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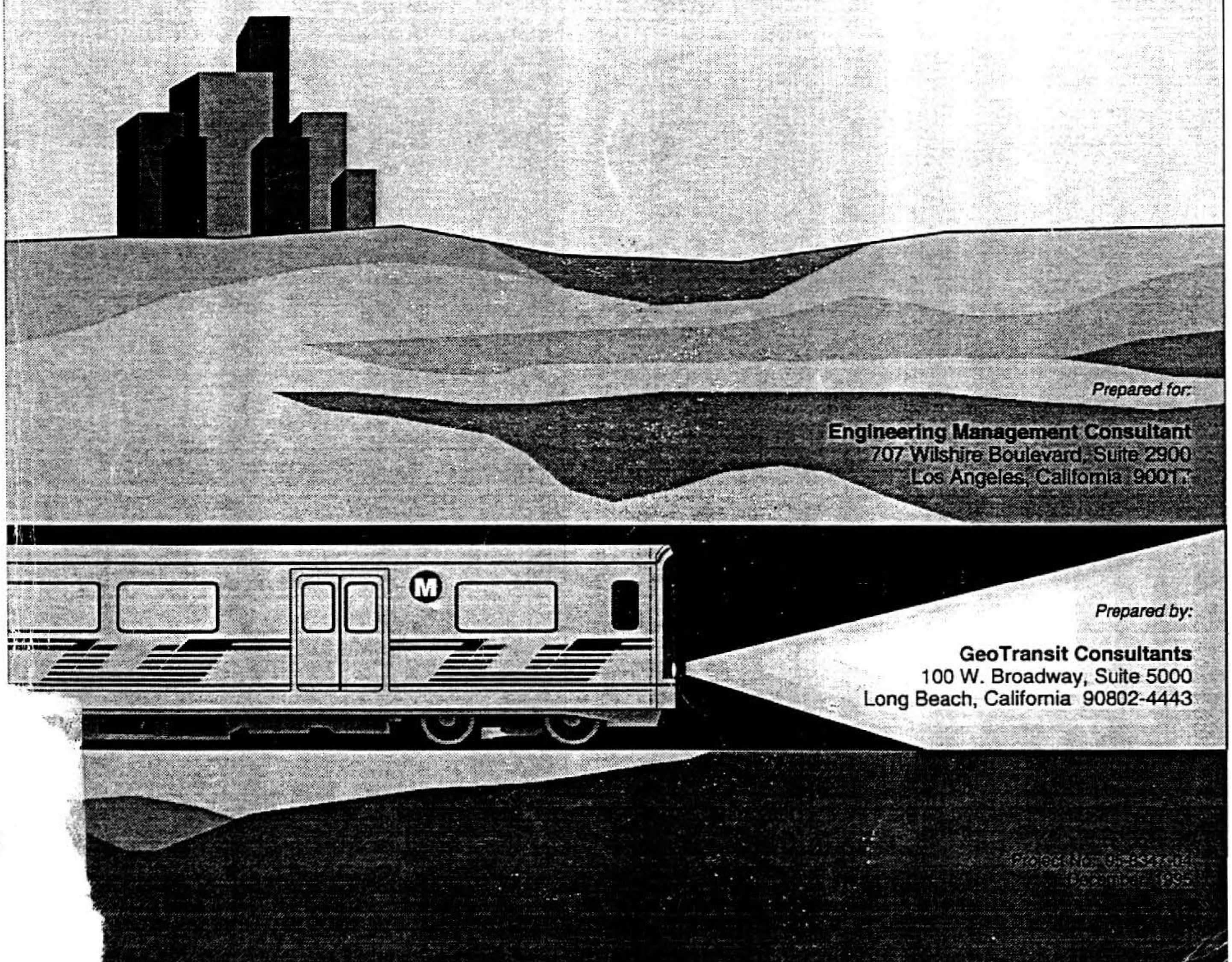
Final Draft
Geotechnical Investigation for:

Union-Little Tokyo Tunnels & Little Tokyo Station Eastside Extension Metro Red Line Project

