46:51:3															
	S–5	25.0	SP–SM		>100			2:86:12	12														
	D–6	30.0	SP		(58)	14.5	116.4																
	B–7	36.5	GW/GP		>100																		
	D–9	45.0	CL/CH	Tf/Tp	(40)	31.7	91.6			50,20,29		29	700										
	S–10	50.0	ML		>100					29,23,6													
	D–11	55.0	ML		(35)	25.5	100.1			38,26,12				54									
	B–12	62.0	CL		>100																		
	D–13	65.0	CL		(60)	22.0	102.8			42,22,20				34									
	S–14	70.0	CL		100					49,23,26									7.71	777		701	1333
	D–15	75.0	CL		(70)	23.9	101.2	0:2:98		42,25,17		31	1250										
	S–16	80.0	CL		>100																		
PE–29	D–1	5.0	SM	Qya	(17)	17.7	106.4																
	S–2	10.0	SM		14																		
	D–3	15.0	GP/GW		(16)	5.0																	
	S–4	20.0	GP/GW		62			2:77:21															

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 15 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Ce	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-29	D-5	25.5	GP/GW	Qya	(>100)																		
	S-6	32.0	SP-SM		>100			20:69:11															
	D-7	35.0	GP		(73)	7.0	128.0																
	S-8	41.0	GP		>100			95:4:1															
	D-9	45.0	GP-GM		(34)	9.0	123.6																
	S-10	50.0	GP		>100																		
	D-11	55.0	GP-GM		(>100)	8.8	98.2																
	S-12	61.0	SW-SM/GP-GM		>100			21:68:11											2.37	203		1645	625
	D-13	65.0	SW-SM/GP-GM		(68)	11.6	124.7																
	S-14	70.0	CH	Tf/Tp	58					53,22,31													
	D-15	75.0	CH		(30)	30.9				54,22,32				120									
	S-16	80.0	CH		59																		
PE-30	S-2	10.0	SP-SM/GP-GM	Qya	22			22:71:7															
	D-3	15.0	SP-SM/GP-GM		(17)																		
	S-4	25.0	GP-GM		>100																		
	D-5	30.0	SP		(77)																		
	D-9	50.0	GW-GM		(67)	16.3		0:89:11															
	S-10	55.0	GW-GM		>100														6.74	674		163	1220
	S-14	75.0	CH/MH	Tf/Tp	58					69,33,36													
	S-15	80.0	CH/MH		>100																		
PE-31	D-1	5.0	SP	Qya	(7)	14.8	90.3																
	S-2	10.0	GP/SP		20																		
	D-3	15.0	GP/SP		(21)	10.7	121.7																
	S-4	20.0	GP/SP		26																		
	D-5	25.0	GP/SP		(40)	25.3		85:13:2															
	S-6	30.0	GP/SP		66																		
	D-7	35.0	GP/SP		(73)	8.8																	
	S-8	40.0	GP/SP		>100																		
	D-9	45.0	GP/SP		(>100)	13.4	119.0																
	S-10	50.0	GP/SP-SM		69			41:51:8															
	D-11	55.0	SM		(46)	17.2	111.3	0:79:21															
	S-12	60.0	ML/CL		>100																		
	D-13	65.0	ML/SM/SP		(23)	27.3	99.7	0:44:56		Nonplastic		31	950										

TABLE 3–4. SUMMARY OF LABORATORY TEST RESULTS (PAGE 16 OF 16)

Boring No.	Sample No.	Depth (ft)	USCS/Visual Soil Classification ASTM D 2487/D 2488	Geological Unit	Equivalent SPT Value	Moisture Content ASTM D 2216 (%)	Dry Density ASTM D 2937 (pcf)	Grain Size Distribution ASTM D 422 Gr:Sa:Fi (%)	Percent Passing #200 Sieve ASTM D 1140 (%)	Atterberg Limits ASTM D 4318 LL,PL,PI (%)	Specific Gravity ASTM D 854	Direct Shear (Peak Strength) Friction Angle (degrees)	Direct Shear (Peak Strength) Cohesion (psf)	Unconfined Compressive Strength ASTM 2166 (psi)	Consolidation Characteristics Cc	Consolidation Characteristics Cs	Consolidation Characteristics Cx	Swell/ Collapse (%)	pH US EPA Method 9045	Chloride Content DOT CA Test 422 (ppm)	Sulfides Content USEPA Method 9030 (mg/kg)	Sulphate Content DOT CA Test 417-B (ppm)	Electrical Resistivity DOT CA Test 532 (ohms-cm)
PE-31	S-14	75.0	ML/CL	Tf/Tp	50																		
	S-15	80.0	ML/CL		59																		

NOTES

1) For California Drive Samples, Equivalent SPT values were obtained by applying the appropriate corrections for different hammer weights, hammer drop, sampler dimensions, and buoyancy and viscous drag within the drilling mud. Equivalent SPT values corrected from drive sampler blowcounts are shown in parantheses
2) Equivalent SPT values in alluvium may not be representative of material density/consistency due to the presence of gravels, cobbles and boulders
3) Since gravels larger than the sampler diameter were present in layers classified as gravel, clayey sand, silty gravel, gravel with sand and sand with gravel, results of gradation tests, fines content (% passing #200 sieve), insitu moisture content and insitu dry density tests for these these materials may not be truly representative
4) Cc, Cs, and Cx are based on vertical strain – log stress plots
5) Electrical resistivity tests (DOT CA 532) were done at in-situ moisture content, however, for near surface samples the test was repeated under saturated conditions also, and the corresponding values are reported in Appendix B, Table B-6

TABLE 3-5. CHEMICAL LABORATORY TEST PROGRAM

Test Type	Number of Tests	Test Procedure
Total Petroleum Hydrocarbons (with carbon chain)	7	EPA 8015
Aromatic Volatile Organic Compounds (BTEX)	7	EPA 8020
Volatile Organic Compounds	7	EPA 8240
Semi-volatile Organic Compounds	7	EPA 8270
Inorganic Analysis (Arsenic)	4	EPA 7060
Inorganic Analysis (Selenium)	4	EPA 7740
Inorganic Analysis (Mercury)	4	EPA 7470
Inorganic Analysis (Dissolved CAM Metals)	3	EPA 7471
Inorganic Analysis (Title 22 TTLC Metals by ICP)	4	EPA 6010
Sulfide	7	EPA 376.1, 376.2
Sulfate and Chloride	7	EPA 300, 325.3, 375.3

TABLE 3-6. SUMMARY OF ANALYTICAL TESTS ON GROUNDWATER SAMPLES

Sample No.	Volatile Organics (μg/L)		Semi-volatile Organics (μg/L)		Metals (mg/L)		Total Sulfides (mg/L)	Total Petroleum Hydrocarbons (mg/L)	Chloride (mg/L)	Sulfate (mg/L)
	Compound (Concentration)	Threshold Level[1]	Compound (Concentration)	Threshold Level[1]	Compound (Concentration)	Threshold Level[1]				
PE-8	Benzene (7,450)	1	ND[2]		Barium (0.13)	1.0	ND[2]	0.33	86.9	174
	Toluene (780)	100[3]	-		Zinc (0.15)	2.0				
	Ethylbenzene (520)	680	-		Vanadium (0.007)	0.02[4]				
	Xylenes (1,070)	1,750								
	Acetone (140)	700[5]								
PE-16	ND[2]		Dimethylphthalate[6] (12)	313,000[7]	Arsenic (0.0046)	0.05	ND[2]	ND[2]	166	351
			bis (2-Ethylhexyl)[6] phthalate (12)	4	Barium (0.076)	1.0				
					Zinc (0.09)	2.0				
PE-18	ND[2]		ND[2]		Barium (0.1)	1.0	ND[2]	ND[2]	161	232
					Copper (0.01)	1.0				
					Selenium (0.0059)	0.01				
					Zinc (0.07)	2.0				
PE-23	ND[2]		ND[2]		Barium (0.11)	1.0	ND[2]	ND[2]	168	303
					Molybdenum (0.011)	0.04[4]				
					Selenium (0.0026)	0.01				
					Zinc (0.028)	2.0[4]				
PE-29	Benzene (250)	1.0	Phenol (16)	4,000[4]	Barium (1.24)	1.0	150	8.4	280	25
	Toluene (54)	100[3]	4-methylphenol (13)	35[5]						
	Ethylbenzene (920)	680	Naphthalene (600)	20[4]						
	Xylenes (320)	1,750	2-methylnapthalene (12)	NAL[8]						
	Vinyl Acetate (24)	NAL[8]	Acenaphthene (33)	20[9]						
	Vinyl Chloride (270)	0.5								

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TABLE 3-6. SUMMARY OF ANALYTICAL TESTS ON GROUNDWATER SAMPLES

Sample No.	Volatile Organics (μg/L)		Semi-volatile Organics (μg/L)		Metals (mg/L)		Total Sulfides (mg/L)	Total Petroleum Hydrocarbons (mg/L)	Chloride (mg/L)	Sulfate (mg/L)
	Compound (Concentration)	Threshold Level[1]	Compound (Concentration)	Threshold Level[1]	Compound (Concentration)	Threshold Level[1]				
PE-30	Benzene (45)	1.0	Naphthalene (2,900)	20[4]	Barium (0.684)	1.0	54	10	241	246
	Toluene (210)	100	2-methylnapthalene (26)	NAL[8]	Zinc (0.104)	2.0				
	Ethylbenzene (1,200)	680	Acenaphthene (18)	20[9]						
	Xylenes (950)	1,750								
	cis-1,2-dichloroethene (5)	6								
PE-31	Benzene (130)	1.0	ND[2]		Barium (0.0765)	1.0	10.5	ND[2]	191	361
	Toluene (2.4)	100								
	Ethylbenzene (5.4)	680								
	Xylenes (4.5)	1,750								
	cis-1,2-dichloroethene (100)	6								
	Trichloroethene (3)	5								
	Vinyl chloride (13)	0.5								

Notes: (1) California Department of Health Services (CDHS) Maximum Contaminant Level (MCL) for Drinking Water.
(2) ND = Not Detected
(3) California Department of Health Services (CDHS) Action Level (MCL) for Toxicity
(4) Suggested No Adverse Response Level (SNARL) for toxicity other than cancer risk per EPA
(5) U.S. EPA Integrated Risk Information System (IRIS) reference dose as a water quality criterion
(6) Common residual laboratory contaminant detected at low levels, hence not considered significant.
(7) U.S. EPA National Ambient Water Quality Criteria for Health and Welfare Protection (Non-Cancer Public Health Effects)
(8) NAL = No published action level
(9) U.S. EPA National Ambient Water Quality Criteria for Health and Welfare Protection (Taste and Odor or Welfare)

μg/L - micrograms per liter, mg/L - milligrams per liter

3.3 FIELD OBSERVATION AND MONITORING OF H_2S

As indicated on the boring logs in Appendix A, sulfurous odors were noted during the drilling of PE-28, PE-29, PE-30 and PE-31. During development of the Monitoring Wells PE-29, PE-30 and PE-31, sulfur odors were again noticed. The air space immediately above water samples taken from these wells was monitored using a multiple gas indicator that indicated measurable hydrogen sulfide (H_2S) concentrations of 46.0, 2.9 and 11.5 parts per million (ppm) for Monitoring Wells PE-29, PE-30 and PE-31, respectively. The results are discussed in Section 5.4.

Section 4.0

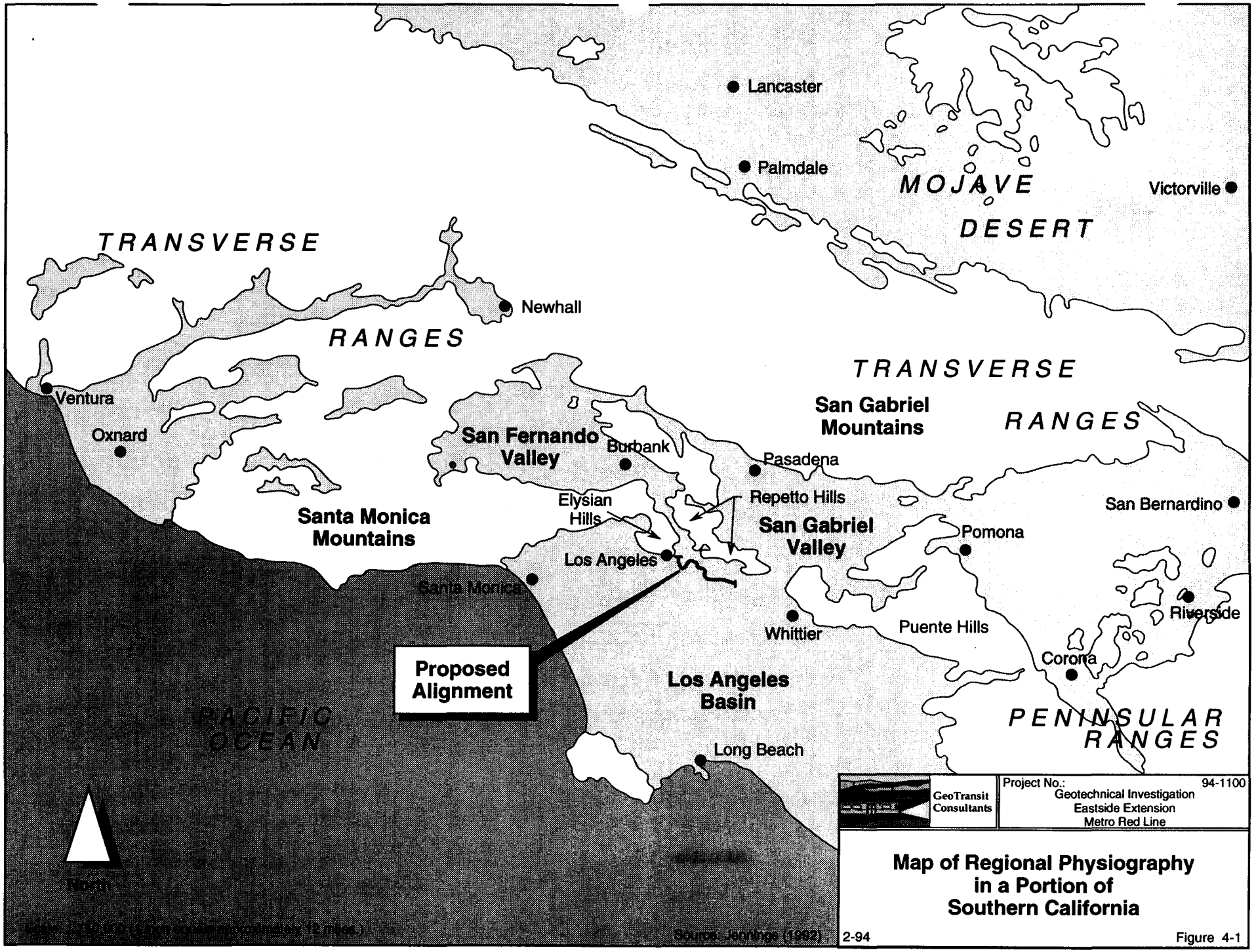
Section 4.0

4.0 GEOLOGIC AND GROUNDWATER CONDITIONS

4.1 REGIONAL GEOLOGIC SETTING

The proposed Metro Rail alignment through the Repetto Hills is on the northern edge of the Los Angeles coastal plain and the underlying structural basin, at the junction between the Transverse Range and Peninsular Range geomorphic provinces in Southern California (Figure 4-1). The Elysian and Repetto Hills in central and eastern Los Angeles are a northwest extension of the Peninsular Ranges trending northwest from Baja California. The east-west oriented San Gabriel, Verdugo and Santa Monica Mountains to the north of the hills are in the western part of the Transverse Ranges, which extend across Southern California from the Colorado Desert to Point Arguello. The Peninsular Ranges are largely defined by right-lateral strike-slip faulting and associated folding parallel to their trend, and the western Transverse Ranges are uplifted by northward-dipping thrust faults along their southern margin. The hilly terrain of the study area appears to result from folding and faulting in a zone of convergence between these major sets of structures.

Bedrock units of the mountainous areas predate structural development of the Los Angeles basin and the present structural setting. They consist of a wide variety of Precambrian to Mesozoic igneous and metamorphic basement rocks and a partial cover of Mesozoic to early Tertiary sedimentary and volcanic strata. Tertiary marine sediments and lesser volcanic rocks that were deposited in the developing Los Angeles basin during Miocene and Pliocene time compose much of the folded and faulted, northwest-trending hills of the present coastal plain. The oldest strata exposed in the southern and western Repetto Hills near the proposed alignment are those of the Puente Formation, which consists primarily of siltstone, claystone and sandstone. Puente Formation strata are conformably overlain by deposits of the Pliocene-age Fernando Formation, which generally grade upward from siltstone near the base to conglomerate near the top. This unit apparently records the final episode of marine deposition in the Los Angeles Basin, before the coastal plain was elevated above sea level.


Lancaster
Palmdale
MOJAVE
DESERT
Victorville
TRANSVERSE
RANGES
Newhall
Ventura
Oxnard
San Fernando
Valley
Burbank
San Gabriel
Mountains
TRANSVERSE
RANGES
Pasadena
Repetto Hills
Elysian
Hills
Santa Monica
Mountains
San Gabriel
Valley
San Bernardino
Pomona
Los Angeles
Santa Monica
Whittier
Puente Hills
Riverside
Corona
Proposed
Alignment
Los Angeles
Basin
PACIFIC
OCEAN
PENINSULAR
RANGES
Long Beach
North
Source: Jennings (1992)
GeoTransit
Consultants
Project No.: 94-1100
Geotechnical Investigation
Eastside Extension
Metro Red Line
Map of Regional Physiography
in a Portion of
Southern California
2-94
Figure 4-1

Deformation of Miocene and Pliocene marine deposits in the Repetto Hills has been accompanied during Pleistocene time by deposition of alluvium from the Transverse Ranges to the north. Cycles of alluvial deposition, continued deformation, and partial erosion have left a fringe of uplifted and dissected alluvial fans and terraces on the flanks of the hills. Much of the proposed course of the Metro Rail tunnel will be in these materials.