Volume I of II



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1.0 EXECUTIVE SUMMARY

1.1 GENERAL

This report presents the results of a geotechnical investigation conducted by GeoTransit Consultants for the planned Little Tokyo/Arts District Station (referred to as the "Little Tokyo Station" hereafter in this report), and the tunnels between the Union Station and the Little Tokyo Station. The tunnels and station are part of the proposed Eastside Extension of the Los Angeles Metro Red Line. The primary purposes of this investigation were to evaluate geologic and geotechnical conditions, and to obtain geotechnical data for planning and design of the tunnels and station. In this report, the locations, dimensions and configuration of the proposed station and tunnels were based on available plans and profiles supplied by the Engineering Management Consultant (EMC) at the time of this investigation.

1.2 PROJECT DESCRIPTION

The tunnels consist of two single track, 18-foot inside and 21-foot outside diameter openings in a double line configuration. The tunnels, also referred to as the "Union-Little Tokyo Tunnels" in this report, extend from Union Station along two south branches. One branch (CR track) trends approximately south to Santa Fe Avenue, whereas the other branch (CL track) initially curves southeast then southwest until it nearly merges with the CR track in the vicinity of First Street/South Santa Fe Avenue intersection. The two parallel tunnel tracks then proceed south along South Santa Fe Avenue to the northern terminus of the Little Tokyo Station.

The proposed Little Tokyo Station consists of a cut-and-cover reinforced concrete structure approximately 60 feet in width and 572 feet in length. The station invert is about 68 to 70 feet below ground surface (BGS). The station is located partly within the existing Los Angeles County Metropolitan Transit Authority (MTA) maintenance yard and partly within the South Santa Fe Avenue right-of-way extending from about 100 feet to 672 feet south of the south curb of Third Street.

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 17 rotary wash borings, three Becker hammer borings and one 30-inch bucket auger boring; installing 10 piezometers/monitoring wells; monitoring groundwater levels; sampling groundwater from monitoring wells; conducting an aquifer pumping test and two downhole seismic velocity surveys; performing a geotechnical laboratory testing program on selected soil and bedrock samples and a chemical testing program on selected groundwater and soil samples; conducting an engineering evaluation; and preparing this report.

1.4 GEOLOGIC SETTING

The Eastside Extension alignment is located along the southern flank of the Repetto Hills area of the Los Angeles Basin. In the project area, the tunnels will be driven through alluvial deposits of Holocene and Pleistocene age, and Tertiary-aged bedrock units of the Fernando and Puente Formations. Alluvium consists of mostly coarse granular deposits with local cobbles and boulders and occasional fine-grained interbeds. 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. This 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 escarpment that forms the southern margin of the City Terrace area in the Repetto Hills can be traced intermittently from near the channel of the Los Angeles River to the Monterey Park Hills (Plate 1). Field investigations being performed concurrently to evaluate the escarpment and its impact on the alignment show that alluvial sediments are deformed along its trace. Those investigations also suggest that the Coyote Pass escarpment projects from the heights of East Los Angeles, west across the Los Angeles River floodplain, and intersects the subject tunnels in the vicinity of the First Street bridge. In that area, a shallow bedrock high buried beneath the alluvial deposits occurs.

The results and details of a project-specific fault investigations performed to delineate and characterize the escarpment and to assess its seismic capability are presented in a separate report. Those results indicate that the Coyote Pass escarpment is the result of active deformation, and that its potential for movement should be considered in the design and construction of the proposed tunnels at the projected fault crossing.

1.5 SUBSURFACE STRATIGRAPHY

The planned tunnel and station excavations will be within alluvium and the underlying bedrock units of the Fernando Formation and Puente Formation. The alluvium is heterogeneous and consists of predominantly coarse-grained materials ranging from sands to gravels with local zones of cobbles and boulders (up to 4 feet in size). Occasional interbeds of fine-grained soils consisting predominantly of sandy clay and clayey silt and lean clay are also present.

Bedrock units of the Fernando and Puente formations underlie the alluvium. Within the planned tunnel excavation depth, the bedrock materials, where encountered, are expected to consist predominantly of very low strength (as defined by the Engineering Geology Field Manual, U.S. Department of the Interior, Bureau of Reclamation) siltstone, claystone and occasionally sandstone with local layers of hard, well-cemented calcareous interbeds up to 5 feet thick, and hard concretionary nodules ranging from approximately 2 to 18 inches in size.

1.6 GROUNDWATER LEVEL

The most recently observed groundwater levels are approximately 30 to 45 feet BGS along the portion of the tunnel alignment from Union Station to the vicinity of Banning Street. South of Banning Street the groundwater level dips to the south at an average gradient of about 5 percent to about 79 feet BGS at the northern terminus of the Little Tokyo Station. In the Little Tokyo Station area, the groundwater table is relatively flat with groundwater levels between about 78 and 80 feet BGS.

A significant difference exists between the groundwater level data from this investigation and the data obtained from a 1983 investigation in the project area by others. In the Little Tokyo Station area, and within the southern end of the tunnel segment south of the Banning Street area, the 1983 data suggests groundwater tables up to 55 feet higher than current levels. These groundwater level differences appear to be consistent with the recorded differences in groundwater levels in a number of water wells in the general area, that are monitored by the Los Angeles County Department of Public Works. Thus, a significant fluctuation of groundwater levels can be anticipated during the design life of the station and tunnel facilities. Such potential groundwater fluctuation should be considered in the design and construction. It is important that groundwater levels in the project area be monitored prior to and during construction.

1.7 GROUNDWATER AND SOIL CONTAMINATION

Groundwater within the planned CR and CL tunnel envelopes between Union Station and approximate Station CR 36+50 (in the vicinity of Boring DD-10) was found to be contaminated with hydrocarbons, hydrogen sulfide (H_2S), and a number of constituents (sulfate, sulfide, chloride, etc.) with concentration levels higher than published threshold concentrations (listed in Table D-5 through D-8 in Appendix D). The subsurface materials, especially the fine-grained Fernando/Puente Formation bedrock below the groundwater table, are locally contaminated with hydrocarbons and H_2S in the same area.

In the remaining tunnel and station areas, subsurface soils and potential perched groundwater may locally be contaminated with hydrocarbons due to the proximity of the project area to the existing Union station oil field and other sources, such as nearby oil pipelines and petroleum storage tanks.

1.8 GASSY CONDITIONS

The proximity of the project area to the existing Union Station oil field suggests the likely presence of methane and other oil field related gases in the project area. Available field observations from this and other investigations in the project area, well development data, and available test data on gas samples indicate that H_2S and Methane are likely to be released from or through groundwater, and that there exists a high potential for accumulation of toxic and explosive gases (especially H_2S and methane) in the project area, especially in the area between Union Station and approximate Station CR 36+50 (in the vicinity of Boring DD-10), where groundwater exists within the planned CR and CL tunnel envelopes.

1.9 UNION-LITTLE TOKYO TUNNELS

The Union-Little Tokyo Tunnels will be in predominantly granular alluvium and the Fernando/Puente Formation bedrock which consists predominantly of very soft (low strength) siltstone, claystone and occasionally sandstone with local hard, well-cemented calcareous beds or nodules. Groundwater was encountered within portions of the planned CR and CL tunnel envelopes between Union Station and approximately Station CR 36+50 (in the vicinity of Boring DD-10). The Fernando/Puente Formation bedrock is anticipated to occur periodically in the CR and CL tunnel envelopes between the vicinity of Ducommun Street and the vicinity of the First Street/South Santa Fe Avenue Intersection.

There are several conditions that impact the tunnel design and construction, face stability, excavation techniques, advance rates, and potential ground loss. These conditions include the local presence of cobbles and boulders; 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 granular alluvium; local presence of hard, well-cemented, calcareous interbeds up to 5 feet thick and hard concretionary nodules up to 18 inches in size within the bedrock units; presence of contaminated groundwater and contaminated alluvium/bedrock materials; and the presence of H_2S, methane, and other potentially toxic/explosive gases.