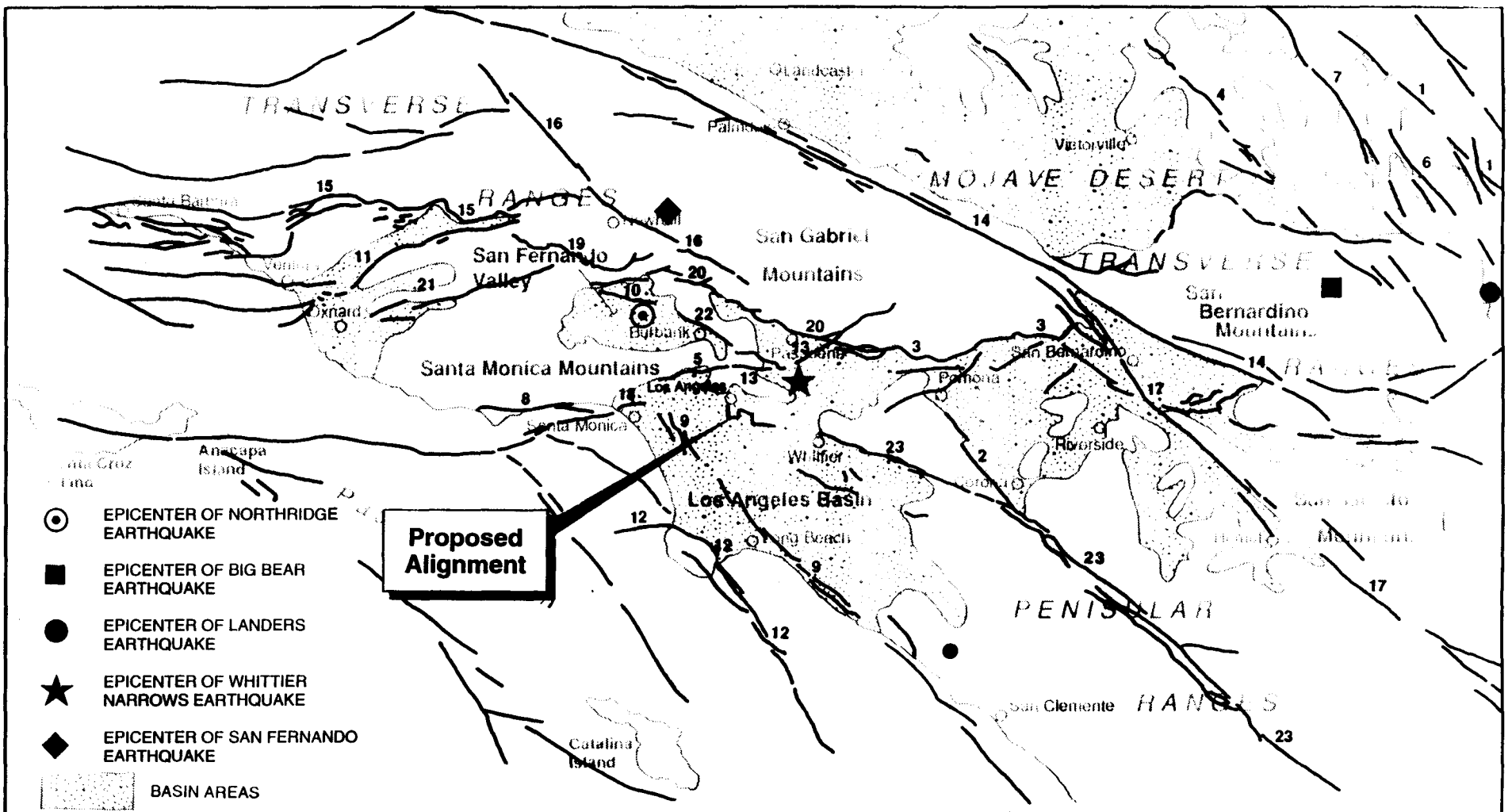
There is current debate among geologists about the geologic structure and ongoing tectonic activity in the Repetto and Elysian Hills. Speculation in the wake of the 1987 Whittier Narrows earthquake suggests that a northeast- to north-dipping extension of faulting beneath the northwest trend of the Whittier fault has produced thrust-fault offsets of well-consolidated bedrock at depth that are expressed in the weaker near-surface materials by folding, faulting or a combination of the two comprising the Repetto and Elysian Hills.

4.2 REGIONAL FAULTING AND SEISMICITY

4.2.1 Faulting

The proposed alignment is located in a high seismic-potential area that has experienced ground shaking from numerous large earthquakes in historical time. The earthquakes are being generated by periodic slip across the northwesterly-trending strike-slip San Andreas and Peninsular Ranges fault systems and on the generally east-west trending thrust faults of the Transverse Ranges.

Figure 4-2 shows the known major active and potentially active faults in the greater Los Angeles area. According to the California Division of Mines and Geology (CDMG), the term "active" applies to any fault that has moved within Holocene time (i.e., the past 11,000 years). Such activity is recognized by displacement of Holocene-age sediments or by direct association with seismic activity. The term "potentially active" applies to a fault that has been active during Pleistocene time (i.e., the past 2 to 3 million years preceding the Holocene). Such faults may have remained active during Holocene time, but direct geologic evidence for continued activity



Source:
Jennings, 1992 and Ziony and Yerkes, 1985.

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Map of Major Faults in a Portion of Southern California

2-94

Figure 4-2

is not available. The CDMG does not specifically define an inactive fault, although they do indicate that a fault may be presumed to be inactive based on "direct geologic evidence" of inactivity during the past 11,000 years or longer (Hart, 1990).

The closest documented active faults to the alignment are the east-west trending Hollywood-Santa Monica and Raymond faults. The Hollywood-Santa Monica fault is located at the southern base of the Santa Monica Mountains about five miles northwest of the alignment. The Raymond fault passes through the northern part of the Repetto Hills into the south Pasadena-San Marino area to the east, and is about five miles north of the alignment at its closest point. A fault that is postulated to be the extension of the Whittier fault to the northeast of the Montebello and Monterey Park Hills area is located approximately four miles northeast of the alignment (Treiman, 1991; Bullard and Lettis, 1993). Other active and potentially active faults that are within 30 miles of the alignment are listed in Table 4-1 together with the San Andreas fault, which has been included in the Table for comparative purposes.

In addition to the fault traces that are shown in Figure 4-2, topographic features having tectonic origins have been identified in the vicinity of the alignment (Plate 1A). An east-west-trending linear escarpment in alluvium that crosses the alignment at two locations (approximate Stations 108+00 and 154+00), probably coincides with the "Coyote Pass fault" as mapped by the California Department of Water Resources (1961). Several investigators have recently interpreted the escarpment to be a tectonic feature related to surface faulting associated with a postulated buried thrust fault system within this part of the Los Angeles basin (Bullard and Lettis, 1993; Sieh, 1993; Dolan and Sieh, 1992a and 1992b; Davis and others, 1989). Our subsurface evaluation of the escarpment and its geologic significance for the tunnel alignment are discussed in Sections 4.4.4 and 6.7.

TABLE 4-1. ESTIMATED SEISMIC CHARACTERISTICS OF PRINCIPAL FAULTS

Fault	Approximate Distance from Alignment[1] (miles)			Magnitude of Maximum Credible Earthquake[2]	Age of Most Recent Displacement[3]
	West End	Center	East End		
Chino	30	27	24	7 1/2	Late Quaternary
Cucamonga	31	29	27	7	Holocene
Hollywood	5	7	9	7 1/2	Holocene
Malibu Coast	22	24	27	7 1/2	Holocene
Newport-Inglewood	8	9	10	7	Historic (1933)
Northridge	20	23	26	7 1/2	Late Quaternary; Holocene
Palos Verdes Hills	18	18	19	7	Late Quaternary; Holocene
Raymond	5	5	7	7 1/2	Holocene
San Andreas	33	33	33	8	Historic (1857)
San Gabriel	16	16	16	7 1/2	Late Quaternary; Holocene
Santa Monica	9	12	15	7 1/2	Late Quaternary; Holocene[4]
San Fernando	16	18	20	7 1/2	Historic (1971)
Sierra Madre	11	12	12	7 1/2	Late Quaternary; Holocene
Verdugo	8	10	12	6 3/4	Late Quaternary; Holocene
Whittier	8	5	4	7 1/2	Late Quaternary; Holocene

(1) Distance measurements are based on fault traces shown in Jennings (1992) and Treiman (1991).
(2) Maximum Credible Earthquake Magnitudes from Mualchin and Jones (1992).
(3) Age of Most Recent Displacement from Jennings (1992) except where noted; multiple ages apply to separate fault segments; "Late Quarternary" is the past 700,000 years; Holocene is the past 11,000 years.
(4) Dolan and Sieh (1992a).

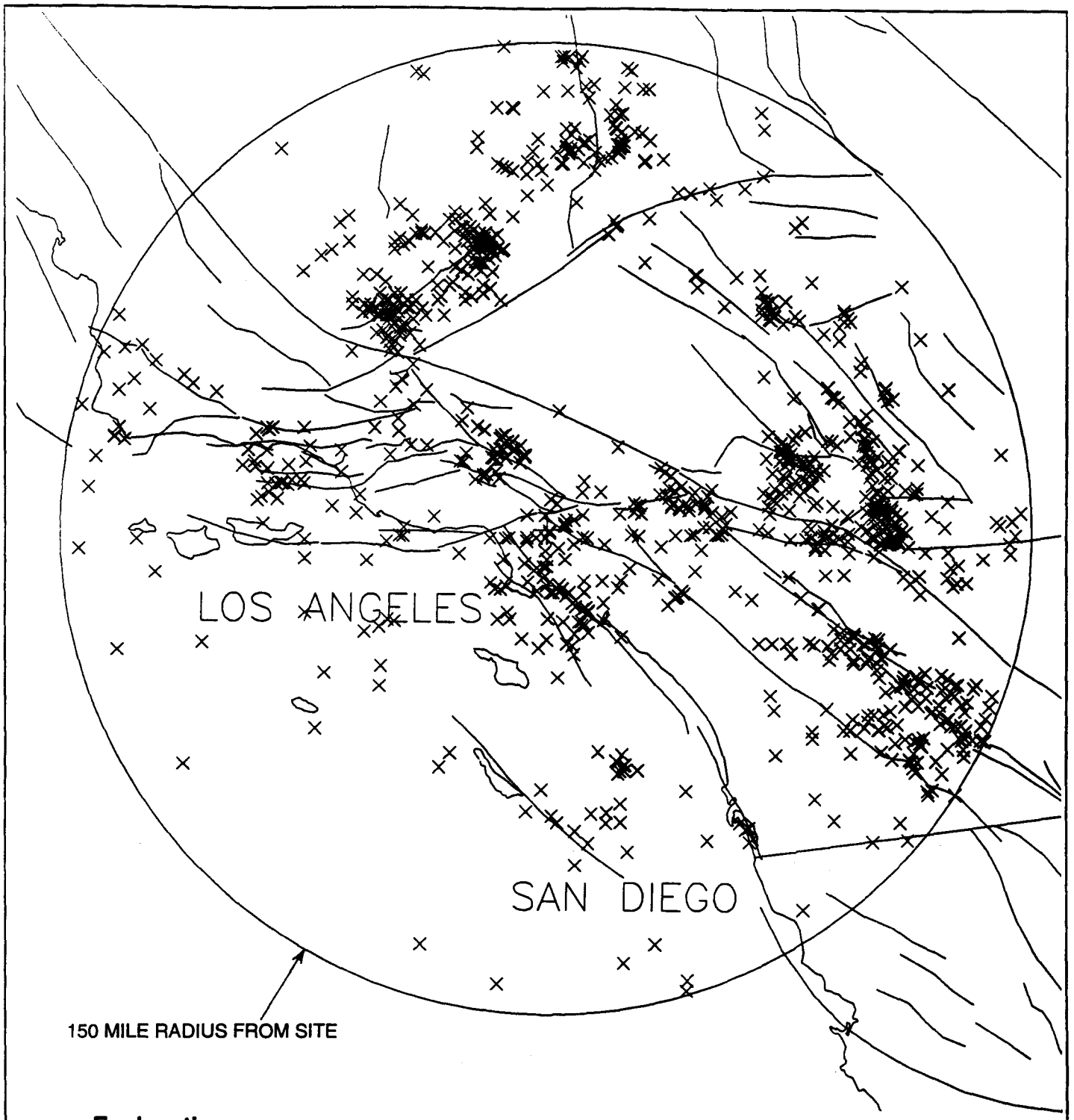
4.2.2 Seismicity

Moderate to large earthquakes can be expected to occur in the site region during the life of the project. In the event that a nearby fault were to slip and produce a major earthquake, very strong ground motions could affect the alignment.

An earthquake computer search (Blake, 1992) was performed to locate historical earthquake epicenters with respect to the alignment. A search radius of 150 miles from the approximate mid-point of the alignment was selected in order to include the larger magnitude earthquakes that have occurred in Southern California. Catalogued earthquakes since the year 1800 with magnitudes ranging from 4 to 8+ are shown in Figures 4-3 and 4-4. The largest historical event was the 1857 Fort Tejon earthquake (estimated M 7.9) on the San Andreas fault, about 125 miles northwest of the proposed alignment. The epicenter of the closest moderate-sized historical earthquake was that of the 1987 Whittier Narrows earthquake (M 5.9), with an epicenter about 6.5 miles east-northeast of the approximate center of the alignment. This earthquake occurred on a previously unknown northeast-dipping buried thrust fault that has since been named the Elysian Park seismic zone (Mualchin and Jones, 1992). More recently, a M 6.6 earthquake occurred on January 17, 1994 on a previously unknown buried thrust fault dipping south beneath the alluvium of the San Fernando Valley. The epicenter of this earthquake was about 24 miles northeast of the alignment. Early records of ground accelerations released by the California Division of Mines and Geology for a strong ground motion instrument at City Terrace indicates maximum free field accelerations of 0.32g horizontal and 0.13g vertical for the January 17, 1994 earthquake.

4.3 REGIONAL HYDROGEOLOGY

The hydrogeology of the greater Los Angeles area includes two general types of groundwater regimes: bedrock uplands and alluvial lowland basins. The bedrock uplands surrounding most of the basins are generally considered to be non-water bearing, while adjacent alluvial basins



Explanation:

× M = 4.0–4.9

Site Location(+):

Latitude - 34.0340 N
Longitude - 118.1920 W

0 50 100 Miles
Scale

North

Source:
Epicenters from Blake, 1992.

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Magnitude 4.0 - 4.9 Earthquakes in Southern California, 1800 - 1993

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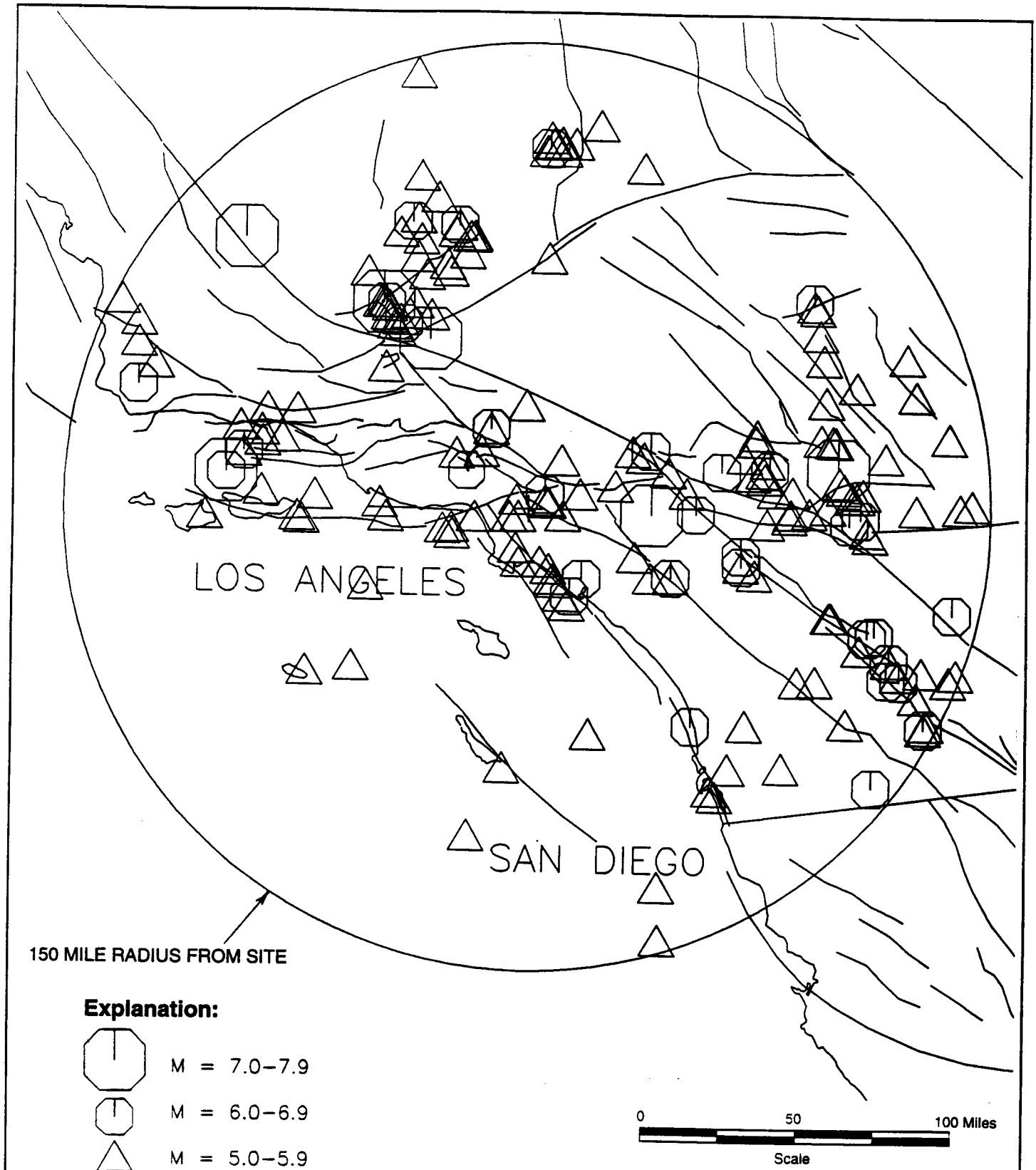
Figure 4-3



Explanation:

M = 7.0–7.9

M = 6.0–6.9

M = 5.0–5.9

Site Location(+):

Latitude - 34.0340 N
Longitude - 118.1920 W

North

0 50 100 Miles

Scale

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Magnitude 5 and Greater Earthquakes in Southern California, 1800 - 1993

Source:
Epicenters from Blake, 1992.

2-94

Figure 4-4

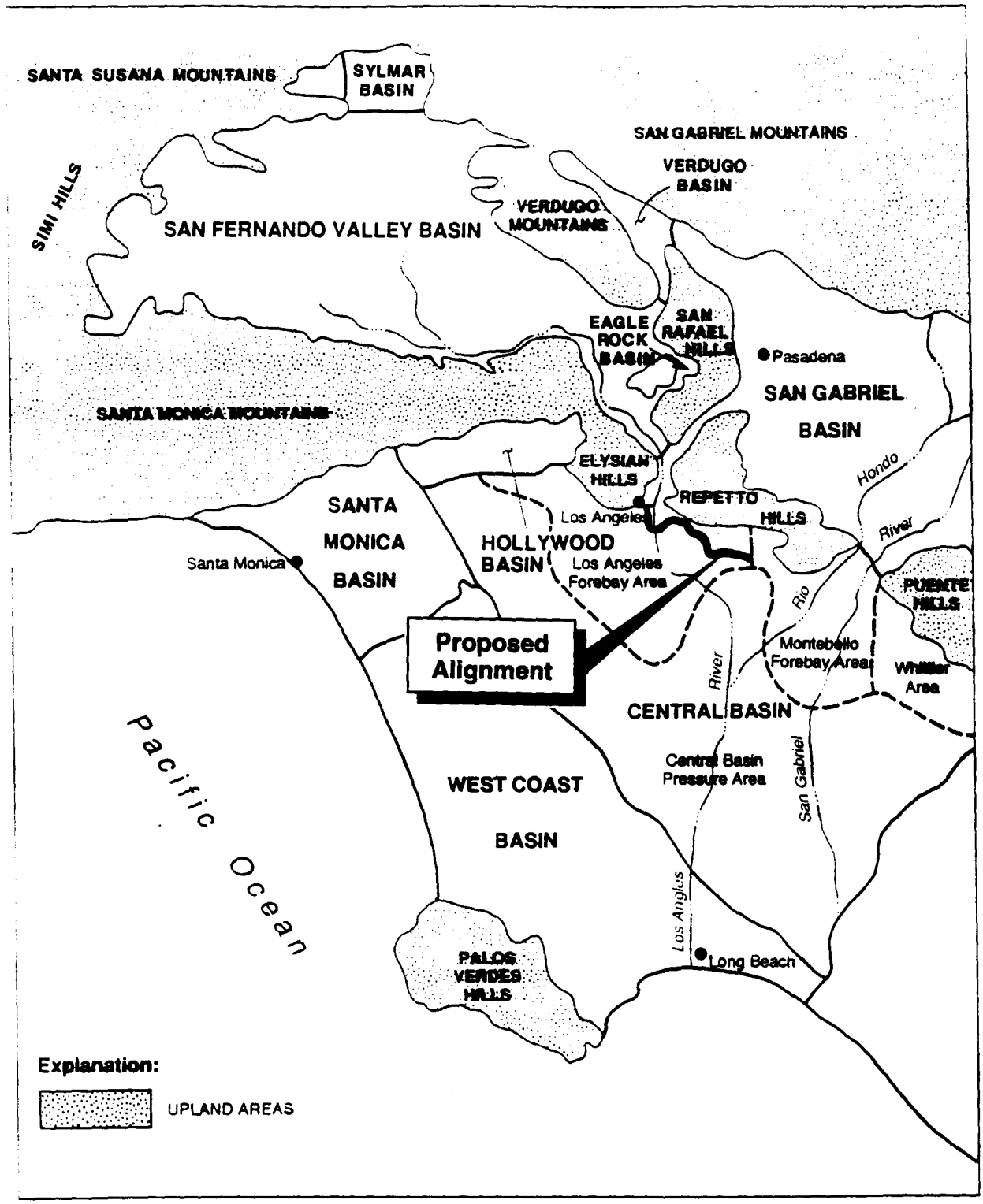
supply groundwater that has been extensively used for domestic, commercial and agricultural purposes.

The California Department of Water Resources (CDWR; 1961) divides the Los Angeles coastal plain into the Santa Monica, Hollywood, Central, and West Coast groundwater basins (Figure 4-5). The Central Basin of the coastal plain is further subdivided into the Los Angeles Forebay, Montebello Forebay, Whittier and Central Basin Pressure Areas. The Eastside Extension alignment lies entirely within the Los Angeles Forebay Area.

The Los Angeles Forebay Area extends southward from the narrows of the Los Angeles River and has been characterized by the CDWR as an area of unrestricted infiltration of surface water. Because of the presence of low permeability sediments in the shallow bedrock of the Repetto and Elysian Hills, however, the actual area of effective surface water infiltration to underlying aquifers is largely restricted to the younger and older alluvial deposits in the vicinity of the narrows.

Groundwater in the Los Angeles Forebay Area occurs in young alluvium and in older permeable Pleistocene sediments. Some limited groundwater also may be present in Pliocene and Miocene bedrock underlying these deposits. According to the CDWR (1961), the water-bearing sediments extend to depths on the order of 1,600 feet below the ground surface, particularly in the southern portions of the Forebay Area.

Aquifers underlying the Forebay Area in the vicinity of the tunnel alignment include the Semiperched, Gaspur, Exposition, Gage and Gardena aquifers at increasing depths in the Holocene and Pleistocene sediments (CDWR, 1961). Most of these aquifers underlie the eastern portion of the alignment. However, because bedrock occurs at relatively shallow depths along the western half of the alignment, only the upper Semiperched and Gaspur aquifers appear to be present in that area (CDWR, 1961). The semiperched aquifer is comprised of the older Pleistocene deposits overlying bedrock near the Repetto and Elysian Hills; the Gaspur aquifer is largely comprised of the coarse-grained Holocene deposits overlying bedrock in the Los



0 10 20 Miles

Scale

North

Source:
California Division of Water Rescources, 1961.

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Eastside Extension
Metro Red Line

Map of Groundwater Basins in the Los Angeles Area

2-94

Figure 4-5

Angeles River Narrows. The aquifers are generally separated from each other by aquicludes, but the aquiclude materials may be locally absent in the northern part of the Forebay Area, allowing hydraulic continuity between aquifers.

4.4 LOCAL GEOLOGIC CONDITIONS

Unconsolidated to weakly consolidated Pleistocene and Holocene alluvial sediments, and consolidated bedrock of the Miocene and Pliocene age Puente and Fernando formations will be encountered during construction of the alignment. Most of the borings and all cone penetrometer tests for this investigation were entirely in the alluvial sediments. Fourteen borings in the western portion of the alignment penetrated through the alluvium into underlying bedrock. Surficial geologic conditions in the vicinity of the alignment are shown in Plate 1A, which also shows the locations of exploration points (e.g. borings) for this investigation as well as selected exploration points from other investigations that are located along or near the alignment. Plates 2A and 2B illustrate the subsurface conditions along the alignment based on the results of the exploratory borings.

4.4.1 Local Topographic Conditions

The proposed alignment begins in the west near Union Station in the pre-channelization flood plain of the Los Angeles River. Where it trends northeastward and crosses the U.S. 101 freeway, the alignment leaves the gently south-sloping floodplain and crosses a series of low alluvial terraces that form the southwestern margin of the City Terrace area in the Repetto Hills. The southwest- to south-sloping terraces are somewhat incised by local drainage channels and appear to be offset by an arcuate escarpment that extends eastward from near the proposed station at First Street and Boyle Avenue.

The eastern portion of the alignment traverses wide drainage channels and the lowest alluvial terraces in the series to the proposed station at Whittier Boulevard and Arizona Avenue, where it enters an extensive alluvial fan surface that slopes southward from the Repetto Hills.

4.4.2 Surficial Deposits

A variety of surficial alluvial deposits underlies the entire alignment. These deposits are differentiated on Plate 1A into two units: Old Alluvium (map symbols Qoa and Qp) of Pleistocene age, and overlying Young Alluvium (Qya) of Holocene age. Most of the alignment will be through Old Alluvium, which begins at the eastern edge of the Los Angeles River Narrows at about Station 93+00. The alignment will be mostly in Young Alluvium within the narrows itself. Young Alluvium also occurs locally along the alignment, within drainage courses eroded into the Old Alluvium, and overlying Old Alluvium on the alluvial fan surface at the east end of the alignment.

Several fluvial terrace surfaces that range in age from middle Pleistocene to Holocene have been identified on the alluvial deposits (Bullard and Lettis, 1993). For the Old Alluvium, these are designated on the geologic map (Plate 1A) by a numerical subscript on the map symbol (e.g. Qoa_1). The configuration of these surfaces indicates that the alluvial deposits south of the Repetto Hills area are being actively tectonically deformed and uplifted (Bullard and Lettis, 1993).

Both granular and fine-grained intervals occur in the young and old alluvial units. Within the Los Angeles River Narrows, granular young alluvial deposits are most common. The sediments there consist largely of sand and gravel with interbedded lenses of gravel, cobbles and boulders. The largest clasts range up to 4 feet in size (Converse Consultants, 1981) with intervals of coarse gravel to large cobbles frequently present. The clasts are primarily composed of granitic and metamorphic rock types and are unweathered and durable. Locally, the base of the alluvium in the Narrows area is characterized by a zone of boulders and cobbles overlying the bedrock (Converse and others, 1984). This condition was found in Borings PE-28 and PE-29, where clasts up to 3 feet in size were encountered.

The granular deposits in the Old Alluvium generally appear to be finer grained than the Young Alluvium in the Narrows area. The deposits primarily consist of sands and gravels with varying

percentages of silt and/or clay. Cobbly zones and possibly some small boulders were encountered in Borings PE-5, PE-6, PE-7, PE-10, PE-11 and PE-21. Drilling through cobbly and bouldery zones was usually accompanied by strong rig chatter and slow progress. As with the Young Alluvium, clasts are composed mostly of granitic and metamorphic rock types.

Intervals of fine-grained strata generally consisting of up to 20-foot thick silt and clay deposit mixed with variable amounts of fine to coarse sand and some gravel are frequently interbedded in the Old Alluvium. Fine-grained beds are relatively uncommon in the Young Alluvium .

4.4.3 Bedrock

Bedrock strata of the Pliocene Fernando Formation crop out both to the north of the alignment in the City Terrace area of the southern Repetto Hills, and to the northwest of the Los Angeles River Narrows along the south base of the Elysian Hills. The Fernando Formation typically consists of massive to indistinctly bedded siltstone or mudstone and well bedded sandstone (Lamar, 1970). The older Miocene Puente Formation is exposed to the north of the Fernando Formation exposures and underlies much of the Elysian and Repetto Hills. The Puente Formation consists of well-bedded siltstone, claystone and very fine sandstone (Lamar, 1970). In the area shown in Plate 1A, the contact between these formations is covered by alluvium over most of its length. Where exposed, the contact is often difficult to locate accurately because the lithologic change between the formations can be gradational (Lamar, 1970). We have therefore not attempted to differentiate the Fernando Formation from the Puente Formation during subsurface investigations; i.e. when referring to bedrock information obtained from borings, bedrock is designated the Fernando/Puente Formation in the text, and on the boring logs by a dual symbol, Tf/Tp.

Bedrock was encountered at various depths in 14 borings along the western half of the alignment (generally west of Station 200+00). Bedrock should be anticipated to periodically occur in the tunnel envelope to the west of Station 140+00, near Boring PE-17. Within the borings, the bedrock material consists of very poorly bedded to distinctly bedded siltstone and claystone with

cemented beds and concretions locally present, and possibly some minor sandstone with conglomerate.

Where observed in the borings, bedding planes have variable inclinations, ranging from less than 20 degrees (Boring PE-29) to near vertical (Boring PE-20). Existing geologic maps (Lamar, 1970; Dibblee, 1989) and other subsurface geologic data (LeRoy Crandall, 1979) indicate that near the alignment, bedding planes are inclined moderately to steeply in a southerly direction and are locally overturned. Numerous folds with axes that trend east-west to west-northwest are present in the Repetto Hills area.

Overall, the bedrock materials range from very soft to soft according to criteria provided by the Bureau of Reclamation in their "Engineering Geology Field Manual". A 4.5-foot thick zone consisting of hard, cemented, calcareous siltstone beds (each up to 1/2-inch thick) was encountered in Boring PE-28. Drilling through this interval resulted in continuous rig chatter and slow progress. Although Boring PE-28 was the only one in which cemented materials were encountered during this investigation, the available literature indicates that cemented beds, lenses and nodules, locally up to two feet thick, are present (Lamar, 1970; Converse, Davis and Associates, 1975; LeRoy Crandall and Associates, 1979; Converse and others, 1981 and 1984).

An interval of uncemented conglomeratic sandstone interbedded with siltstone was encountered below approximately nine feet of siltstone at a depth of 98 feet BGS in Boring PE-21. The sandstone is fine to coarse grained with gravel-sized clasts from approximately 1/4-inch to 1 1/4-inch in size and is iron-oxide stained.

The bedrock is typically dark olive gray when fresh and brownish when weathered. Within the Los Angeles River Narrows, the bedrock generally appears to be fresh or unweathered. Elsewhere, the weathering zone locally extends to a depth of 58 feet BGS, as observed in Boring PE-20.

4.4.4 Local Faulting

An east-west trending topographic escarpment that forms the southern margin of the City Terrace area in the Repetto Hills is as much as 80 feet high and can be traced as an intermittent feature from near the channel of the Los Angeles River in the west to the southern base of the Monterey Park Hills near Atlantic Boulevard in the east (Plate 1A). The escarpment is highest along the southern edge of the heights of City Terrace and diminishes to an indistinct feature that is less than 20 feet high near its intersection with the tunnel alignment. A second topographic escarpment occurs approximately one mile to the south. The southern escarpment has an east-northeast to northeast trend and its surface expression is relatively subdued compared to the City Terrace escarpment.

Geologic studies following the 1987 Whittier Narrows earthquake (M 5.9) attribute these and similar escarpments in the Elysian Park and Repetto Hills of central and eastern Los Angeles to ongoing folding and faulting. Seismologic, geodetic and geomorphic analyses indicate that the escarpments could result from either surface faulting or near-surface folding of weakly consolidated materials that overlie movements on deeply buried (or "blind") thrust faults (Davis and others, 1989). If continuous folds or faults extend northwestward from the Repetto Hills area, across the floodplain of the Los Angeles River, and into the Elysian Hills, as proposed by Dolan and Sieh (1992a, b, c), then the tunnel alignment will cross one of these features at as many as three locations along its length. (See location of escarpment on Plate 1A).

Previous Studies

Mapping compiled by the California Department of Water Resources (1961) first references the "Coyote Pass Fault" along the trend of the escarpment in the City Terrace area. In contrast, maps of the western Repetto Hills by Lamar (1970) and Dibblee (1989) show the escarpment as an undeformed erosional feature on dissected fans and terraces of Pleistocene alluvial deposits at the southern margin of the hills.

Recent mapping and topographic analysis of the Repetto Hills area demonstrate that the hills have been deforming during Quaternary time. Maximum rates of uplift are estimated to range from 0.1 mm/yr to 0.25 mm/yr (Bullard and Lettis, 1993). Bullard and Lettis (1993) believe that a front of active deformation has migrated south through the hills in response to thrust faulting at depth. They report that the escarpment in the City Terrace area is a "tectonic feature related to surface faulting", and that it results from right-oblique movement in the hanging wall of a deeply buried thrust fault, but they cite no exposures of offset or deformed materials in the escarpment to support this conclusion.

Studies of apparently offset stream courses along and near the escarpment in the City Terrace area also suggest that surface faulting with right-oblique fault displacements has occurred along the escarpment during Pleistocene time (Plate 1A, Dr. James Dolan, personal communication, 1993).

The log of a sewer tunnel excavation through City Terrace (LeRoy Crandall and Associates, 1979) supplies the only known information to confirm faulting in the escarpment materials. This log indicates that the escarpment is underlain by silty to cobbly alluvium which dips to the south between 25 and 46 degrees. The sewer tunnel passes through a fault zone of apparently minor displacement that consists of variably dipping clay shears in terrace alluvium. The fault zone parallels the Coyote Pass escarpment and coincides with one of the west-draining stream courses that are just to the north of the escarpment, and inferred by Dolan (personal communication, 1993) to be offset.

Current Investigations

If the City Terrace escarpment is the site of tectonic deformation due to faulting or periodic folding, it could cause deformation to tunnel structures intersecting it. Because it is topographically most prominent, the Coyote Pass escarpment in the vicinity of the proposed alignment was investigated on a preliminary basis for this report to evaluate its relationship to potential faulting. The investigation method consisted of continuous sampling and analysis of

six hollow-stem auger borings on and near the slope to search for evidence of faulting or near-surface deformation of sediments (Plate 3). The escarpment at Evergreen Avenue between Brooklyn Avenue and New Jersey Street was selected for drilling because of its ease of access and proximity to the proposed tunnel alignment, which passes beneath the escarpment about 1,200 feet to the west.

The six exploratory borings encountered folded alluvial deposits that are capped at the surface by a less folded and partially eroded residual soil (Appendix A and Plate 3). Deposits in each boring were found in sedimentary sequences which indicate that episodic pulses of floodplain deposition from about 5 feet to 30 feet thick occurred in the area prior to uplift or tilting. Each depositional sequence begins at depth with well-sorted sand to silty sand and gravel, and grades upward through variably sandy or gravelly silt. The tops of the sequences have dark-colored plastic clay horizons that developed by deposition and weathering at the ground surface during periods of sediment ponding and non-deposition. Renewed deposition of coarse-grained sediments buried each weathered clay horizon during a subsequent episode of flooding.

Consistent thicknesses of apparently laterally continuous sequences of beds indicate that the ground surface in the present escarpment area was an undeformed floodplain at the time of deposition. Subsequent to deposition, the sequences appear to have been folded into a shallow asymmetric syncline within and below the present escarpment. The deposits dip southward somewhat more steeply than the ground surface beneath most of the escarpment; they appear to be back tilted (i.e., inclined slightly northward) beneath the base of the escarpment and the planar terrace surface that extends south from the scarp.

As much as 7 feet of residual soil is preserved on the deposits on the lower slope of the escarpment and on the planar surface to the south. Remains of the soil become thinner on the higher escarpment and are locally absent where they have been stripped by erosion of the escarpment face. The cross-cutting relationship between the soil and the deposits it formed on indicates that the initially flat-lying deposits were folded and partially eroded prior to soil

formation. Both the deposits and the capping residual soil were additionally folded and further eroded, resulting in their present configuration in the escarpment.

The age of the alluvial deposits is unknown, but the nature of the residual soil allows a rough estimate of its age and approximate time of folding. Samples from the boreholes and exposures on the nearby escarpment face show the soil to be a well-developed weathering horizon. The reddish brown color of the soil, the included weathered and friable rock fragments, the intensity of soil ped development to produce a subangular blocky structure, and the presence of common clay skins on soil peds and around intact clasts indicate that the soil is characteristic of the Ramona soil series, as defined and mapped in central and southern California by the U.S. Department of Agriculture Soil Conservation Service (Woodruff and others, 1970; Knecht, 1971; Wachtell, 1978). The soil development occurs on both units Q_1 and Q_2 (map symbol Qoa_1, and Qoa_2, respectively on Plate 1) of Bullard and Lettis (1993) who show the escarpment materials as their Q_1 (map symbol Qoa_1) mapping unit. The least developed soil in the Ramona series, which appears comparable to the soil in the study area, is believed to have begun formation about 140,000 years ago (Ponti, 1985). Bullard and Lettis estimate the age of their Q_1 unit as less than 500,000 years. Partial folding and erosion of the deposits could thus have occurred between 500,000 and 140,000 years ago, whereas continued deformation resulting in the present topography would have occurred since 140,000 years ago. An estimate of the minimum age of deformation was not possible in this study, because deposits younger than 140,000 years were not found over the soil in any of the borings.

The investigations confirm tectonic deformation as the cause of the Coyote Pass escarpment but no evidence for major fault offset of the deformed deposits or soil was found in the borings. The relatively wide spacing between the borings allows the possibility that small vertical or larger lateral offsets could have been missed in exploration. A large vertical surface displacement such as suggested by Bullard and Lettis (1993) appears unlikely at the location investigated, given the distribution of seemingly continuous strata across the base of the escarpment slope. The lateral offsets of drainage channels proposed by Dolan (personal

communication, 1993), which total as much as 2,000 feet cannot be precluded with the data currently available.

Sieh (Dr. Kerry Sieh, personal communication, 1994) suggests the possibility of another fault which coincides with the southerly topographic escarpment. The escarpment crosses the alignment near Boring PE-5 (Plate 1A) and continues in a westerly direction directly south of Whittier Boulevard. West of Boring PE-5, the escarpment is obscure having little to no topographic expression. Northeast of the boring, the escarpment is the transition from mostly dissected older alluvial deposits to the northwest to comparatively undissected younger alluvial deposits to the southeast. No investigation of the southerly escarpment for this preliminary geotechnical investigation was performed.

Our subsurface exploration completed for feasibility studies indicates that the zone of deformation associated with the topographic escarpment at the southern edge of the City Terrace area is, in part, the result of folding of Pleistocene alluvial deposits. Bullard and Lettis (1993) and Dolan (1993, personal communication) propose that this folding accompanies right-oblique surface offsets on a north-dipping thrust fault.

Bullard and Lettis attribute the escarpment, which is locally as much as 80 feet high, to a vertical component of faulting. Dolan interprets west-trending drainage channels in the terrace surface above the escarpment to be offset as much as 2,000 feet by lateral displacement on multiple faults that parallel the escarpment face in a zone about 1,200 feet wide. Dolan and Sieh (1992a) propose that the escarpment is a portion of a fault zone that extends about 6 miles from the Elysian Hills in the west, to the eastern end of the Repetto Hills.

With little yet understood about the tectonic development of the escarpment, we can presently do little more than speculate about the potential effects of possible future tectonic activity associated with the escarpment on the proposed tunnel alignment. Deformed residual soil on alluvium in the escarpment face is perhaps as much as 140,000 years old. If we accept that both the Bullard and Lettis, and the Dolan hypotheses are correct, we can speculate about the possible

impact. For example, if the height of the escarpment and the apparent lateral offset of drainage channels have occurred entirely since the development of the residual soil, then there has been as much as about 80 feet and 2,000 feet (25 meters and 300 meters) of vertical and horizontal fault displacement, respectively, during the past 140,000 years. These figures yield average vertical and horizontal tectonic rates of slip of about 0.2 mm and 4 mm per year. If the escarpment in the City Terrace area is a portion of a larger, 6 mile-long fault, then future activity along this fault could generate earthquakes as large as magnitude 6.5 or 7. Earthquakes of magnitude 6.5 or 7 can result in as much as 6.5 feet of surface displacement per event (Bonilla and others, 1984).

The apparent correlation of sedimentary sequences found in the six boreholes within and below the escarpment face indicate that the apparent vertical separation of sedimentary horizons approximately equals the vertical relief of the ground surface across the present escarpment topography, confirming that sediments and the ground surface have been tectonically deformed as proposed by Bullard and Lettis (1993). Vertical displacements remain a distinct possibility in the future. The apparent lateral offsets of drainages suggested by Dolan cannot be disproved without additional subsurface investigations, but other geomorphic data tend to refute such a hypothesis. For example, two other drainage channels from the heights of City Terrace trend directly south across the proposed zone of faulting without apparent lateral offset. If faulting was responsible for Dolan's apparent offsets, all of the channels should be offset comparable distances, but they are not.