Construction or pre-construction dewatering along the tunnel alignment will result in accumulation of contaminated groundwater. This water will require treatment to reduce contaminant (mainly H_2S and hydrocarbons) concentrations to within limits acceptable for disposal to local storm drains.

The gassy conditions in the tunnel area dictates the need for proper ventilation and monitoring of methane, H_2S and other toxic/explosive gases by the Contractor during tunnel construction to provide a safe working environment in conformance with U.S. and California OSHA requirements.

1.10 LITTLE TOKYO STATION

Station excavation will be primarily in alluvium which is heterogeneous and predominantly granular in nature. it is anticipated that the cut-and-cover station excavation can be achieved by conventional excavation methods. However, suitable excavation equipment to handle the potential local presence of large boulders up to 4 feet in size would likely be required. Most of the alluvium in the station area will run or ravel readily. Thus, timely application of ground support is important to prevent ground loss.

Although requiring monitoring and verification before and during station construction, no preconstruction dewatering is anticipated assuming the groundwater levels during station construction are the same as or similar to the present groundwater levels (about 10 feet below planned station invert).

The planned station excavation will require vertical cuts and shoring due to proximity of the station to existing buildings and limited construction space within the public right-of-way. Based on local practice in similar subsurface conditions, soldier piles and lagging with internal bracing and/or tiebacks are the most likely shoring systems. Design and construction of appropriate excavation support systems to ensure little or no ground loss and to provide a safe work site is the responsibility of the Contractor. Recommended lateral earth pressures for design as well as various design and installation considerations are provided in this report.

Based on the results of the investigation, it is anticipated that the site soils can adequately support the planned main station structure with acceptable total and differential settlements. Various geotechnical design parameters are provided for station design. Where appropriate, ranges of design parameters are provided to account for variability of the subsurface materials.

1.11 SEISMIC DESIGN CRITERIA

No project-specific seismic hazard analyses have been conducted for the Eastside Extension. A seismic hazard study was not part of the scope of this investigation. For geotechnical analyses and design purposes, the results of a 1983 study titled "Seismological Investigation and Design Criteria" prepared by CCI/ESA/GRC for the Metro Rail Project were used in accordance with instructions given by EMC. There have been significant changes in our understanding of the seismicity of the area, the state of the art in seismic hazard analyses, and local code requirements since 1983. Geotechnical recommendations presented herein should be reviewed and revised as appropriate, should seismic criteria be revised in the future.

1.12 LIQUEFACTION POTENTIAL

Liquefaction potential of subsurface soils in the tunnel segment and station area was evaluated. The results indicate that the potential for liquefaction of the soils in the project area under the maximum design earthquake (as defined in the 1983 "Seismological Investigation and Design Criteria" report for the Metro Rail Project) is very low and is not a consideration for design.

2.0 INTRODUCTION

2.1 GENERAL

This report presents the results of a geotechnical investigation for the Little Tokyo/Art District Station (referred to as the "Little Tokyo Station" in this report) and the tunnels between the Union Station and the Little Tokyo Station. The station and tunnels are part of the proposed Eastside Extension of the Los Angeles Metro Red Line. The investigation was performed to support the engineering efforts being undertaken by Engineering Management Consultants (EMC) for the Los Angeles County Metropolitan Transit Authority (MTA).

This geotechnical investigation is part of an overall geotechnical investigation for the design of the first portion of the proposed Metro Red Line Eastside Extension which begins at the southeastern terminus of the Union Station in Los Angeles, and ends at the proposed First/Lorena Station and Tail Track Tunnels in East Los Angeles. This portion of the Eastside Extension is approximately 3.7 miles long and consists of twin tunnels and four cut-and-cover stations, including the tunnels and the Little Tokyo Station addressed in this report. Plate 1 presents the layout plan showing the locations of the tunnel segment and Little Tokyo Station with respect to the Eastside Extension alignment.