Explanations other than faulting are possible to develop the apparent lateral offsets of channels. The west-trending drainage channels could have developed from erosion along weak zones in the terrace alluvium that are controlled by minor faulting, such as that described in the sewer tunnel log or by the east-west strike of bedding within alluvial strata that are titled south as much as 46 degrees to the south within the terrace (LeRoy Crandall and Associates, 1979). Neither of these possibilities requires the large lateral offsets of the channels by faulting.

Given the existing data, the escarpment could be generated by either folding or faulting, although data mostly support folding as the mechanism. If the escarpment results from folding only of near-surface materials over an active buried thrust fault, then 6.5 feet of uplift by folding of the surface could result in about one or two degrees of tilting of the ground over the 200-foot width of the escarpment. If the relative offset of the terrace surface is expressed as surface faulting, then 6.5 feet of vertical offset could occur at one location or could be distributed over the 200-foot width at several locations.

The occurrence of tectonic events (fault slip or folding) appears to be relatively infrequent. For example, at Evergreen Avenue, where subsurface exploration was conducted, the relative difference in elevation across the escarpment is only about 30 feet. If events occurred in 6.5 foot increments, then only 4 such events may have occurred over the past 140,000 years, giving a recurrence of 35,000 years at that location. If the relative offset is less, then the recurrence could be assumed to be more frequent.

The most southerly escarpment identified by Dr. Sieh has not yet been investigated but may have similar geologic characteristics as the Coyote Pass escarpment.

4.5 LOCAL GROUNDWATER CONDITIONS

Groundwater levels along the alignment were monitored by 16 piezometers which were screened as indicated in Table 3-2. Piezometers installed in Borings PE-31, PE-30, PE-29, PE-18, PE-25 and PE-23 indicate that groundwater within the Los Angeles River area occurs at depths varying from approximately 35 feet below ground surface (BGS) in Boring PE-29 to 79 feet BGS in Boring PE-18. A possible barrier to groundwater flow may occur in the vicinity of Borings PE-27 and PE-28, where a shallow buried bedrock ridge was encountered in the Los Angeles River Narrows at depths ranging from 45 feet to 50 feet BGS (Plate 2A). The bedrock surface slopes down to depths ranging from 70 feet to 74 feet BGS to the north of the bedrock ridge and is 85 feet BGS and deeper to the south. The origin of the buried ridge is not known, but its location is approximately aligned with the projection of the escarpment of the "Coyote Pass fault" across

the Los Angeles River Narrows. Groundwater depth, historically have been much shallower in the narrows area. According to Converse and others (1984), groundwater was present in the alluvial deposits at depths ranging from 20 feet to 30 feet BGS between the Union Station area and Fourth Street.

Groundwater levels were monitored in ten piezometers to the east of the Los Angeles River Narrows (Table 3-2). Between Borings PE-21 and PE-16 groundwater appears to be perched in the older alluvial sediments that overlie relatively shallow bedrock. Groundwater depths in this section of the alignment range from approximately 52 feet to 79 feet BGS, and thus are at elevations slightly above, within and below the tunnel envelope (Plate 2A).

The piezometers located to the east of Boring PE-16 were constructed within predominantly coarse-grained older and younger alluvium, and have remained dry since their installation, with the exception of Boring PE-8, where groundwater was measured at a depth of 62.5 feet BGS in early December 1993. The piezometer was found to be dry in early January, 1994, suggesting a temporary local perched groundwater condition. Other data also indicate that perched groundwater conditions locally occur within these sediments. CTL Environmental Services (1992) encountered groundwater at a depth of approximately 45 feet BGS in the Calvary Cemetery area to the north of Borings PE-6 and PE-7. The presence of clayey intervals of low permeability that are found interbedded with coarser deposits in the borings suggest that perched groundwater should be anticipated to occur locally.

Regional groundwater data indicate that groundwater beneath the eastern part of the alignment occurs at depths on the order of 130 feet BGS (Los Angeles County Department of Public Works, various water level measurements for deep water wells).

Section 5.0

Section 5.0

5.0 SUBSURFACE CONDITIONS

5.1 SUBSURFACE STRATIGRAPHY AND GROUNDWATER CONDITIONS

5.1.1 General

The proposed alignment crosses mostly Quaternary-aged alluvium and local areas of bedrock. The thickness of alluvium ranges from less than 25 feet in the northwestern portion of the alignment to in excess of 500 feet at the southeastern end. Young (Holocene) Alluvium is present along the western portion of the alignment within the Los Angeles River floodplain and along several drainage paths traversing portions of the alignment. Within the Los Angeles River floodplain, Holocene alluvium is underlain by bedrock of the Fernando and Puente formations. Elsewhere the Holocene alluvium is underlain by Pleistocene-aged Old Alluvium and bedrock of the Fernando and Puente formations. The Fernando and Puente formations within the alignment area consist predominantly of siltstone and claystone interbedded with occasional layers of sandstone and local well-cemented hard calcareous interbeds.

Based on the stratigraphy interpreted from the results of subsurface investigations, the proposed alignment has been divided into two segments: the western segment covering the Los Angeles River floodplain and the eastern segment covering the remainder of the alignment.

Plate 2A presents a generalized cross sectional profile showing the subsurface stratigraphy for the entire alignment. It is based on the results of 30 borings, PE-1 through PE-30, and follows the CR Track. Plate 2B presents a cross sectional profile through the CL Track from Union Station to the proposed Little Tokyo Station, based on data from the current and previous investigations. The subsurface profiles are, in general, consistent with previous investigations in the area (Converse Consultants and others, 1981 and 1984; Caltrans, 1953, 1957a, 1957b, 1963, 1964, 1985a, 1985b; Earth Technology, 1986 and 1987a, b, c, d). Groundwater depths,

observed in the Little Tokyo Station area during the current investigations, however, were much lower than those recorded in the previous investigations (Converse Consultants, 1984).

5.1.2 Western Segment

The western segment extends approximately 1.5 miles from Union Station (approximate Station 13+20) in Los Angeles to the vicinity of the East Third Street/South Pecan Street intersection (Station 93+00), east of Boring PE-22. The western segment includes Little Tokyo Station near the South Santa Fe Avenue/Third Street intersection. Eleven borings (PE-22 through PE-31 and PE-18) were drilled within this segment during the current investigations.

Along this segment, the subsurface stratigraphy is generally represented by a shallow fill zone (to 9 feet thick) underlain by Young Alluvium and bedrock of the Fernando/Puente Formation. The bedrock was encountered in Borings PE-18, and PE-26 through PE-31 at depths ranging from about 45 feet to 85 feet BGS.

The alluvium is heterogeneous within this segment. Within the depths of exploration, the alluvium consists predominantly of loose to very dense granular soils occasionally interlayered with fine-grained soils consisting of sandy clays and clayey silts of low to medium plasticity. The granular alluvium mainly consists of silty sands (with and without gravel), poorly to well graded sand (with and without silt and/or gravel), and poorly to well graded gravel (with and without silt and/or sand) with some cobbles and boulders. The relatively high blowcounts (in excess of 100) measured within some of the sand and silt layers are a result of the presence of gravels and cobbles and are not considered representative of the density/consistency of the sand/silt matrix surrounding the gravels or cobbles. A significant portion of the granular layers within the proposed tunnel zone consists of poorly to well graded sands and may be susceptible to raveling and running/flowing conditions during tunneling. Also, cobbles (3 to 12 inches in size) and boulders up to 36 inches in size were encountered during the current investigation, as evidenced by a combination of factors including rock fragments in the cuttings, zero or low sample recovery, blowcounts greater than 100 and drill rig behavior. Cobbles were encountered

in Borings PE-18, PE-23 through PE-26, and PE-28 through PE-31. The presence of boulders was observed in Borings PE-23, PE-25, and PE-28 through PE-30. Boulders up to 48 inches in size were reported in a previous investigation (Converse Consultants, 1981). The cobbles and boulders are primarily composed of very hard to extremely hard granitic and metamorphic rock types that are unweathered and durable.

The bedrock encountered within this segment consists primarily of weak, slightly weathered to fresh, thinly laminated to massive siltstone and claystone interbedded with occasional hard well-cemented calcareous beds and conglomeratic sandstone layers.

The groundwater levels in this segment were monitored in five monitoring wells (PE-23, PE-18, and PE-29 through PE-31) and one piezometer (PE-25). Each of the piezometers/monitoring wells was screened within a zone that generally encompassed the tunnel envelope as indicated in Table 3-2 and Plates 2A and 2B. The observed groundwater levels are also shown in the same plates and table. In general, the groundwater level is approximately 30 to 40 feet BGS near the Union Station area and south of U.S. 101 Freeway up to the vicinity of First Street. South of First Street and in the vicinity of the Los Angeles River, the groundwater level was observed to be approximately 70 to 80 feet BGS. East of the Los Angeles River groundwater was observed at about 50 to 60 feet BGS. Groundwater levels reported in previous investigations (Converse Consultants and others, 1981 and 1984; Earth Technology, 1987; Caltrans, 1985) are also shown on Plates 2A and 2B. Significant differences exist in groundwater elevations recorded during the current investigation and those recorded in 1983 by Converse Consultants particularly in the vicinity of Boring PE-18 (Little Tokyo Station). The groundwater level recorded at this location in 1983 was about 55 feet higher than measured during the current investigation. This discrepancy cannot be easily explained by any known geologic condition and would require further investigation. An accurate definition of the groundwater conditions in this area is critical since groundwater will affect the need for dewatering over a significant portion of the tunnel and at the Little Tokyo Station.

5.1.3 Eastern Segment

The eastern segment extends approximately 5.1 miles from the East Third Street/South Pecan Street intersection (Station 93+00) to the Whittier Boulevard/South Oakford Drive intersection (Station 363+92.39) in East Los Angeles. The eastern segment includes six stations: First/Boyle, Brooklyn/Soto, First/Lorena, Whittier/Rowan, Whittier/Arizona, and Whittier/Atlantic.

The generalized cross sectional profile shown in Plate 2A also includes the eastern segment. The eastern segment profile is based on the results of 20 borings (PE-1 through PE-17, and PE-19 through PE-21), and seven CPT soundings (CPT-1 through CPT-6 and CPT-6A).

Along the eastern segment, the subsurface stratigraphy is generally represented by a shallow fill zone (to 4 feet thick) underlain by Young and Old Alluvium. The Young Alluvium which exists within several drainage courses that cross the alignment and at the eastern end of the segment, is underlain by the Old Alluvium and bedrock. Bedrock of the Fernando/Puente Formation consisting of interlayered claystone, siltstone and sandstone was encountered below the alluvial deposits along the northwestern portion of this segment at Borings PE-14, PE-15, PE-17, and PE-19 through PE-21. At the eastern end, the bedrock is estimated to be relatively deep (in excess of 500 feet BGS). The subsurface stratigraphy is, in general, consistent with that observed in previous Caltrans borings in the vicinity of this segment.

The alluvium is extremely heterogeneous in the eastern segment with a high percentage of fine-grained materials in comparison to the western segment. Within the depths of exploration, the alluvium consists predominantly of loose to very dense granular soils consisting of clayey sands, silty sands, poorly to well graded sands (with and without gravel), gravels (with and without sand/silt), cobbles and boulders; and medium stiff to hard cohesive soils consisting of clays, sandy clays, sandy silts and silts of low to medium plasticity. Although the coarse and fine-grained alluvium is interlayered, the proposed tunnel profile traverses some zones that are predominantly coarse grained, some that are predominantly fine grained and some that are

closely interlayered. Significant zones of poorly to well graded sands and gravels that could potentially ravel or run within the tunnel excavation were encountered in Borings PE-1, PE-2, PE-5 through PE-7, PE-9 through PE-19, and PE-21. The high blowcounts measured within sand layers are caused by the presence of gravels and should not be considered representative of the density/consistency of the matrix material in the alluvium. Cobbles (3 to 12 inches in size) and occasional boulders (greater than 18 inches) were also observed in Borings PE-6, PE-7, PE-10, PE-11 and PE-21, as evidenced by rock fragments in the cuttings, low sample recovery, high blowcounts and rig behavior observed during field exploration. As in the western segment, the cobbles and boulders are composed of very hard to extremely hard, unweathered granitic and metamorphic rock types. However, the occurrence of cobbles and boulders is less frequent in the eastern segment and it will not be as severe a constraint for tunneling conditions as in the western segment.

The bedrock encountered within this segment consisted predominantly of weak, intensely weathered to fresh, thinly laminated to massive siltstone and claystone with occasional sand layers and hard, well-cemented calcareous layers. Sandstone and conglomeratic sandstone layers were encountered in Boring PE-21.

The groundwater levels in this segment were monitored in eight piezometers (Borings PE-1, PE-3, PE-7, PE-11, PE-13, PE-17, PE-19 and PE-21) and two monitoring wells (Borings PE-8 and PE-16). The screened zones within the borings are shown in Table 3-2 and Plate 2A. The screened intervals did not cover the entire tunnel envelope in Borings PE-1, PE-3, PE-7, PE-11, PE-13 and PE-16 due to the revision in the proposed tunnel profile after completion of the field exploration program.

Groundwater was observed in Piezometers PE-17, PE-19 and PE-21, Monitoring Well PE-16 and temporarily in Monitoring Well PE-8. The observed groundwater levels are shown in Plate 2A and Table 3-2. Groundwater was initially observed in Monitoring Well PE-8 at 55 feet BGS on December 2, 1993. However, following well development and water sampling the water level did not rise in the well and the well was found to be dry. Previous Caltrans exploration

near the topographic high at Boring PE-20 indicated groundwater at an elevation of 337 feet or 22 feet BGS. Results of our monitoring and review of available data indicate that the groundwater is approximately 20 to 60 feet BGS, and, in general, about 10 to 15 feet above the bedrock for the western portion of the eastern segment between Borings PE-21 and PE-14. In the remaining portion of the eastern segment (east of Boring PE-14), groundwater was not encountered in any of the monitoring wells, with the exception of the temporary presence of perched water in PE-8. Available regional data suggests that the groundwater level is deeper than about 150 feet BGS in this area.

Local perched groundwater zones are likely along the entire eastern segment, as suggested by the temporary presence of water in Monitoring Well PE-8, previous studies (CTL Environmental Services, 1992) in the Calvary Cemetery area (approximate Station 293+00) where groundwater was encountered at 45 feet BGS, and the presence of "wet" coarse grained soil samples observed in several borings during drilling (Plate 2A).

5.2 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

The engineering properties of subsurface materials, as obtained from results of laboratory tests, are summarized in Table 3-4. Blowcount data (equivalent SPT N-values) from standard penetration tests and drive sampling are shown in the borehole logs and presented in Table 3-4. Interpretations drawn from the CPT soundings are presented with the CPT logs in Appendix A.

Table 5-1 presents a summary of the measured/interpreted ranges of relevant geotechnical parameters for the various material types encountered within the two segments. For purposes of presentation, the alluvium has been broadly categorized into fine-grained and coarse-grained alluvium. The fine- and coarse-grained alluvium are interlayered within both segments. However, the alluvium within the western segment is predominantly coarse grained.

TABLE 5-1. SUMMARY OF ESTIMATED ENGINEERING PROPERTIES

ENGINEERING CHARACTERISTICS	WESTERN TUNNEL SEGMENT			EASTERN TUNNEL SEGMENT		
	FINE-GRAINED ALLUVIUM	COARSE-GRAINED ALLUVIUM	BEDROCK (Tf/Tp?)	FINE-GRAINED ALLUVIUM	COARSE-GRAINED ALLUVIUM	BEDROCK (Tf/Tp?)
USCS Classification	CL,CH,ML CL-ML	SP,SW,GP,GW,SM,SC GP-GM,SW-GP SP-SM,SW-SM	ML,MH,CL	CL,CH,ML CL-ML	SP,SW,GP,GW,SM,SC GW-GM,GP-GM SP-SM,SW-SM	ML,MH,CL SW,SP
Equivalent SPT Blow Counts	2 - >100	2 - 100	29 - >100	4 - >100	5 - >100	20 - >100
Insitu Moisture Content (perent)	9 - 27	2 - 25	22 - 32	6 - 39	5 - 26	17 - 34
Insitu Dry Density (pcf)	100 - 124	90 - 131	92 - 106	80 - 126	86 - 136	96 - 105
Fines Content (% passing #200 Sieve) (percent)	50 - 76	1 - 49	84 - 98	51 - 99	3 - 50	89 - 99
Specific Gravity	2.72	--	--	2.71 - 2.73	2.79	2.72 - 2.74
Liquid Limit (percent)	26 - 57	--	29 - 69	18 - 55	--	37 -61
Plasticity Index (percent)	3 - 37	--	1 - 36	4 - 33	--	9 - 37
Peak Shear Strength Cohesion, (psf)	900 - 1600	--	700 - 1250	150 - 1700	600 - 700	600 - 2750
from laboratory tests Friction Angle, (degrees)	27 - 33	--	29 - 31	24 - 38	29 - 34	23 - 32
Undrained Shear Strength (interpreted from CPT soundings) (psf)	--	--	--	1240 - 20800	--	--
Friction Angle (interpreted from CPT & SPT) (degrees)	--	27 - 48	--	--	27 - 48	--
Unconfined Compressive Strength (psi)	--	--	34 - 120	--	--	64 - 99
pH	7.89 - 8.59	2.38 - 9.09	7.52 - 7.71	6.52 - 8.80	5.50 - 8.88	5.96 - 7.29
Chloride Content (ppm)	112 - 172	97 - 674	119 - 777	99 - 256	33 - 230	113 - 120
Sulphate Content (ppm)	87 - 289	61 - 1645	183 - 701	11 - 2071	24 - 136	29 - 99
Electrical Resistivity (ohms-cm)	1887 - 3333	286 - 4000	1333 - 7692	958 - 12987	1395 - 9937	638 - 1081
Compression Index - Cc	0.07	--	--	0.09 - 0.13	--	0.11 - 0.16
Swelling Index - Cs	0.01	--	--	0.01 - 0.02	--	0.02 - 0.04
Rate of Secondary Compression - Cx	0.0014	--	--	0.0023 - 0.0028	--	0.0017 - 0.0021
Swelling (-)/Collapse (+)	-0.01	--	--	+0.02 - +0.21	--	-0.53 - -0.014

NOTES:

1. Cc, Cs, and Cx are Based on vertical strain - log stress plots
2. Western Tunnel Segment corresponds to Borings PE-18 and Borings PE-22 through PE-31
3. Eastern Tunnel Segment corresponds to Borings PE-1 through PE-17, Borings PE-19 through PE-21 and CPT Soundings CPT-1 through CPT-6A
4. Only one laboratory test result is available wherever range of properties is not shown
5. Equivalent SPT Blow Counts in alluvium may not be representative of material density/consistency due to the presence of gravels, cobbles and boulders
6. Results of gradation, insitu moisture content and insitu dry density tests on granular alluvium may not be representative due to the presence of gravels, cobbles and boulders

Where the recorded SPT blowcounts and interpreted equivalent SPT blow counts from drive samples are relatively high in the alluvium, they should not be considered representative of the material density/consistency. The high blowcounts recorded are due typically to the presence of gravels and cobbles within the alluvium and do not reflect the relative denseness of the alluvial matrix. Also, due to the presence of gravels and cobbles larger than the sampler size, the results of gradation, in situ moisture content and in situ dry density tests on the granular alluvium may not be truly representative of the total deposit.

The key laboratory soil engineering properties relevant to the design and construction of the tunnel and stations, based on this investigation, include the following:

- Gradation (particle size distribution and fines content), index tests and classifications of materials to be encountered within the tunnel envelope and the station excavations.

- Shear strength characteristics of materials to be excavated within tunnels and cut and cover stations, and materials supporting station foundations.

- Corrosivity of soils within the tunnel and station zones.

- Compressibility of soils below station foundations

5.2.1 Grain Size Distribution

Results of grain size distribution and fines content (percentage passing #200 sieve) tests are summarized in Table 3-4, Table 5-1 and in Table B-1 of Appendix B. The measured fines content of the alluvial layers is also indicated on Plate 2A. The bulk of the gradation and fines content tests were performed on selected granular samples in the vicinity of the tunnel and station excavation. This was done primarily to evaluate areas of cohesionless sands and gravels which may be susceptible to ravelling/running/flowing conditions. Adequate grain size analyses could not be performed on samples from the gravelly and cobbly layers since representative

samples of such materials cannot be obtained from small diameter boreholes/samplers. In layers classified as gravel, clayey gravel, silty gravel, gravel with sand and sand with gravel, gradation curves presented may not be truly representative of the entire deposit and may only reflect gradations of finer matrix materials in the coarse alluvium.

Results of gradation tests show the presence of significant zones of granular alluvium with low fines content (poorly to well graded sands and gravels) that would be potentially susceptible to raveling, running or flowing conditions within tunnel and station excavations. Such zones exist within the entire western segment and within significant portions of the eastern segment, particularly in Borings PE-1, PE-2, PE-5 through PE-7, PE-9 through PE-19, and PE-21).

5.2.2 Classification of Fine-Grained Soils

Sample classifications per USCS and ASTM guidelines accompany the borehole logs and laboratory test summary tables. Results of Atterberg Limit tests are presented in Table 3-4, Table 5-1 and Table B-2 of Appendix B. Results indicate that the bulk of the fine-grained alluvium consist of clays, sandy clays and silts of low to medium plasticity with liquid limits typically ranging from 18 to 50, and plasticity indices ranging from 3 to 30. The bedrock materials are predominantly fine grained and consist of siltstone and claystone of low to high plasticity, with liquid limits ranging from 29 to 69 and plasticity ranging from non-plastic to a plasticity index of 37.

5.2.3 Shear Strength

Laboratory direct shear tests (Table 3-4, Table 5-1 and Table B-3 of Appendix B) performed on selected representative samples of fine grained alluvium showed peak cohesion values ranging from 150 to 1,700 psf, and peak friction angles ranging from 24 to 38 degrees. The tests were performed on relatively undisturbed medium stiff to very stiff samples from depths ranging from 25 to 75 feet BGS. Undrained shear strength of the fine-grained alluvium as interpreted from the CPT soundings typically ranges from 1,200 psf to greater than 20,000 psf.

Shear strength of granular alluvium may be estimated based on equivalent SPT blowcounts or interpreted from CPT data. Based on the CPT and SPT data, friction angles range from about 27 degrees for the loose silty sands to 48 degrees for the very dense sands and gravels (Table 3-4). Laboratory direct shear tests on medium dense to dense silty sands resulted in peak cohesion values ranging from 600 to 720 psf, and friction angles ranging from 29 to 34 degrees (Tables 3-4, 5-1 and B-3).

Direct shear tests on siltstone and claystone bedrock samples (Tables 3-4, 5-1 and B-3) resulted in peak cohesion values ranging from 600 to 2,700 psf, and friction angles ranging from 23 to 32 degrees. Unconfined compressive strengths of selected claystone and siltstone samples ranged from 34 to 120 psi (Tables 3-4, 5-1 and B-4).

5.2.4 Corrosivity

Results of corrosivity tests are summarized in Table 3-4 and Table B-5 of Appendix B. Results of soluble sulfate content tests in soil samples and sulfate content tests in groundwater samples indicate that, with the exception of one sample from Boring PE-2, the subsurface materials are predominantly mildly to moderately corrosive to concrete. The sample tested from Boring PE-2 appears to be severely corrosive (soluble sulfate content greater than 2,000 ppm).

Results of electrical resistivity tests (286 to 12,987 ohm-cm) indicate that most of the subsurface materials are moderately corrosive to extremely corrosive to metals. Very corrosive materials (electrical resistivity less than 2,000 ohm cm) were encountered in Borings PE-1 through PE-3, PE-6, PE-10, PE-11, PE-13 through PE-16, PE-18, PE-21 through PE-23, PE-26, PE-29 and PE-30.

5.2.5 Compressibility

Consolidation tests were performed on four samples of very stiff to hard, fine grained alluvium from the 35-foot to 60-foot depth range, and two samples of claystone from the Fernando/Puente Formation. The test results summarized in Table 3-4 and Table B-6 of Appendix B indicate that the compression index (ratio of vertical strain to log stress) of the fine grained alluvium ranges from 0.07 to 0.13, while that of the siltstone/claystone ranges from 0.11 to 0.16. The results indicate that the measured compressibility is consistent with the types of materials (very stiff) tested.

5.3 LIQUEFACTION POTENTIAL

The proposed alignment is located in an area having a high seismic potential. It is also located within five miles or less of the east-west-trending Hollywood and Raymond faults and a postulated extension of the Whittier fault, each one of which has the potential for a Maximum Credible Earthquake (MCE) of Magnitude 7.5. Based on the attenuation relationship of Joyner and Boore (1982), the peak ground surface acceleration (PGA) associated with the MCE on any one of these faults is estimated to range from 0.6 to 0.75g along the alignment. The CDMG Open-File Report 92-1 (California Department of Conservation, 1992) which provides contours of estimated PGA values from MCEs in California, indicates peak accelerations ranging from 0.5g to 0.6g for the alignment.

A significant potential effect of seismic shaking is soil liquefaction. Liquefaction is a phenomenon in which saturated soils (typically silts or sands) undergo a temporary loss of strength during vibrations caused by earthquakes. In extreme cases, the soil particles can become suspended in groundwater and the soil deposits become mobile with fluid-like behavior. The factors known to influence liquefaction potential include: grain size, relative density of soil, groundwater level, degree of saturation, confining pressures, and the intensity and duration of ground shaking.

Within the project limits, several areas with relatively shallow groundwater, have been identified by various agencies as being potentially liquefiable. The CDMG Special Publication 99 (California Department of Conservation, 1988), which provided earthquake planning scenarios for a major earthquake on the Newport-Inglewood fault zone, has identified some areas in the vicinity of the Los Angeles River, north of the San Bernardino Freeway (10), with medium liquefaction susceptibility. The U.S. Geologic Survey Professional Paper 1360 (U.S. Geological Survey, 1985), which presents articles on the earthquake hazards in the Los Angeles region, indicates that some areas, particularly west of the Los Angeles River near Union Station, have a moderate to high liquefaction potential. The alignment also crosses some areas east of the Los Angeles River that have been identified as potentially liquefiable in the Los Angeles County Seismic-Safety Element Map (County of Los Angeles Department of Regional Planning, 1990).

A site-specific liquefaction potential evaluation based on the available borehole, CPT and groundwater information was performed for an anticipated PGA of 0.7g (from a Magnitude 7.5 earthquake). Groundwater was conservatively assumed to be at a depth of 30 feet BGS within the western segment, at the proposed tunnel crown elevation within the western portion of the eastern segment (west of Boring PE-14), and very deep (greater than 150 feet) over the eastern portion of the alignment (Borehole locations PE-1 through PE-13). The evaluation was carried out using procedures outlined by Seed et al (1983) and Seed (1987) for liquefaction under level ground.

A significant portion of the alluvium along the alignment is granular and contains variable amounts of gravels and cobbles. Typically, high blowcounts were observed within the granular alluvium layers. However, the SPT blowcounts are not a good indicator of liquefaction potential in coarse alluvium with gravels and cobbles. The liquefaction potential analyses presented herein is therefore only applicable to those layers of granular alluvium free of significant amounts of gravels, cobbles and boulders. Additional field investigations to evaluate the relative density of these coarse materials by using equipment such as the Becker hammer, are recommended for evaluating the liquefaction potential within gravelly/cobbly layers.

TABLE 5-2. ALLUVIAL LAYERS WITH A POTENTIAL FOR LIQUEFACTION OR STRENGTH LOSS

Alignment Segment	Borehole No./ CPT No.	Assumed Groundwater Depth (feet)	Depth Range (feet, BGS)	Material Type
Western Tunnel Segment	PE-22	30	63-68	Sand with silt (SP-SM) to well graded sand with silt and gravel (SW-SM)
	PE-25	30	65-68	Silty sand (SM) to sandy silt (ML)
Eastern Tunnel Segment	PE-15	36	50-53	Silty sand (SM)
			58-63	Sand with silt (SP-SM)
	PE-16	32	54-65	Sand with silt (SP-SM) to silty sand (SM)
	CPT-5	32	35-37	Silty sand (SM) to sandy silt (ML)
	PE-19	30	30-33	Silty sand (SM)

Note: 1. Peak ground acceleration of 0.7g was used for liquefaction analysis for the eastern and western tunnel segments.

Results of the liquefaction evaluation for granular layers free of gravels/cobbles are presented in Table 5-2. The results indicate that within the western tunnel segment, 3-foot to 5-foot thick potentially liquefiable sand layers occur in Boreholes PE-22 and PE-25 in the 63-foot to 68-foot depth range. Within the western portion of the eastern segment (west of Boring PE-14), pockets of potentially liquefiable sand layers, 2 feet to 11 feet thick, are evident at Borehole/CPT locations PE-15, PE-16, CPT-5 and PE-19. Liquefaction is not considered likely for the section of the alignment east of PE-13, provided the groundwater levels remain relatively deep as currently observed.

The liquefiable layers identified above appear to be localized and occur within or below the tunnel zone. Potential impacts of liquefaction may not be significant and may only include localized loss of support around the tunnel, and settlements on the order of a few inches. Additional zones of potentially liquefiable zones are likely to exist within the gravelly layers. Additional studies to obtain representative blow count data, as discussed above, are needed to properly evaluate the liquefaction susceptibility of these layers.

5.4 SOIL AND GROUNDWATER CONTAMINATION

This section presents a preliminary assessment of soil and groundwater contamination along the alignment based on the data obtained from the current investigation and available data from previous investigations in the vicinity of the alignment.

5.4.1 Data from the Current Investigation

The scope of environmental monitoring and testing performed in this investigation included a limited chemical testing program on groundwater samples, screening soil samples with an OVA for the potential presence of volatile organic compounds (VOCs), and monitoring selected groundwater samples for hydrogen sulfide (H_2S) using a multiple gas indicator. The results of chemical testing and H_2S monitoring are presented in Sections 3.2.2 and 3.3, respectively. Headspace OVA readings and field observations of hydrocarbon and sulfur odors are presented

in the boring logs (Appendix A). Significant OVA readings (exceeding 10 ppm above background levels) as well as locations where odors were noticed are summarized in Table 5-3.

Based on the results from this investigation the following preliminary conclusions were developed:

1. Groundwater from monitoring wells PE-29 to PE-31 is likely contaminated with hydrocarbons above California Department of Health Services (CDHS) Maximum Contaminant Levels (MCLs) for drinking water. The contaminants that require treatment prior to disposal include, but may not be limited to total petroleum hydrocarbons (TPH), volatile organic compounds (VOCs including benzene, toluene, ethylbenzene, xylene, vinyl chloride, etc.), and a number of semi-volatile organic compounds (SVOCs).

2. Groundwater from Monitoring Wells PE-29 to PE-31 is also contaminated with H_2S as evidenced by the release of H_2S from groundwater samples.

3. Similarly, zones of subsurface soils in the vicinity of Borings PE-28 and PE-29 are likely to be contaminated with hydrocarbons and H_2S as evidenced by the groundwater contamination and high OVA readings and observed hydrocarbons and sulfurous odors in the area.

4. No evidence of hydrocarbons or H_2S contamination was found in soil and groundwater samples from Boring/Monitoring Well PE-18. This may be due to the presence of a "geologic barrier" somewhere between Borings PE-29 and PE-18 as discussed in Section 4.5.

5. High OVA readings in Boring PE-15 located within the Boyle Heights Oil Field may be indicative of potential hydrocarbon contamination in the subsurface soils

TABLE 5-3. SUMMARY OF SIGNIFICANT OVA READINGS AND FIELD OBSERVATIONS OF ODORS

Boring No.	Groundwater Depth (feet)	Depth Range (feet)		Depth Range (feet) with OVA Reading		Maximum OVA Reading Above Background	
		Hydrocarbon Odor Observed	Sulfur Odor Observed	>10 ppm Above Background	>100 ppm Above Background	Quantity (ppm)	Depth (feet)
PE-31	38	35 to 50	55 to 80	35 to 60	65 to 80	670	80
PE-30	37	-	30 to 80	50 to 74	75 to 80	>1,000	80
PE-29	35	-	32 to 75	35 to 65	70 to 80	940	80
PE-28	No data	-	45 to 50	45 to 70	75 to 80	100	75 and 80
PE-15	No data	-	-	-	65 to 90	770	80 and 85
PE-8	55 (perched groundwater)	-	-	70	-	15	70

and the potential existence of hydrocarbon gases (e.g. methane) in the general area.

6. High concentrations of petroleum hydrocarbons detected in the perched groundwater sample from Monitoring Well PE-8 appear to be related to a nearby active LUST (Leaking Underground Storage Tank List - Regional Water Quality Control Board) site at the Thrifty Gas Station located at 3981 Whittier Boulevard. Thus, other nearby active LUST sites or sites with past and ongoing activities that may impact the soil and groundwater along the alignment have been investigated as part of environmental assessments and are discussed in a separate report.

5.4.2 Other Available Data

Several previous environmental investigations have been conducted to assess soil and groundwater near the Union Station area and the MTA Railroad Maintenance Yard located along South Santa Fe Avenue approximately between Fourth Street and Commercial Street. Available reports from those investigations were reviewed. Table 5-4 presents a summary of these data and their primary findings.

The data generally confirm the presence of contamination discussed above and represent a valuable supplementary database for assessing soil and groundwater contamination between Union Station and the proposed Little Tokyo Station.

TABLE 5-4. SUMMARY OF AVAILABLE SOIL, GROUNDWATER AND GAS CONTAMINATION DATA FROM OTHER INVESTIGATIONS NEAR UNION STATION AREA

Sources	Location/Area of Investigation	Primary Findings
Converse Consultants (1984)	MOS-1 Contract A-100 area including portion between Union Station and the vicinity of the proposed Little Tokyo Station	■ Mixture of H_2S and hydrocarbon gases (including methane) released from groundwater and exploded in a monitoring well during a pump test near west end of Union Station. ■ Boring CEG-2 (about 2,000 feet east of Union Station) encountered oil stain in soil samples and sulfur odor. A gas sample from this boring contained 100 ppm methane and 500 ppm ethane. ■ Oil stains and sulfur odor were encountered in soil samples from borings near Union Station
Woodward-Clyde Consultants (1986)	El Monte busway	■ Soil contamination with volatile and semi-volatile organic compounds to a depth of 30 feet.
Earth Technology (1986; 1987a, b, c, d)	A-130 corridor east of Union Station including Denny's Restaurant (Vignes Street off-ramp from U.S. 101 Freeway and Ramirez Street)	■ Sulfur and hydrocarbon odors in borings and oil stains in soil samples. ■ Soil and groundwater samples from the vicinity of Denny's Restaurant were contaminated with petroleum hydrocarbons. ■ High OVA readings (>1,000 ppm above background level) were observed at Boring B-204, B-302A and B-303A).
Levin-Fricke (1993) RWQCB (1993)	Gateway Center at southwest corner of Macy Street and Vignes Street near Union Station	■ Groundwater contamination with H_2S and volatile organic compounds (VOCs). ■ Ongoing treatment system for groundwater from dewatering (450,000 gpd) using hydrogen peroxide to oxidize H_2S, filtration of sulfur and/or suspended solids and active carbon to remove VOC. ■ Capacity of the treatment plant is 1.2 million gallons per day.
Law/Crandall (1993)	Metro Pasadena Line	■ Two borings adjacent to Union Station recorded OVA readings >50 ppm in soil samples.

Section 6.0

6.0 DESIGN AND CONSTRUCTION

6.1 GENERAL

This section discusses some of the key geotechnical issues and constraints identified during this preliminary investigation that should be considered in the design and construction of the tunnels and stations along the Eastside Extension alignment. The scope of this preliminary investigation was limited to characterizing subsurface and groundwater conditions by widely-spaced borings, and a few monitoring wells at selected locations. Therefore, the findings and discussions presented in this section are preliminary and will require further evaluation when additional information becomes available.

6.2 SUMMARY OF RELEVANT SUBSURFACE STRATIGRAPHY

Based on current plans and profiles, the proposed tunnels and stations will be within alluvium and the Fernando/Puente Formation bedrock. The alluvium is heterogenous and non-uniform. In the western segment (from Union Station to the vicinity of PE-22 at approximate Station 93 + 00), alluvium consists predominantly of gravel, gravelly sand, sand and silt with local cobbles and boulders (up to 4 feet in size) and occasional and localized layers of sandy clay and clayey silt. Alluvium in the eastern segment (remaining portion of Eastside Extension) is older and consists of fine-grained alluvium consisting of clay, sandy clay and clayey silt and granular alluvium similar to that found in the western segment but with fewer cobbles and boulders, particularly boulders larger than 18 inches in size. The granular alluvium over a large portion of the entire alignment consists of sands and gravels with low fines content and would be susceptible to raveling, running/flowing conditions within tunnel/station excavations. The boulders and cobbles encountered are typically very hard to extremely hard unweathered granitic and metamorphic rock types. Within the tunnel envelope and station excavation depths, the bedrock materials, when encountered, are expected to consist predominantly of very low-strength (defined as having uniaxial compressive strength less than 4,000 psi) siltstone, claystone and

sandstone, except for local zones of hard, well-cemented calcareous interbeds with a maximum thickness of about 5 feet. Except for such local hard and well-cemented interbeds, the Fernando/Puente Formation, for tunneling purposes, is expected to behave similar to hard and dense soils.

6.3 TUNNEL

6.3.1 Excavation and Support

Tunnels along Eastside Extension will be in alluvium except the following approximate sections where tunnels will be partly (mixed face conditions) or entirely within Fernando/Puente Formation:

- From the vicinity of Boring PE-29 to the vicinity of Boring PE-18 (i.e. north of Little Tokyo Station)

- From the vicinity of Boring PE-21 to the vicinity of Boring PE-17.

Further delineation of alluvium/bedrock contact would be required to refine the limits of the above sections.

It is anticipated that soft ground/soft rock tunneling methods will be generally applicable except where boulders or local well-cemented interbeds in the bedrock are encountered. Boulders will likely require special handling and use of suitable tunneling machines to achieve an efficient rate of advance, and to provide face stability and reduce potential ground settlement. Boulders should be anticipated within the entire western segment and portions of the eastern segment (particularly in the vicinity of Borings PE-5, PE-6, PE-7, PE-10 and PE-11). Large boulders to 4 feet in size and hard interbeds in the bedrock will likely reduce advance rates and may require splitting in the face or on the mucking conveyor. Previous tunneling experience in similar subsurface conditions indicates that tunnels along the Eastside Extension can be advanced

using mechanical excavation method within a shield and with initial support consisting of precast concrete liners. The tunnels will be finished with a final lining of cast-in-place concrete.

In addition to large boulders, tunneling partly or fully in alluvium along the alignment will encounter raveling and running conditions because of the predominantly granular nature of the alluvium. Slow raveling conditions (dewatered or above groundwater silty sand and clayey sand) should not be a major concern in properly conducted shielded mechanical excavations, provided the initial lining support and backfilling of the tail voids are applied in a timely fashion. Fast raveling conditions and running/flowing conditions can be anticipated in cobbles, gravels, gravelly sand, and poorly graded sands above or below groundwater, or well-graded sand below groundwater. Fast ravelling and running/flowing conditions are anticipated over a major portion of the tunnel within alluvium. Subsurface materials encountered in the tunnel zone in Borings PE-1, PE-2, PE-5 through PE-7, PE-9 through PE-17, and PE-21 through PE-31 may be subject to such conditions. These conditions will require use of one or a combination of the following provisions to enhance face stability and to reduce potential settlement:

- Dewatering from the surface or ahead of the excavation. The feasibility, design and cost of the dewatering system will depend upon the hydraulic head and level of groundwater contamination, if any. Potential for groundwater contamination is discussed in Section 6.5.
- With dewatering, use of an open shield fitted with breasting doors and poling plates (or movable hood and jack systems) for excavation face control and to help mitigate the potential for, and effects of cave-ins.
- Use of a shield with a pressure regulated trap door
- Use of a suitable earth pressure balance (EPB) machine
- Stabilization of the granular soil zones near and around the tunnel crown by chemical grouting from the tunnel face or compaction grouting (cost effective if the granular zones are localized)

Even in areas where the groundwater table is below the tunnel zone, local flowing ground conditions will occur where perched groundwater is encountered. Local perched groundwater is possible since fine-grained soils are locally present in the alluvium throughout the alignment.