2.2 PROJECT DESCRIPTION

Locations, dimensions, and configuration of the proposed station and tunnels presented in this report were based on available plan and profile drawings supplied by EMC at the time of this investigation.

The plan and ground surface and tunnel/station profiles along the tunnel alignment and the Little Tokyo Station are shown in two plans and profiles presented in Section 4.0 (Plates 4 and 5). The following section describes the dimensions and configurations of the tunnel alignment and station. All elevations used in this report are with respect to the City of Los Angeles datum.

2.2.1 Union - Little Tokyo Tunnels

The tunnel segment consists of two single track, 18-foot inside and 20-foot outside diameter openings in a double line configuration. In this report the tunnel segment between the Union Station and the Little Tokyo Station is also referred to as the "Union - Little Tokyo Tunnels". Starting from the southeastern terminus of the Union Station (Station CR 13+00 and Station CL 13+00) the Union - Little Tokyo Tunnels run along two branches. One branch (CR track) trends approximately south to South Santa Fe Avenue, and the other branch (CL track) initially curves southeast, then southwest and then nearly merges with the CR track in the vicinity of the intersection of South Santa Fe Avenue and First Street. The tunnel alignment then proceeds south along South Santa Fe Avenue to the northern terminus of the Little Tokyo Station (approximate Station CR 40 + 84).

The existing ground surface along the CR and CL Tracks dips gently towards the south and varies from approximate Elevation 278 feet at the northern end (approximate Station CR 13 + 30) to approximate Elevation 267 feet at the southern end (approximate Station CR 40 + 84). Current plans indicate that the tunnel invert of the CR and CL tracks varies from about 43 feet to 78 feet below ground surface (BGS). Tunnel gradients are variable and range from 0 to 3 percent.

2.2.2 Little Tokyo Station

The Little Tokyo Station consists of a cut-and-cover reinforced concrete station structure and auxiliary facilities. The station starts at Station CR 40 + 84 (northern terminus) and ends at Station CR 46 + 56 (southern terminus). The station is about 572 feet in length with an inside width of about 60 feet. The Little Tokyo Station is located partly within the existing MTA Maintenance Yard and partly within the South Santa Fe Avenue right-of-way from about 100 feet to 682 feet south of the south curb of Third Street.

The ground surface at the station site varies from approximate elevation 267 feet at the northern terminus to approximate Elevation 265 feet at the southern terminus. The top-of-rail elevation within the station area is approximately 205 feet. The station invert is at approximate Elevation 197 feet. Thus, the depth of excavation for station construction varies from approximately 68 to 70 feet.

2.3 OBJECTIVE AND SCOPE

The objective of the geotechnical investigation was to evaluate subsurface soil and groundwater conditions and to obtain geotechnical data for planning and design of the planned tunnels and station.

The scope of this investigation consisted of the following:

- Review of available literature and reports regarding the geologic, geotechnical, groundwater and seismic conditions in the project area.

- 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

- Performance of a field exploration program, including:

  - Drilling and sampling of 17 rotary wash test borings
  - Drilling and sampling of one 30-inch diameter bucket auger boring to evaluate size and distribution of coarse alluvium
  - Drilling of three Becker hammer borings to evaluate the consistency of coarse alluvium
  - Installing 10 monitoring wells
  - Monitoring groundwater levels at all available monitoring well locations, including those previously installed
  - Obtaining groundwater samples from selected monitoring wells for chemical testing

- Conducting two downhole seismic geophysical surveys to determine shear wave velocity and dynamic modulus characteristics of the subsurface materials

- Performance of a pump test to evaluate the hydraulic conductivity characteristics of coarse alluvium and to estimate water inflows and dewatering needs.

- Performance of a laboratory testing program on selected representative soil and water samples to assess the index and engineering properties of subsurface materials, and to evaluate their chemical characteristics.

- Preparation of this report documenting the results of the geotechnical investigation and providing recommendations for the design.

2.4 PREVIOUS INVESTIGATIONS AND AVAILABLE DATA

A number of project-specific and non project-specific geologic, geotechnical and environmental investigations were previously performed in the project area. Results of these previous investigations conducted by GeoTransit consultants and others were compiled and reviewed to help plan this investigation and to supplement the results of this investigation.