6.3.2 Groundwater Control

Available groundwater level data from this investigation indicate that possibly the entire, or at least a significant portion of the tunnel envelope within alluvium will likely be below groundwater. The following sections of the alignment are apparently the most likely to be affected:

- from Union Station to somewhere between Borings PE-28 and PE-18.
- from the vicinity of Boring PE-25 to the eastern boundary of the First/Boyle Station (between Borings PE-20 and PE-21)
- from the vicinity of Boring PE-19 to the vicinity of Boring PE-14.

As discussed in Section 5.1.2, there is a significant discrepancy with respect to groundwater level data between the current investigation and the 1983 measurements by Converse Consultants (1984), particularly in the area of Borings PE-18 and PE-26 where the 1983 levels are approximately 60 feet higher than current levels. The 1983 data suggest that the entire western segment of the tunnel could be below groundwater level. This discrepancy will have significant impact on the extent of dewatering needs and liquefaction potential evaluations, and will require resolution prior to final design. Thus, until further refinement, the conclusions and recommendations in this section should be considered preliminary.

Dewatering of the portions identified above will be necessary to enhance stability, and mitigate the potential for ground settlement and for inflows of water during tunnel excavation.

Available groundwater information also appears to indicate that groundwater levels approximately between Boring PE-14 and the eastern terminus (Station 363 + 92.39) of the alignment are likely to be near or below tunnel inverts. Dewatering may become necessary in a portion of this interval if the groundwater levels are substantially higher due to seasonal fluctuations.

In general, local small inflows due to perched water conditions can be anticipated during tunneling throughout the alignment.

6.3.3 Liquefaction Potential

Liquefaction of soils surrounding the tunnels may cause loss of support and excessive deformation/settlement of the tunnels. Thus, liquefaction may significantly impact the tunnel performance during and after a design earthquake event and is an important consideration.

A majority of alluvium along the Eastside Extension is granular in nature and consists of variable amounts of gravel and cobbles. High SPT blowcounts were observed in the granular alluvium along the alignment (Section 5.0 and boring logs in Appendix A). These high blowcounts are due to the presence of gravel and cobbles and do not reflect the consistency (denseness) of the overall granular deposits. They are therefore not reliable indicators of liquefaction potential of gravelly and cobbly soils. Additional blowcount data using a Becker hammer are needed to evaluate the liquefaction susceptibility of these gravelly and cobbly layers. For this investigation, liquefaction evaluation was therefore limited only to layers free of gravels or cobbles.

The results of our limited liquefaction evaluation performed for granular layers free of gravels or cobbles (Section 5.3) indicated the presence of potentially liquefiable medium dense sand layers, 2 to 11 feet thick, within and below the tunnel zone, in the vicinity of Borings PE-15, PE-16, PE-19, PE-22 and PE-25. The consequences of liquefaction within these zones may include localized loss of support around the tunnel and settlements on the order of a few inches.

The lined tunnel is not expected to experience any serious adverse impacts due to liquefaction of these layers. However, the impact of potential liquefaction within the gravelly and cobbly sand layers should be evaluated prior to final design. Additional liquefaction studies are particularly recommended between Union Station and the proposed Little Tokyo Station, and in the vicinity of the First/Boyle Station.

6.4 CUT-AND-COVER STATIONS

6.4.1 Excavation Methods

A total of seven cut-and-cover stations are proposed within the Eastside Extension alignment. Excavation and foundation support for five of these stations are anticipated to be completely within alluvium. Soft Fernando/Puente Formation bedrock is expected within at least portions of the other two stations (First/Boyle and Brooklyn/Soto). Thus, excavation and foundation support for these two stations may be partly or totally within bedrock. The subsurface geotechnical conditions at the station areas indicate that cut-and-cover excavation can be achieved using conventional mechanical excavation methods. However, suitable excavation equipment to handle large boulders within alluvium and for localized well-cemented hard interbeds within bedrock would likely be required.

6.4.2 Shoring Support

Shoring will be required due to the proximity of the stations to existing buildings and roads, and limited construction space along the alignment. Various shoring systems may be appropriate. These include various temporary walls such as sheet pile, soldier pile, precast, and slurry walls supported by tiebacks, anchors and/or internal bracing struts. The most appropriate shoring system must consider subsurface conditions, excavation geometry, the dewatering scheme, construction procedures, characteristics of nearby buildings, and local experience. Based on local practice in the Los Angeles area with subsurface geotechnical conditions similar to those encountered at Eastside Extension, soldier piles and lagging walls with tiebacks and/or internal

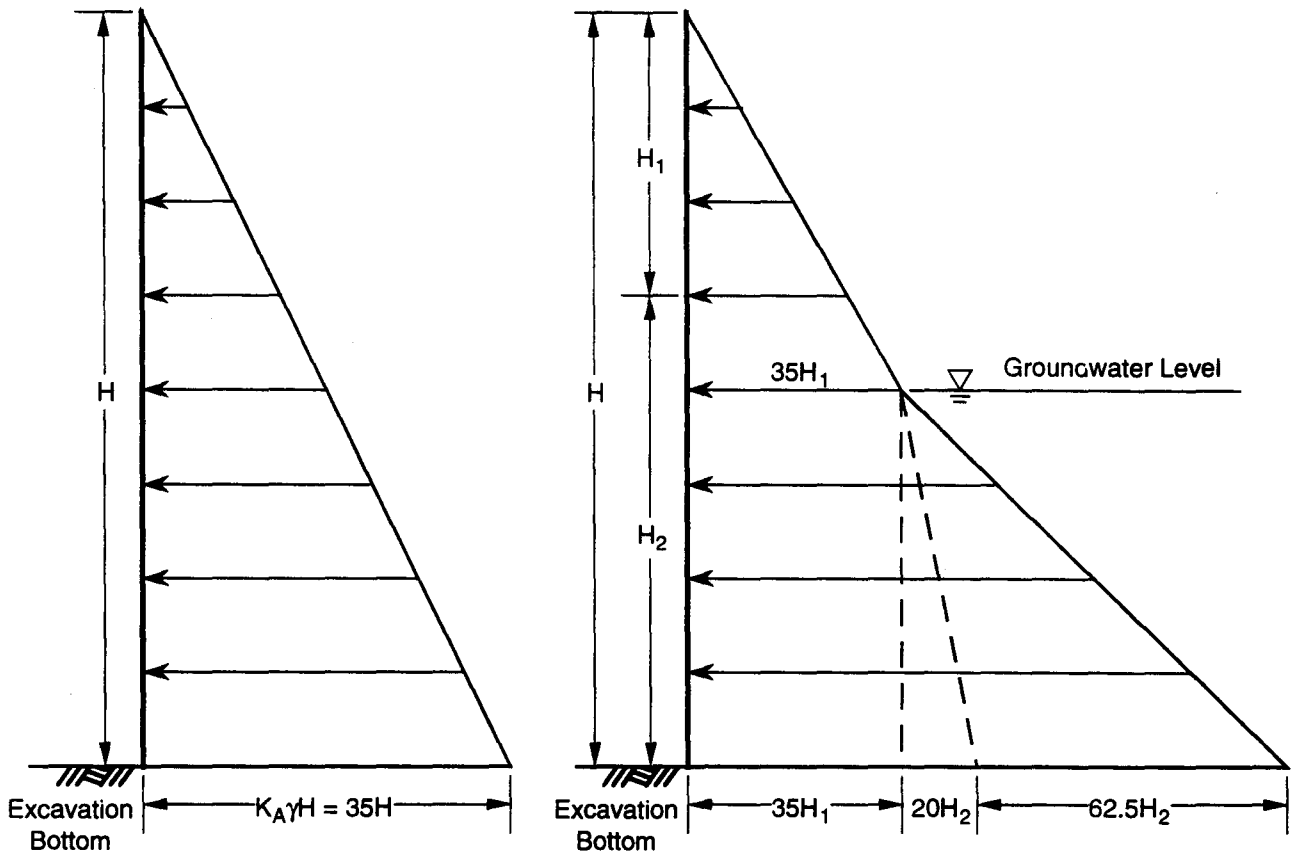
bracing (struts and wales) are the most likely shoring systems. The use of slurry wall construction for support of excavations in lieu of soldier piles and lagging would be relatively expensive and may not be practical.

Preliminary lateral earth pressure recommendations for shoring design in predominantly dense and granular alluvium are provided in Figures 6-1 and 6-2. At each station, site-specific lateral earth pressure distributions that consider material type, density (consistency), and surcharge effects should be developed prior to final shoring design. Design of the shoring system should also take into consideration the potential presence of cobbles and boulders, caving/flowing sands, high groundwater levels, local perched groundwater zones, and/or well cemented, hard interbeds within the bedrock.

Local noise abatement requirements, the presence of cobbles and boulders in alluvium, and the existence of Fernando/Puente Formation with local well-cemented hard interbeds generally preclude the use of conventional impact driving to install soldier piles. Thus, the soldier piles, if used, would likely be installed in predrilled holes. Rock coring of hard interbeds and large size boulders may be required. Slurry or casing will be required to handle potential caving conditions within granular alluvium.

6.4.3 Dewatering and Groundwater Control

Available groundwater data indicate that the groundwater table will be about 25 to 30 feet above the station invert at the First/Boyle Station. As indicated in Section 6.3.2 additional investigations will be required to establish possible groundwater fluctuations at the Little Tokyo Station. In the remaining five station areas, the groundwater table is anticipated to be near or below station inverts. The presence of thick, predominantly granular layers of alluvium below groundwater indicate that dewatering prior to and during excavation in the First/Boyle Station area, and possibly the Little Tokyo Station area, will be required. Additional field exploration, field pump testing and groundwater quality testing will be required prior to designing a suitable dewatering system.



(A) Dewatered behind Shoring

(B) Not Dewatered behind Shoring

Notes:

1) γ = unit weight of soil
2) K_A = active earth pressure coefficient
3) All earth pressures in psf
4) H, H_1, H_2 in feet
5) At each station, site-specific lateral earth pressure distributions that consider material type, density (consistency) and surcharge effects should be developed and used for final design of shoring

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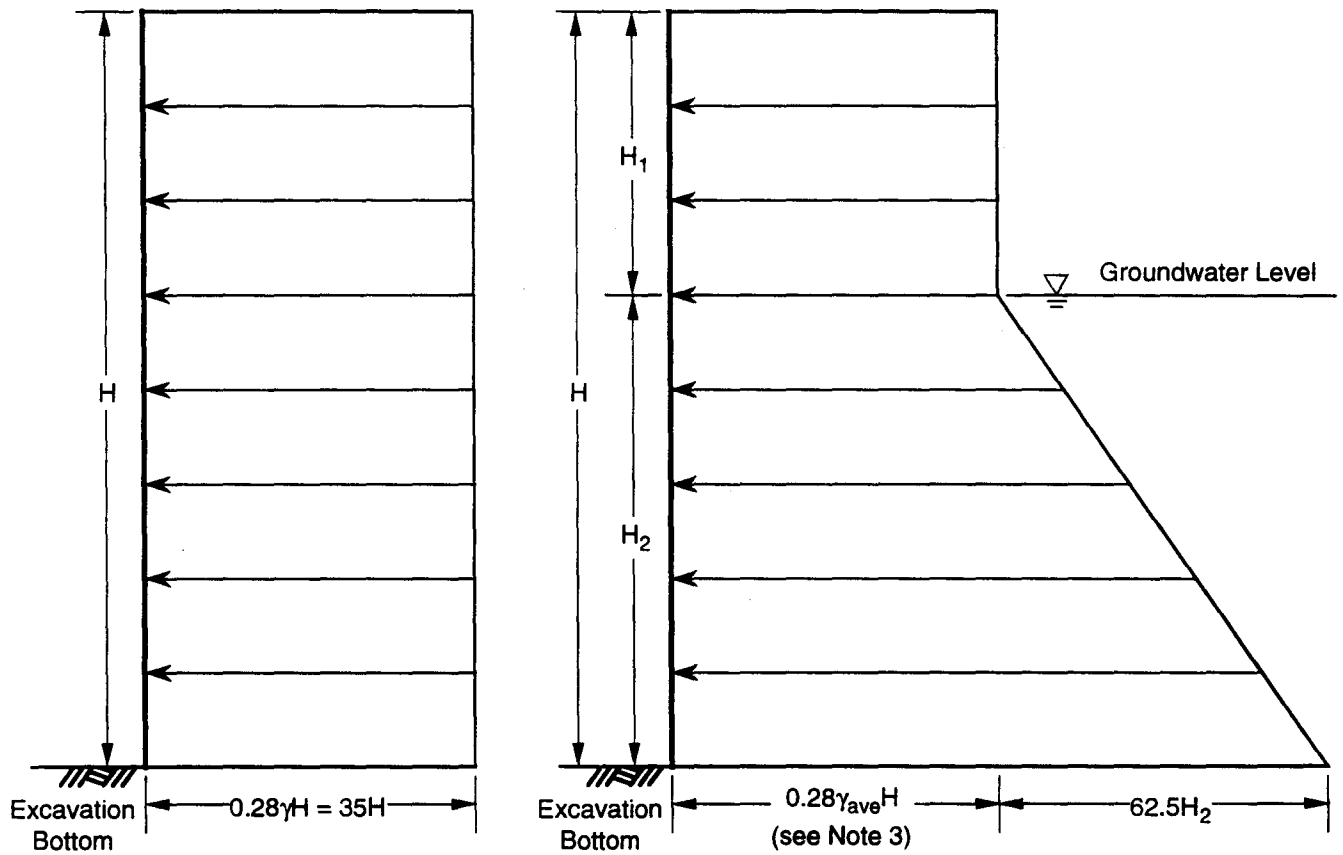
Preliminary Lateral Earth Pressure Distribution on Cantilevered Sheeting

2-94

Figure 6-1



(A) Dewatered behind Shoring

(B) Not Dewatered behind Shoring

Notes:

1) γ = unit weight of soil
2) γ_{sub} = submerged unit weight of soil
3) $\gamma_{ave} = \dfrac{\gamma H_1 + \gamma_{sub} H_2}{H_1 + H_2}$ = Average unit weight
4) All earth pressures in psf
5) H, H_1, H_2 in feet
6) At each station, site-specific lateral earth pressure distributions that consider material type, density (consistency) and surcharge effects should be developed and used for final design of shoring

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Project No. 94-1100
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Preliminary Lateral Earth Pressure Distribution on Braced Sheeting

2-94

Figure 6-2

Local small inflows due to localized perched groundwater zones can be anticipated during excavation and construction of all the proposed stations along the alignment.

6.4.4 Bottom Stability and Foundation Support

Subsurface materials at the excavation bottoms for the planned stations will be predominantly granular alluvium and Fernando/Puente Formation bedrock. Groundwater levels at all stations are anticipated to be near or below station inverts except at the First/Boyle Station and possibly the Little Tokyo Station. At these locations, dewatering will be required to lower the groundwater to mitigate the potential for bottom instability due to heaving, hydraulic uplift or piping.

In general, the materials encountered at the foundation level will provide adequate foundation support for the proposed structure. However, foundations may straddle transitions between bedrock and alluvium, and across different soil types within alluvium, with varying bearing (strength) and compressibility characteristics. Under such conditions, some foundation preparation measures such as overexcavation and recompaction may be necessary to limit potential differential settlements. Appropriate foundation types will depend on structure-loading characteristics which are not defined at this time. Foundation design recommendations can be provided after further structural and station-specific subsurface information becomes available.

6.4.5 Liquefaction Potential

Impacts of potential liquefaction in station areas may include loss of vertical and lateral support, increased lateral pressure on station walls, increased buoyancy and induced settlement/movement. Because of shallow groundwater conditions, liquefaction potential at the First/Boyle Station, and possibly the Little Tokyo Station, is of concern, while liquefaction potential at the remaining stations is relatively low due to deeper groundwater level. As stated in Section 6.3.3, further studies are required to evaluate the liquefaction potential of the gravelly/cobbly soils at the First/Boyle and Little Tokyo stations.

6.4.6 Structure/Street Protection

Most of the planned stations along the alignment will be close to existing structures and/or streets generally supported on foundations located above the planned depths of station excavation. Thus, provisions to protect these existing structures from potential damages due to station construction must be considered in the design and construction of the planned stations.

6.5 SOIL/GROUNDWATER CONTAMINATION

Limited data from this investigation (Sections 3.2 and 5.4) and available data from previous investigations near Union Station suggest that known or potential soil and groundwater contamination within the tunnel envelope and station excavation limits may exist at the following approximate locations:

- Known soil and groundwater contamination with petroleum hydrocarbons and hydrogen sulfide (H_2S) from Union Station to somewhere between Borings PE-18 and PE-29.

- Potential soil and local perched groundwater contamination with petroleum hydrocarbons in portions of the alignment along Whittier Boulevard in the vicinity of the active Thrifty Station LUST (Leaking Underground Storage Tank List - Regional Water Quality Control Board) site located at 3981 Whittier Boulevard.

- Potential contamination with hydrocarbons in the vicinity of Boring PE-15 as evidenced by the presence of high OVA readings of the samples from this boring.

As previously discussed, local perched groundwater conditions can be anticipated within alluvium, especially along the eastern segment. Depending on the locations, water inflows from some of the local perched water zones may be contaminated with hydrocarbons as evidenced by the high BTEX readings from the perched water samples from Monitoring Well PE-8.

There are several sites/facilities, as well as abandoned oil wells located close to the alignment, that may have past and current activities associated with underground storage tanks or other environmental implications. These sites may have potential impacts on soil and groundwater contamination along the alignment. High OVA readings were observed in Boring PE-15 which was located near an abandoned oil well.

A detailed environmental assessment has been conducted to identify potentially contaminated sites that may impact the alignment and to characterize the extent of contamination. The results of that assessment are discussed in a separate report.

6.6 GASSY CONDITIONS

The potential accumulation of methane and other gases within oil fields in the Los Angeles Basin is well known. The proposed alignment will traverse the known boundaries of the Union Station and Boyle Heights oil fields. Thus, the potential for accumulation of toxic and explosive gases, especially methane and hydrogen sulfide (H_2S), exists along the Eastside Extension segment, as evidenced by the following:

- Most of the soil samples from Borings PE-28 through PE-31 (within the Union Station Oil Field) and from Boring PE-15 (within the Boyle Heights Oil Field) obtained at or adjacent to the tunnel envelope exhibited high OVA readings

- Strong sulfur odors, possibly from the presence of H_2S, were documented during drilling and sampling of Borings PE-29 through PE-31 located within the Union Station Oil Field.

- Monitoring of the head spaces immediately above water samples from Monitoring Wells PE-29 through PE-31 (within the Union Oil Field) exhibited H_2S concentrations of 2.9 ppm to 46 ppm (permissible exposure limit is 10 ppm).

- Known soil and groundwater contamination with petroleum hydrocarbons and H_2S in the vicinity of the Union Station area.

The results presented above appear to suggest that the primary area of concern for H_2S along the alignment may be from the Union Station to somewhere between Borings PE-18 and PE-29. Areas of concern for methane include the portions of the alignment that traverse the known Union Station and Boyle Heights oil fields (approximately between Union Station and Boring PE-25, and between Borings PE-19 and PE-17, respectively).

In addition to being potentially present above the groundwater table within the area of concern, H_2S may also be present within the previously saturated zones that became unsaturated upon dewatering, as a result of H_2S occupying the voids created by dewatering. This possibility should be considered in the design, construction and operation of the facilities within the affected area.

6.7 POSSIBLE FAULT CROSSINGS

As discussed in Section 4.0, the Coyote Pass escarpment and associated lineaments, and a similar topographic escarpment located to the south may involve near-surface faulting and could cross the Eastside Extension alignment at as many as four places. Near-surface faulting has the potential of being one of the most significant concerns with respect to the tunnel design and construction. Detailed investigations to better understand the geometry and nature of these possible fault zones and their seismic activity and capability will be needed to assess potential impacts on safety issues, and design of the tunnel and stations.