Existing non project-related geotechnical and environmental data are summarized in Tables 2-1 and 2-2, respectively. Locations of borings/wells referred to in these tables are shown in Plates 1 and 2.

Previous project-specific investigations for the Eastside Extension performed by GeoTransit Consultants include the following:

- Preliminary geotechnical investigation (GeoTransit Consultants, 1994a) which includes drilling and sampling of six geotechnical borings (PE-18 and PE-27 through PE-31) and installation of four monitoring wells in four of the boring locations in the project area.

TABLE 2-1. EXISTING SUBSURFACE INFORMATION BASED ON AVAILABLE NON PROJECT-SPECIFIC INVESTIGATIONS IN THE PROJECT AREA

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: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+	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 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'
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

TABLE 2-1. EXISTING SUBSURFACE INFORMATION BASED ON AVAILABLE NON PROJECT-SPECIFIC INVESTIGATIONS IN THE PROJECT AREA

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:3-16	Converse and others, 1984	262	20	-	-	-	1-3	3-20+	-	
CC:3-17	Converse and others, 1984	262	20	-	-	-	0-4	4-20+	-	
CC:3-33	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'
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'
53-2673: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-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

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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-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'
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

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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-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	-	-	
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+	-	
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'

TABLE 2-1. EXISTING SUBSURFACE INFORMATION BASED ON AVAILABLE NON PROJECT-SPECIFIC INVESTIGATIONS IN THE PROJECT AREA

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-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 (?)
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	-	-	-	-	-				

TABLE 2-1. EXISTING SUBSURFACE INFORMATION BASED ON AVAILABLE NON PROJECT-SPECIFIC INVESTIGATIONS IN THE PROJECT AREA

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	
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+	
2776A	Los Angeles/County Department of Public Works	-	225	-	-	-	-	-	-	

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

TABLE 2-2. SUMMARY OF AVAILABLE SOIL, GROUNDWATER AND GAS CONTAMINATION DATA FROM NON PROJECT-SPECIFIC INVESTIGATIONS IN THE PROJECT 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 in a monitoring well during a pump test near west of Union Station. ■ Boring CEG-2 (about 2,000 feet east of Union Station) encountered oil stain in soil samples from Puente Formation, between 38 and 100 feet below ground surface (BGS), and first detected sulfur odor at a depth of about 37 feet. A gas sample from this boring contained 100 ppmv methane and 500 ppmv ethane. ■ Oil stains and sulfur odor were also encountered in soil samples from other borings near Union Station.
Woodward-Clyde Consultant (1986)	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 (between Vignes Street off-ramp from U.S. 101 Freeway and Ramirez Street)	■ Sulfur and hydrocarbon odors and oily, tar-like substances in borings and oil stains in soil samples between 15 and 85 feet BGS. ■ Soil and groundwater samples from the vicinity of Denny's Restaurant were contaminated with petroleum hydrocarbons, VOCs and SVOCs ■ High OVA readings (> 1,000 ppm above background level) were observed in three borings (one near Union Station and two on Center Street between U.S. 101 Freeway and E. Commercial Street) between 6 feet and 87 feet BGS ■ Chemical test results indicate that a number of soil and groundwater samples contained total petroleum hydrocarbons (TPH) and VOCs with concentrations above the corresponding threshold levels (action levels).
Levine-Fricke (1993) RWQCB (1993)	Gateway Center at southwest corner of Macy Street and Vignes Street near Union Station	■ H_2S and following constituents in excess of water quality objectives (set by RWQCB for discharge) in groundwater samples collected prior to dewatering: benzene (120 μg/L), ethylbenzene (1,090 μg/L), 1,1-dichloroethane (30 μg/L), tetrachloroethylene (76 μg/L), toluene (52 μg/L), trichloroethylene (96 μg/L), xylenes (138 μg/L), total dissolved solids (1,550 mg/L), chlorides (162 mg/L), sulfates (474 mg/L) and sulfides (12 mg/L). ■ Ongoing treatment system for groundwater from dewatering (average 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.

- Fault Investigation for "Preliminary Engineering Program", Eastside Extension, Metro Red Line Project" (GeoTransit Consultants, 1994c) which includes drilling and sampling of two geotechnical borings (DD-8 and DD-11) in the project area.

- Stage II Environmental Site Assessment, Eastside Extension, Metro Red Line Project (GeoTransit Consultants, 1994b) with environmental sampling in nine environmental borings (EB-18, EB-20 through EB-25, EB-27 and EB-28) and installation of one nested well (EB-22).

Locations, penetrations and detailed logs of the above geotechnical borings are included in Appendix A. Locations of these previous project-specific geotechnical borings with respect to the station and tunnel alignment are presented in Section 3.0. Based on the results of the preliminary geotechnical investigation, the following were identified as key issues to be addressed in the final geotechnical investigation for the project area:

- A detailed geotechnical investigation program with closely spaced borings would be necessary to fill in the data gaps and to provide sufficient site and structure-specific data for the tunnel and station design. Since a considerable length of the tunnel and the majority of station excavation will be in coarse alluvium containing gravel, cobbles and boulders, the explorations for the final design level should include large diameter bucket-auger borings to better estimate the extent and size distribution of cobbles and boulders. Also, use of Becker hammer drilling would be necessary to estimate the consistency (blow counts) of gravelly soils for an evaluation of their liquefaction potential.

- A "bedrock high" exists in the area of the north-south portion of the alignment between Union Station and the proposed Little Tokyo Station. This bedrock high may represent the location of a fault or "groundwater barrier" that could explain the large differences in groundwater levels observed between the borings in this area. The location lies along the general trend of the Coyote Pass fault, and if a groundwater barrier is present, the fault could be considered active. A detailed hydrogeologic investigation consisting of series of monitoring wells will be required in this area.

- In the project alignment the groundwater levels are above portions of the tunnel invert. The high groundwater levels and the presence of coarse granular materials will require preconstruction dewatering for tunnel construction. Performance of a pumping test and water quality characterization would be necessary to provide the required information to potential contractors bidding on construction.

These issues were considered in developing the scope of this investigation described in Section 2.3. The results of the Stage II environmental site assessment (GeoTransit Consultants, 1994b) are incorporated in this report to characterize subsurface soil, groundwater and gas contamination in the project area.

3.0 FIELD EXPLORATION, FIELD TESTING AND LABORATORY TESTING

3.1 GENERAL

This section provides a description of the subsurface exploration, field testing and laboratory testing performed for this program. The field exploration and field testing program are part of a larger overall geotechnical investigation program being performed for the entire Eastside Extension alignment. Applicable results from the overall geotechnical investigation program and previous investigations in the project area (Section 2.4) were also used in developing findings and conclusions presented in this report.

3.2 GEOTECHNICAL BORINGS

Seventeen wash borings, one bucket auger boring, and three Becker hammer borings were drilled in the project area. Fifteen of the borings (identified by the prefix DD-) were drilled within the tunnel segment. Six of the borings (identified by the prefix SD-) were drilled within the Little Tokyo Station area. Locations and penetration depths of these borings are summarized in Table 3-1 and Plate 3. For completeness, geotechnical borings completed during our preliminary investigations in this area (GeoTransit Consultants, 1994a, 1994b) have also been included in Table 3-1.

The borings were logged in the field by a geologist or engineer under the direct supervision of a Registered Geotechnical Engineer (RGE) or a Certified Engineering Geologist (CEG). The materials were classified in general accordance with American Society of Testing and materials (ASTM) Standards and the Unified Soil Classification Systems. The field logs were refined and reclassified, if appropriate, after further laboratory examination and testing of selected soil samples. Boring logs are presented in Appendix A.