6.8 OTHER CONCERNS

The Fernando and Puente formations are known to contain scattered zones of very hard concretionary nodules and cemented beds similar to those encountered in the construction of the first Metro Rail segment (MOS-1). Although this material was encountered at only one location

during this investigation, the possibility of its presence elsewhere along the Eastside Extension alignment exists.

If encountered, nodules up to 18 inches in diameter and zones of beds up to 4.5 feet in thickness should anticipated. Such nodules and beds may slow the rates of tunneling advance and station excavation as well as present difficulties for soldier pile installation.

Due to the proximity of Eastside Extension to two known oil fields (Union Station and Boyle Heights Oil Fields) where numerous exploratory/production wells exist, there exists a potential for the presence of undocumented cased or uncased abandoned oil wells within the tunnel envelope and station excavation limits, especially along the portion of the alignment located within these known oil fields. In addition to requiring considerable time to remove the casings, such abandoned wells, if encountered, may contain large quantities of water or even oil under pressure which can rush into the tunnel or station excavation within a few seconds. The abandoned wells may also contain residual accumulations of hydrogen sulfide, methane or other toxic/explosive gases.

Results of sulfate content tests indicate that the soils are mildly to moderately corrosive to concrete except near the location of Boring PE-2, where the soil may be severely corrosive (maximum sulfate content in excess of 2,000 ppm). Type II cement should be used for concrete in contact with mildly to moderately corrosive soil. Type V cement is required for concrete in contact with soils containing sulfates in excess of 2,000 ppm. Results of laboratory electrical resistivity tests indicated that the soils are predominantly moderately to extremely corrosive to metals. As indicated in Section 5.2.4, samples from 17 of the 31 borings tested very corrosive to metals (electrical resistivity less than 2,000 ohm-cm). Most of the remaining samples tested moderately corrosive (electrical resistivity between 2,000 and 5,000 ohm-cm). Very corrosive samples were found in the fine and coarse alluvium (within the tunnel zone and near surface) and bedrock.

Section 7.0

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

Although more detailed geologic and geotechnical investigation programs will be needed to support the future engineering effort for the proposed Eastside Extension, the results of this preliminary geotechnical investigation have provided a needed database for a general understanding of the geologic, geotechnical and environmental conditions, and a preliminary characterization of associated engineering parameters and potential ground behavior along the alignment.

As previously discussed, the alignment was divided into the following two segments based on subsurface conditions:

- Western segment - about 1.5 miles long, from Union Station to about 300 feet east of Boring PE-22, approximate Station 93+00.

- Eastern segment - about 5.1 miles long from Station 93+00 to the eastern terminus of the Eastside Extension.

Based on the results of this investigation and current plans and profiles, tunnel and station excavations within the alignment will be predominantly within alluvium, with some portions in bedrock of the Fernando/Puente Formations. Alluvium in the western segment is Holocene-age and consists predominantly of coarse-grained materials ranging from sands to cobbles and boulders of up to 4 feet in size with occasional layers of fine-grained materials. Alluvium in the eastern segment is generally more fine-grained and contains less cobbles and boulders (both in extent and size).

Based on the results of this investigation, the following conclusions can be made:

1. The tunnels along the Eastside Extension can be advanced using soft ground/soft rock mechanical excavation equipment within a shield and with initial support consisting of precast concrete liners.

2. Identified potential concerns for tunneling along the alignment include the presence of cobbles and boulders (up to 4 feet, and greater than 18 inches along the western and eastern segments, respectively), shallow groundwater conditions within alluvium along portions of the alignment, raveling and running/flowing conditions associated with the predominantly granular alluvium within the tunnel envelope, local hard, well-cemented interbeds and the potential presence of hard concretionary nodules (up to 18 inches in size) within the bedrock, and presence of hydrogen sulfide and methane.

3. Topographic features that are inferred to be fault related intersect the alignment at several locations. The "Coyote Pass fault" may potentially cross the alignment at as many as three locations. A similar feature located to the south crosses the eastern portion of the alignment. If the "Coyote Pass fault" and the southerly inferred fault are found to be active, such a finding will become one of the most significant constraints in the design and construction of the tunnel and station facilities.

4. To enhance face stability and reduce ground settlement, dewatering to lower groundwater below tunnel inverts along with specific provisions, such as, a shield with a movable hood (poling plates) and breasting doors, use of an earth pressure balance (EPB) machine, or stabilization of granular soil zones, will be necessary. Initial lining support and backfilling of tail voids should be applied in a timely manner.

5. Large boulders and hard interbeds in the bedrock will likely reduce advance rates and may require splitting at the face or in the mucking conveyor.

6. Hydrogen sulfide (H_2S) is likely to be present, primarily from Union Station to somewhere between Borings PE-29 and PE-18, where a barrier to H_2S and groundwater contamination may exist. The H_2S is a significant concern for tunnel construction and operation.

7. Methane may be encountered along those portions of the alignment traversing the Union Station and Boyle Heights oil fields. The potential presence of methane and other gases should be considered in the design and construction of the tunnels and stations.

8. Excavation of the seven planned cut-and-cover stations along the alignment can be accomplished using mechanical excavation methods with readily available equipment and conventional shoring provisions. Again, the presence of large boulders within alluvium and local well-cemented hard interbeds within bedrock may require special handling and may slow the rates of excavation and shoring installation.

9. Preconstruction dewatering will be required at the First/Boyle station where groundwater levels are expected to be about 30 feet above the station invert. High groundwater levels may also impact the excavation at the Little Tokyo Station. Additional studies will be required to estimate the potential for high groundwater level fluctuations in this area. Preconstruction dewatering for the other five stations does not appear to be necessary. However, localized inflows due to perched groundwater conditions can be anticipated.

10. Local zones of potentially liquefiable layers, 2 to 11 feet thick, exist within and below the tunnel envelope. However, data from this investigation were not

sufficient to perform a proper evaluation of the liquefaction potential in areas of gravelly and cobbly alluvium. These areas include the portion of alignment from Union Station to the vicinity of Boring PE-28, and the areas within the limits of the First/Boyle and Little Tokyo stations. Additional investigations by Becker hammer to delineate liquefaction potential in coarse materials will be necessary.

11. Available project data files and limited chemical tests on groundwater samples obtained in this investigation indicate potential groundwater and soil contamination in some areas along the alignment. These include soil and groundwater contamination with hydrocarbons and H_2S from Union Station to somewhere between Borings PE-18 and PE-29, as well as potential soil and local perched groundwater contamination with hydrocarbons and/or metals or chemical compounds in the immediate vicinity of known boundaries of the Union Station and Boyle Heights oil fields, known active LUST sites, and areas with past and current activities that may have a potential for contamination. In addition to impacting disposal, groundwater contamination will affect the details and requirements of dewatering as well as other important issues such as the presence of H_2S in unsaturated zones produced by dewatering.

12. Subsurface soils are moderately to extremely corrosive to metals. For the most part, soils are mildly to moderately corrosive to concrete. Type II cement should be adequate for most of the alignment. However, occasional corrosive zones that may require Type V cement exist along the alignment.

7.2 RECOMMENDATIONS

In addition to providing a needed database for preliminary engineering design, the results of this investigation have also identified a number of constraints and data gaps. Further investigations will be needed to support future design and construction activities. Future investigations should

include geologic, geotechnical, geophysical, hydrogeologic and environmental assessments. These investigation programs should include, but not necessarily be limited to the following:

1. Perform a detailed geotechnical investigation program with closely-spaced geotechnical borings along the entire alignment for a more detailed understanding of the subsurface conditions. A majority of the borings should be drilled and sampled using large diameter bucket auger rigs (above groundwater table) and Becker drill rigs (above and below groundwater) in order to evaluate the following: extent and size distribution of cobbles and boulders; representative grain size distribution (large bulk samples) and consistency (as indicated by the penetration resistance to the Becker hammer) of gravelly and cobbly soils; liquefaction potential assessment in shallow groundwater areas from Union Station to somewhere between Borings PE-28 and PE-29, and within the First/Boyle and Little Tokyo Station areas.

2. Carry out a detailed fault study program, including additional geologic mapping, borings, trenches, and geophysical surveys, to evaluate faulting, folding and potential seismic activity in connection with the Coyote Pass fault and a similar inferred fault to the south, and their potential impacts on the Eastside Extension tunnel design. The program should include extended lines of borings at various locations across the escarpment to better characterize the geometry of folding, trenches on the escarpment and across inferred offset stream channels to search for both evidence of near-surface faulting and deposits that might permit an understanding of the timing of tectonic activity, and geophysical studies to delineate possible offset bedrock at depth.

3. Conduct a geophysical program to evaluate seismic wave velocities to assist in liquefaction potential assessment for shallow groundwater areas.

4. Perform closely-spaced geotechnical borings with multi-stage piezometers/monitoring wells supplemented by a geophysical survey in the area between Borings PE-18 and PE-29 where a barrier (apparently either due to high bedrock surface or the existence of a

fault zone) to groundwater contamination and H_2S appears to be present. Delineation of the barrier location may define the southern boundary of H_2S and/or significant groundwater contaminations.

5. Install additional piezometers and monitoring wells to supplement the existing data and to better define the geohydrological settings including groundwater levels and quality along the alignment, and to resolve the groundwater level discrepancy between this investigation and the 1983 data by others, especially between Borings PE-29 and PE-24.

6. Perform field pump tests in the areas that require pre-construction dewatering for tunnel and station construction, as well as determine groundwater quality to help design suitable dewatering systems and treatment systems, if required.

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Section 8.0

Section 8.0

8.0 REFERENCES

Blake, T.F., 1992. EQSEARCH - A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs. IBM-PC Version 2.0.

Bonilla, M.G., Mark, R.K., and Lienkaemper, J.J., 1984. Statistical Relations Among Earthquake Magnitude, Surface Rupture Length, and Surface Fault Displacement. Bulletin of the Seismological Society of America, Vol. 74, No. 6, p. 2379-2411.

Brown and Caldwell Consultants, 1993. Revised Workplan for Verification Sampling of the Western Pump Island. Area Station No. 6153. Prepared for Los Angeles County Department of Public Works, dated September 20, 1993.

Bullard, T.F., and Lettis, W.R., 1993. Quaternary Fold Deformation Associated with Blind Thrust Faulting, Los Angeles Basin, California. Journal of Geophysical Research, Vol. 98, No. B-5, pp. 8349-8369.

California Department of Conservation, 1988. Seismic Intensity Distribution, in Planning Scenario for a Major Earthquake on the Newport-Inglewood fault zone. Special Publication 99, Map 55.

California Department of Transportation (Caltrans), 1985a. Log of Test Borings, Los Angeles River Busway Bridge and Overhead, Bridge No. 53-2673, 4 sheets.

California Department of Transportation (Caltrans), 1985b. Left Retaining Wall, Log of Test Borings, Los Angeles River Busway Bridge and Overhead, Bridge No. 53-2673, 2 sheets.

California Department of Transportation (Caltrans), 1964. Log of Test Borings, Bridge No. 53-1150.

California Department of Transportation (Caltrans), 1963. Log of Test Borings, Route 165/70 Separation, Bridge No. 53-101.

California Department of Transportation (Caltrans), 1957a. Log of Test Borings, First Street Undercrossing, Bridge No. 53-1305.

California Department of Transportation (Caltrans), 1957b. Log of Test Borings, Brooklyn Avenue Overcrossing, Bridge No. 53-1314.

California Department of Transportation (Caltrans), 1953. Log of Test Borings, Bridge No. 53-881, 2 sheets.

California Department of Water Resources, Southern District, 1961. Planned Utilization of The Groundwater Basins of the Coastal Plain of Los Angeles County. Appendix A - Groundwater Geology, Bulletin No. 104.

Converse, Davis and Associates, 1975. Geologic Investigation, City Terrace Trunk Sewer Section 1 (Tunnel), HUD Project No. WS California - 394, Contract No. H-602-4069. Prepared for Los Angeles County Sanitation District No. 2, dated May 14, 1975.

Converse, Davis and Associates, 1973. Engineering Investigation, Proposed City Terrace Trunk Sewer Section 3, City Terrace, East Los Angeles, Los Angeles County, California. Prepared for Los Angeles County Sanitation District, dated January 19, 1973.

Converse, Davis and Associates, 1972. Geologic Investigation, Proposed City Terrace Trunk Sewer, Section 3, City Terrace, East Los Angeles, Los Angeles County, California. Prepared for County Sanitation District, dated December 15, 1972.

Converse Consultants, Earth Sciences Associates, Geo/Resource Consultants, 1984. Geotechnical Report, Metro Rail Project, Design Unit A100. Prepared for Metro Rail Transit Consultants, dated February 1984.

Converse Consultants, Inc., Earth Sciences Associates and Geo/Resource Consultants, 1983. Seismological Investigation and Design Criteria. Prepared for Southern California Rapid Transit District Metro Rail Project, dated May 1983.

Converse, Ward, Davis, Dixon, Earth Science Associates, Geo/Resource Consultants, 1981. Southern California Rapid Transit District, Metro Rail Project, Geotechnical Investigation Report, Volumes I and II. Prepared for Southern California Rapid Transit District, dated December 21, 1981.

County of Los Angeles, Department of Regional Planning, 1990. Liquefaction Susceptibility, Plate 4.

CTL Environmental Services, 1992. Site Assessment for Underground Storage Tank Area, Calvary Cemetery, 4201 Whittier Boulevard, Los Angeles, California, Los Angeles County Department of Public Works, File No. I-15743. Prepared for Archdiocese of Los Angeles, 1531 West 9th Street, Los Angeles, California dated 1992.

Dibblee, T.W., Jr., 1989. Geologic Map of the Los Angeles Quadrangle, Los Angeles County, California. Dibblee Geologic Foundation, Map DF-22.

Dolan, J. 1993. Personal Communication.

Dolan, J.F. and Sieh, K., 1992a. Paleoseismology and Geomorphology of the Northern Los Angeles Basin: Evidence for Holocene Activity on the Santa Monica Fault and Identification of New Strike-Slip Faults through Downtown Los Angeles, in EOS, Transactions of the American Geophysical Union, Vol. 73, p. 589.

Dolan, J.F. and Sieh, K., 1992b. Tectonic Geomorphology of the Northern Los Angeles Basin: Seismic Hazards and Kinematics of Young Fault Movement, in Engineering Geology Field Trips, Orange County, Santa Monica Mountains and Malibu. Association of Engineering Geologists, Southern California Section, 35th Annual Meeting, p. B-20 to B-26.

Dolan, J.F. and Sieh, K., 1992c. Structural Style and Tectonic Geomorphology of the Western Los Angeles Basin - Seismic Hazards and Kinematics of Young Fault Movement, in Engineering Geology Field Trips, Orange County, Santa Monica Mountains and Malibu. Association of Engineering Geologists, Southern California Section, 35th Annual Meeting, p. B-27 and B-28.

The Earth Technology Corporation, 1987a. Subsurface Investigation at the Metro Rail Realigned A-130 Corridor, Los Angeles, California. Prepared for Metro Rail Transit Consultants, dated December 23, 1987.

The Earth Technology Corporation, 1987b. The Phase IV Subsurface Investigation near the Metro Rail A-130 Corridor, Los Angeles, California. Prepared for Metro Rail Transit Consultants, dated September 1987.

The Earth Technology Corporation, 1987c. The Phase III Subsurface Investigation near the Metro Rail A-130 Corridor, Los Angeles, California. Prepared for Metro Rail Transit Consultants, dated April 24, 1987.

The Earth Technology Corporation, 1987d. The Phase I Subsurface Investigation at the Metro Rail A-130 Corridor, Los Angeles, California. Prepared for Metro Rail Transit Consultants, dated February 10, 1987.

The Earth Technology Corporation, 1986. The Subsurface Investigation at the Metro Rail A-130 Corridor, Los Angeles, California. Prepared for Metro Rail Transit Consultants, dated December 22, 1986.

Emcon Southwest, 1992. Fuel Storage and Waste-Oil Tanks and Sumps Closure Report, 3600 East Third Street at Indiana Street, Los Angeles, California, WIC No. 204-4530-5202. Prepared for Shell Oil Company, dated March 1992.

Emcon Southwest, 1993. First Quarter 1993 Progress Letter, Former Shell Service Station WIC No. 204-4530-5202, 3600 East Third Street at Indiana Street, Los Angeles, California. Prepared for Los Angeles County Department of Public Works, dated June 21, 1993.

Greensfelder, R.W., 1974. Maximum Credible Rock Acceleration from Earthquakes in California. California Division of Mines and Geology, Map Sheet 23.

Hart, E.W., 1990. Fault Rupture Hazard Zones in California, Revised 1990, with Addendum. California Division of Mines and Geology, Special Publication 42.

Jennings, C.W., 1992. Preliminary Fault Map of California. California Division of Mines and Geology, Open-File Report 92-3.

Joyner, W.B., and Boore, D.M., 1982. Prediction of Earthquake Response Spectra. U.S. Geological Survey, Open-File Report 82-977, 16 p.

Knecht, 1971. Soil Survey, Western Riverside Area, California. U.S.D.A. Soil Conservation Service, 157p.

Lamar, D.L., 1970. Geology of the Elysian Park - Repetto Hills area, Los Angeles County, California. California Division of Mines and Geology, Special Report 101, 45p.

LeRoy Crandall and Associates, 1970. Final Report of Geological Investigation, City Terrace Trunk Sewer, Section 1, Project No. 72-223-NH. Prepared for County Sanitation District No. 2 of the County of Los Angeles, dated July 6, 1979.

Levin-Fricke, 1993. Monthly Discharge Monitoring Report, Gateway Center Construction Site, Los Angeles, California, October 1993, NPDES Permit No. CA0063134, Order No. 93-024, Compliance File No. CI7267.

Mualchin, L. and Jones A.L., 1992. Peak Acceleration from Maximum Credible Earthquakes in California (Rock and Stiff Soils). California Division of Mines and Geology, Open-File Report 92-1.

Ponti, D.J., 1985. The Quaternary Alluvial Sequence of the Antelope Valley, California: Geological Society of America, Special Paper 203, p 79-96.

Regional Water Quality Control Board - Los Angeles, 1993. Waste Discharge Requirements - Catellus Development and Southern California Rapid Transit District (Gateway Center Construction Site) (NPDES Permit No. CA0063134).

Seed, H.B., 1987. Design Problems in Soil Liquefaction. Journal of Geotechnical Engineering. ASCE, Vol. 113, No. GT3, p. 827-845.

Seed, H. B., Idriss, I.M. and Arango, I., 1983. Evaluation of Liquefaction Potential Using Field Performance Data. Journal of the Geotechnical Engineering, Vol. 109, No. 3, March 1983.

Sieh, K. 1994. Personal Communication.

Sieh, K. 1993. Letter to Engineering Management Consultant, Chief Tunnel Engineer. February 19, 1993.

Thrifty Oil Company, 1993. Supplemental Information, Vapor Extraction Test at Thrifty Oil Company Service Station #032, 3981 East Whittier Boulevard, East Los Angeles, California, File #I-10935. Prepared for County of Los Angeles Department of Public Works, dated May 5, 1993.

Tinsley, J.C., Youd, T.L., Perkins, D.M., Chen, A.T.F., 1985. Evaluating Liquefaction Potential, in Evaluating Earthquake Hazards in the Los Angeles Region - An Earth-Science Perspective. U.S. Geologic Survey, Professional Paper 1360, p. 263-315.

Treiman, J.A., 1991. Whittier Fault Zone. California Division of Mines and Geology, Fault Evaluation Report, FER-222, 17p.

U.S. Department of Interior, Bureau of Reclamation, undated. Engineering Geology Field Manual.

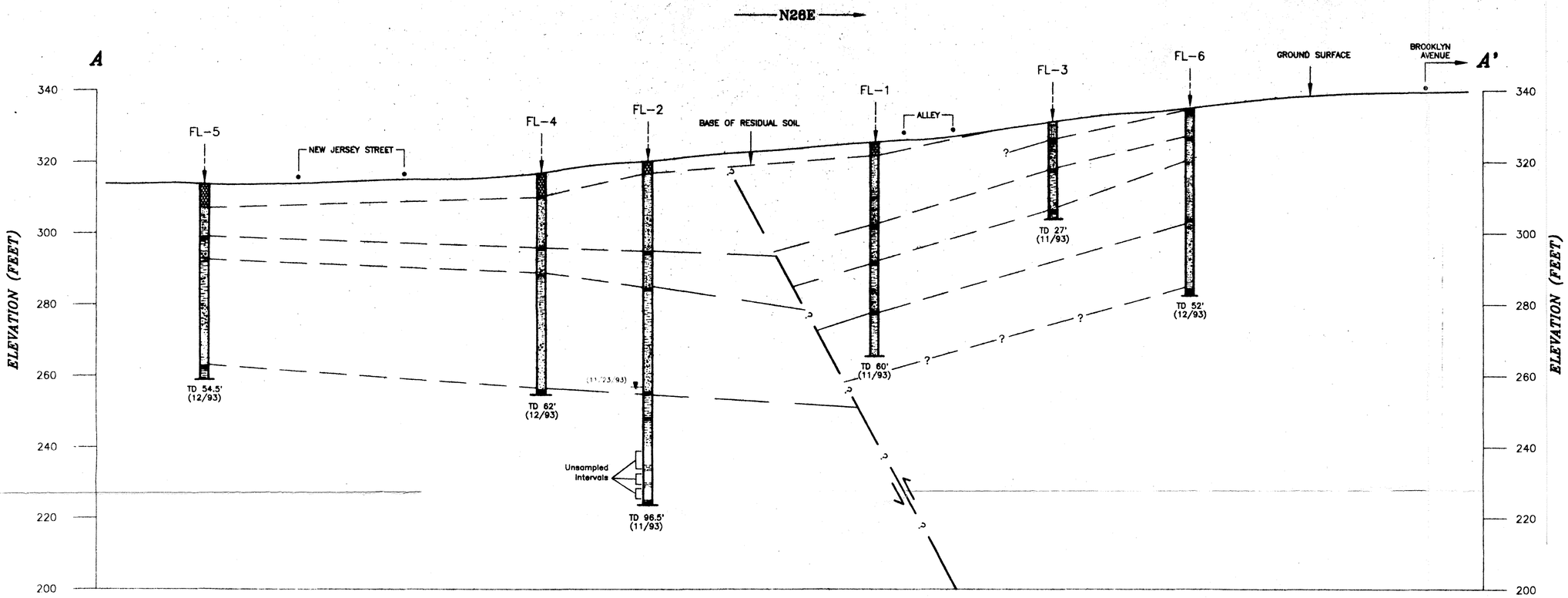
Wachtell, J.K., 1978. Soil Survey of Orange County and the Western Part of Riverside County, California. U.S.D.A. Soil Conservation Service, 149p.

Woodruff, G.A., McCay, W.J., an Sheldon, W.B., 1970. Soil Survey, Antelope Valley, California. U.S.D.A. Soil Conservation Service, 187p.

Woodward Clyde Consultants, 1986. Hazardous Materials Investigation at the Construction Site of the Los Angeles Busway, Volumes 1 and 2. Prepared for C.C. Meyers, Inc., Pico Rivera, California.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., and Vedder, J.G., 1965. Geology of the Los Angeles Basin, California-An Introduction. U.S. Geological Survey Professional Paper 420-A, 57p.

Ziony, J.I. and Yerkes, R.F., 1985. Evaluating Earthquake and Surface-Faulting Potential, in Evaluating Earthquake Hazards in the Los Angles Region - An Earth-Science Perspective. U.S. Geological Survey, Professional Paper 1360, p. 43-91.

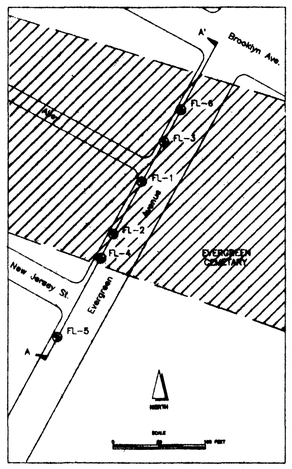


Scale: 1 inch = 20 feet (Horizontal & Vertical)

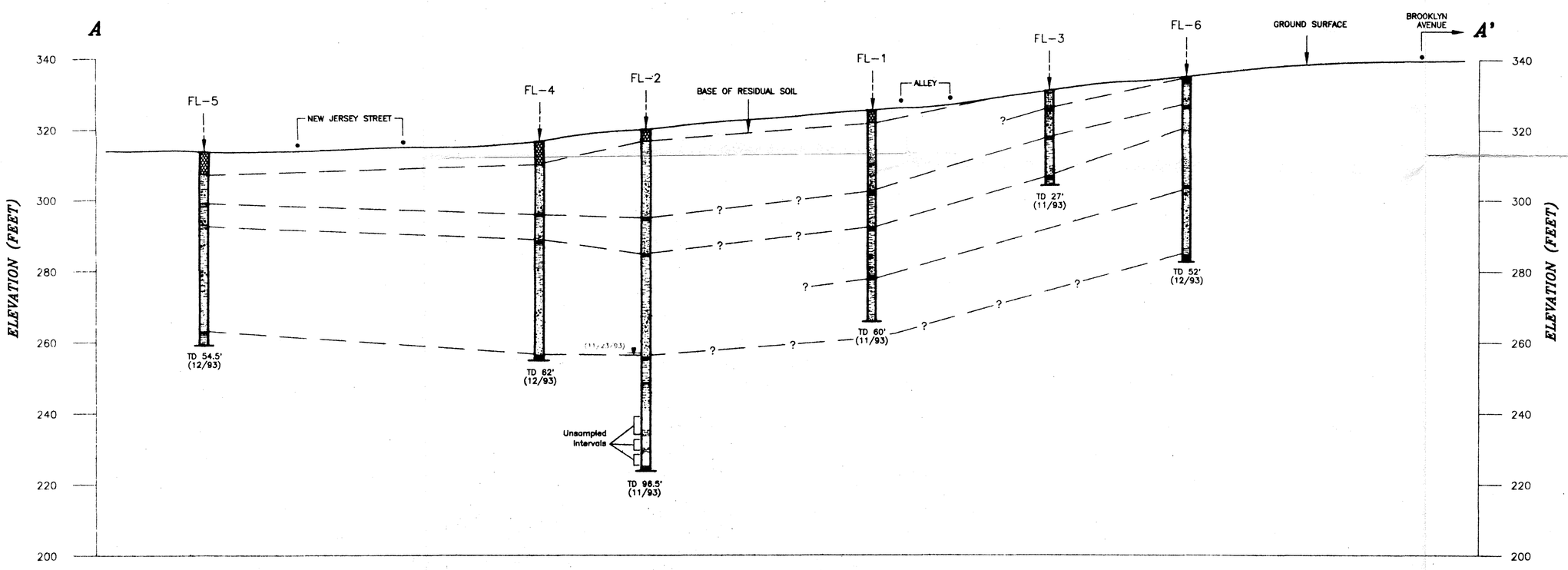
0 20 40 FEET
SCALE

FAULT INTERPRETATION

LOCATION MAP



Notes:
1) Hatchered area is escarpment slope inclined to the south.
2) See Plate 1A for location with respect to alignment.



Scale: 1 inch = 20 feet (Horizontal & Vertical)

0 20 40 FEET
SCALE

FOLD INTERPRETATION

Borehole Explanation

RESIDUAL SOIL
CLAY
SILT
sandy
gravelly
SAND
GRAVEL

Interpreted correlation of clay horizons encountered in borings

Groundwater elevation measured on date indicated

Note: Residual soil is a pedogenic soil horizon developed in-situ at the ground surface by weathering processes.

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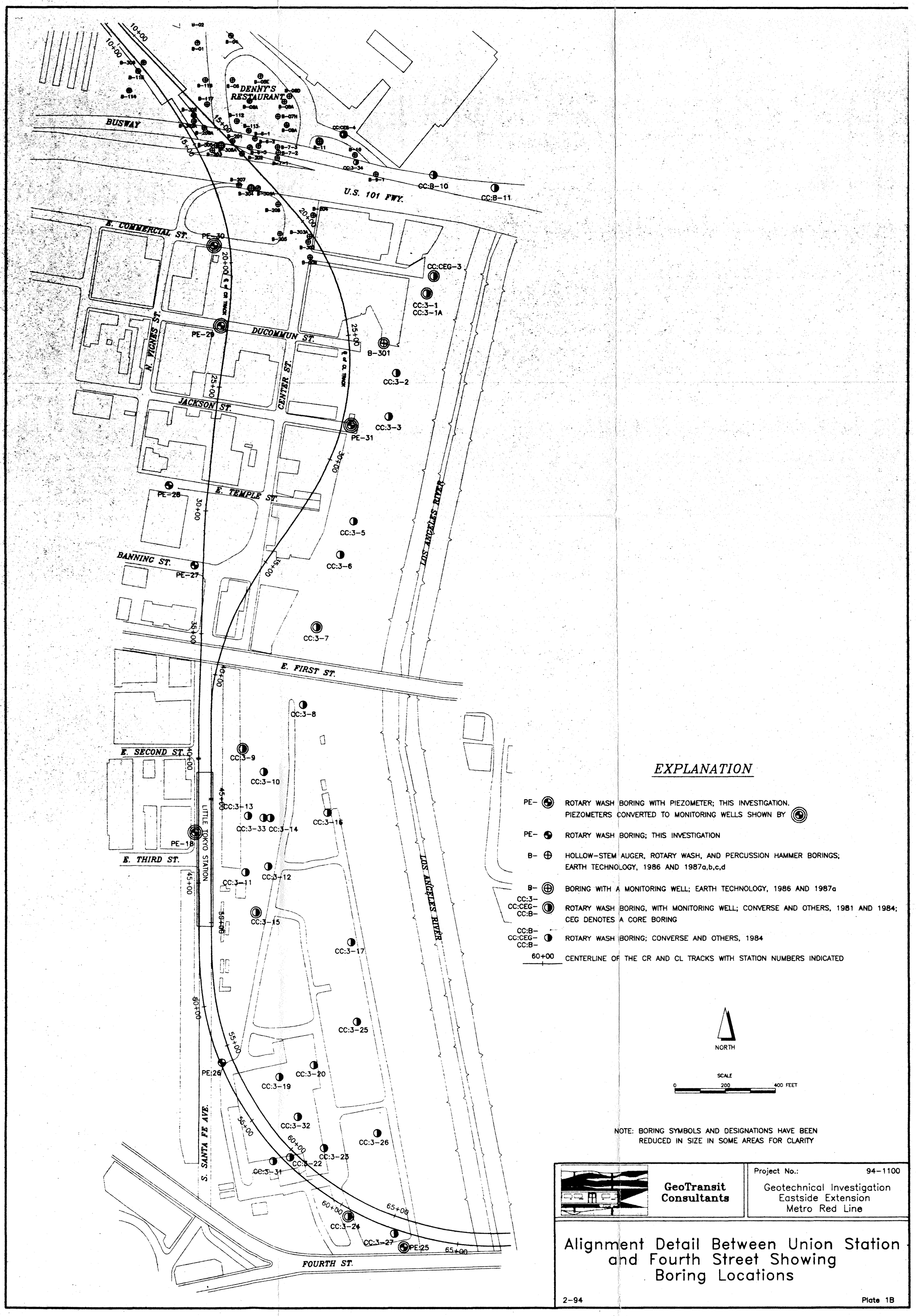
Geotechnical Investigation
Eastside Extension
Metro Red Line

Cross Section A-A' Across the "Coyote Pass Fault" Escarpment

2-94

Plate 3


DENNY'S RESTAURANT
BUSWAY
U.S. 101 FWY.
E. COMMERCIAL ST.
DUCOMMUN ST.
N. VIGNES ST.
CENTER ST.
JACKSON ST.
E. TEMPLE ST.
BANNING ST.
E. FIRST ST.
E. SECOND ST.
E. THIRD ST.
FOURTH ST.
S. SANTA FE AVE.
LITTLE TOKYO STATION
LOS ANGELES RIVER
CR TRACK
CL TRACK
PE-30
PE-29
PE-31
PE-28
PE-27
PE-18
PE-26
PE-25
B-301
CC:B-10
CC:B-11
CC:CEG-3
CC:3-1
CC:3-1A
CC:3-2
CC:3-3
CC:3-5
CC:3-6
CC:3-7
CC:3-8
CC:3-9
CC:3-10
CC:3-13
CC:3-33
CC:3-14
CC:3-16
CC:3-11
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CC:3-15
CC:3-17
CC:3-25
CC:3-20
CC:3-19
CC:3-32
CC:3-26
CC:3-23
CC:3-22
CC:3-31
CC:3-24
CC:3-27
10+00
15+00
20+00
25+00
30+00
35+00
40+00
45+00
50+00
55+00
60+00
65+00
EXPLANATION
PE- ROTARY WASH BORING WITH PIEZOMETER; THIS INVESTIGATION. PIEZOMETERS CONVERTED TO MONITORING WELLS SHOWN BY
PE- ROTARY WASH BORING; THIS INVESTIGATION
B- HOLLOW-STEM AUGER, ROTARY WASH, AND PERCUSSION HAMMER BORINGS; EARTH TECHNOLOGY, 1986 AND 1987a,b,c,d
B- BORING WITH A MONITORING WELL; EARTH TECHNOLOGY, 1986 AND 1987a
CC:3- CC:CEG- CC:B- ROTARY WASH BORING, WITH MONITORING WELL; CONVERSE AND OTHERS, 1981 AND 1984; CEG DENOTES A CORE BORING
CC:B- CC:CEG- CC:B- ROTARY WASH BORING; CONVERSE AND OTHERS, 1984
60+00 CENTERLINE OF THE CR AND CL TRACKS WITH STATION NUMBERS INDICATED
NORTH
SCALE
0 200 400 FEET
NOTE: BORING SYMBOLS AND DESIGNATIONS HAVE BEEN REDUCED IN SIZE IN SOME AREAS FOR CLARITY
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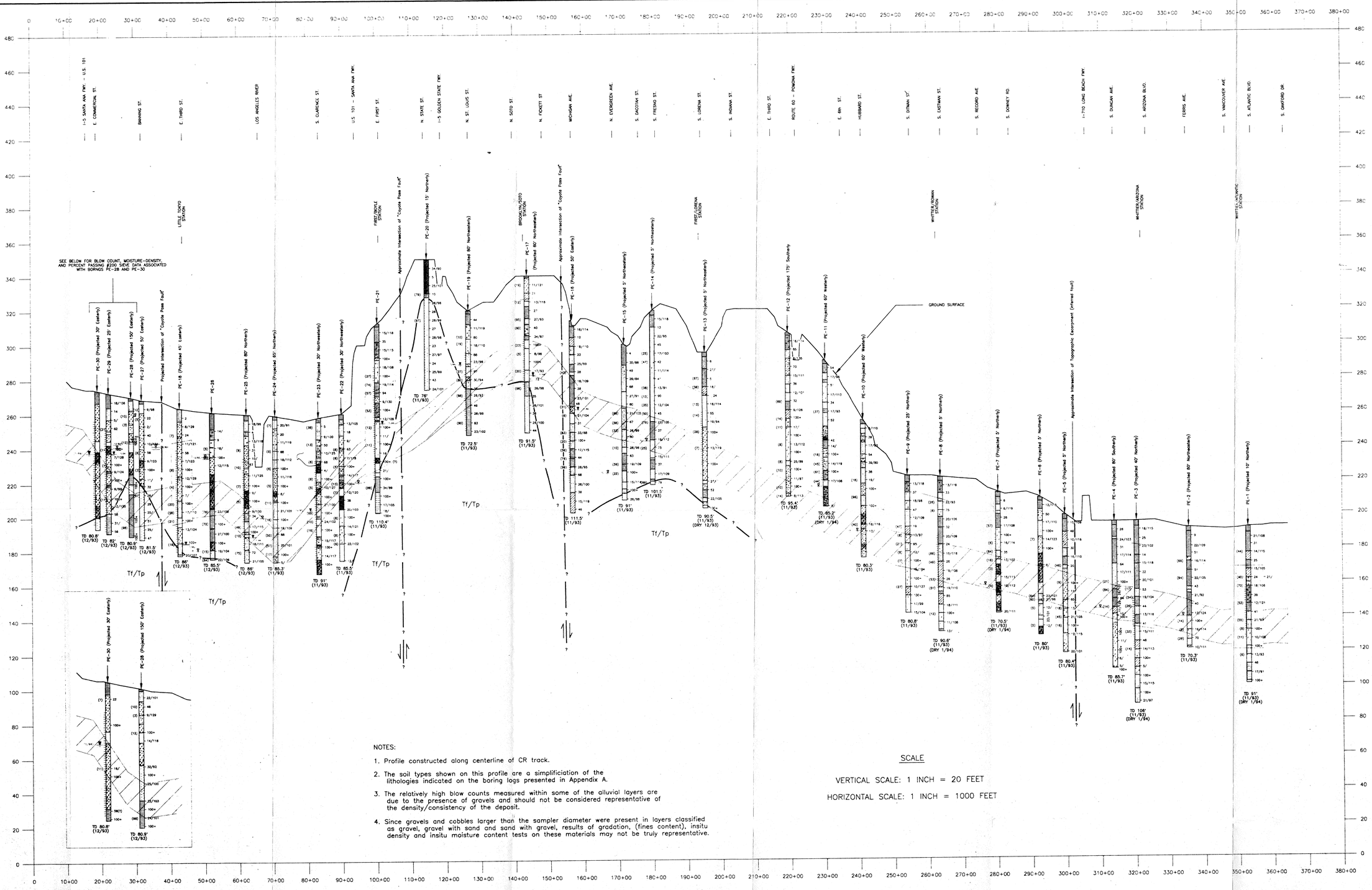
Geotechnical Investigation
Eastside Extension
Metro Red Line
Alignment Detail Between Union Station and Fourth Street Showing Boring Locations
2-94
Plate 1B



NOTES:

1. Profile constructed along centerline of CR track.
2. The soil types shown on this profile are a simplificiation of the lithologies indicated on the boring logs presented in Appendix A.
3. The relatively high blow counts measured within some of the alluvial layers are due to the presence of gravels and should not be considered representative of the density/consistency of the deposit.
4. Since gravels and cobbles larger than the sampler diameter were present in layers classified as gravel, gravel with sand and sand with gravel, results of gradation, (fines content), insitu density and insitu moisture content tests on these materials may not be truly representative.

SCALE

VERTICAL SCALE: 1 INCH = 20 FEET

HORIZONTAL SCALE: 1 INCH = 1000 FEET

SYMBOLS

- SILT, SILT with SAND (ML)
- SANDY SILT (ML)
- CLAY, CLAY with SAND (CL)
- SANDY CLAY (CL)
- CLAY (CH)
- SAND (SP, SW, SP-SM, SP-SC, SW-SM, SW-SC)
- SAND with GRAVEL (SP, SW, SP-SM, SP-SC, SW-SM, SW-SC)
- SILTY SAND, SILTY SAND with GRAVEL (SM)
- CLAYEY SAND, CLAYEY SAND with GRAVEL (SC)
- GRAVEL, GRAVEL with SAND (GP, GW, GP-GM, GP-GC, GW-GM, GW-GC)
- SILTSTONE
- SANDSTONE with SILTSTONE INTERBEDDED
- CLAYSTONE
- FILL
- BEDROCK CONTACT, queried where uncertain

Tf/Tp: FERNANDO/PUENTE FORMATION

FAULT; dashed where approximately located, queried where uncertain; arrows indicate direction of relative displacement

Percent Passing #200 Sieve Indicated in Parenthesis = (68)

Screen Interval

- (11/93) Groundwater Level Reported by Converse and others, 1984
- Possible Groundwater Occurrence During Drilling
- (11/93) Groundwater Level and Date Measured
- 16/102 = Moisture Content (%) / Dry Density (pcf)
- 16/ = Moisture Content (%) Only
- 21 = SPT Blow Count for Final 12 Inches of Penetration
- Cobbly and /or Bouldery Zone; Cemented Bedrock

TD 90.5' (11/93) (DRY 12/93) — Total Depth of Boring; Date Drilled; Dry Hole and Date Measured

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Geotechnical Investigation
Eastside Extension
Metro Red Line

Subsurface Profile of the Eastside Extension

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PLATE 2A