3.2.1 Rotary Wash Borings

Rotary wash borings for the geotechnical subsurface exploration program were drilled using Mayhew 1,000 and Midway 13 mud rotary drill rigs with 4-7/8-inch diameter tricone drill bits producing nominal 5- to 6-inch diameter boreholes. Boring DD-3C was drilled to a diameter of 10 inches to accommodate the 6 inch well casing for the aquifer pump test. Borings were generally drilled to depths of about 20

TABLE 3-1 SUMMARY OF FIELD EXPLORATION PROGRAM

Boring #*	Type of Boring	Approximate Station	Approximate Offset From Centerline of CR (ft)	Approximate Offset From Centerline of CL (ft)	Location	Approximate Ground Surface Elevation (ft)	Approximate Tunnel Invert/(Station Bottom) Depth/ Elevation (ft)	Total Penetration Depth (ft)	Piezometer Installation (Groundwater depth in feet. BGS measured September 1995)
DD-1	Rotary-Wash	CL 21+70	-	0	Commercial	273.5	62.5/ 211	88	Yes (31.12)
DD-2 (1)	Becker Hammer	CL 25+50	-	0	Du commun	272.5	73.5/ 199	62	Yes (31.64)
DD-2S	Rotary -Wash	CL 25+51	-	8 Left	Du Commun	272	74/198	100	No
DD-3	Rotary-Wash	CL 28+68	-	10 Right	Jackson	270	71/199	73	Yes (35.56)
DD-3C	Rotary-Wash	CL 28+43	-	18 Left	Jackson	270	71/199	73.5	Yes (35.68)
DD-3D (2)	Rotary-Wash	CL 28+80	-	50 Left	Jackson	270	71/199	36.5	No
DD-4	Rotary-Wash	CL 35+60	-	0	Banning/Center	270	69/201	98	Yes (42.48)
DD-4-1	Rotary-Wash	CL 31+95	-	20 Right	Center	272	75/197	98	No
DD-5 (3)	Bucket Auger	CL 43+70	-	0	Red Line Yard	267	65/202	32.5	No
DD-6 (4)	Becker Hammer	CR 25+60	0	-	Jackson	272	*1 73/199	69.0 *2	No
DD-7	Rotary-Wash	CR 32+05	5 Left	-	Santa Fe/Banning	269	71/198	93.0	Yes(41.30)
DD-8	Hollow-S &Rotary W	CR 33+25	0	-	Santa Fe	268	69/199	96.2	No
DD-9	Rotary-Wash	CR 34+75	0	-	Santa Fe/First	266	65/201	93.0	Yes (54.68)
DD-10	Rotary-Wash	CR 36+35	4 Right	-	Santa Fe	266	64/202	92.5	Yes (74.92)
DD-11	Hollow-Stem	CR 38+00	0	-	Sante Fe	268	64/204	102	No
DD-12	Rotary-Wash	CR 39+50	0	-	Santa Fe	267	63/204	86.0	Yes (dry)
DD-65	Rotary-Wash	CL 19+10	-	10 Right	Center	275	57/218	75	No
SD-1	Rotary-Wash	CL 40+60	-	0	Red Line Yard	267	65/202	95.5	No
SD-2 (5)	Becker Hammer	CR 42+45	0	-	Santa Fe	266	(63/203)	34.5	No
SD-3 (6)	Rotary Wash	CL 43+95	-	0	Red Line Yard	265	(63/202)	35.5	No
SD-3A	Rotary Wash	CL 43+55	-	0	Red Line Yard	265	(63/202)	126.0	No
SD-4	Rotary Wash	CR 45+55	0	-	Red Line Yard	265	(63/202)	95.5	No
SD-5	Rotary Wash	CL 46+80	-	7 Left	Little Tokyo Station	264	(63/201)	100.5	Yes (78.30)
PE-18	Rotary Wash	CR 42+90	50 Right	-	Third/Sante Fe	265	62/203	86	Yes(78.24)
PE-27	Rotary Wash	CR 31+96	40 Right	-	Banning/Santa Fe	270	72/198	81.5	No
PE-28	Rotary Wash	CR 28+90	130 Right	-	Vignes/Temple	270	75/195	80.9	No
PE-29	Rotary Wash	CR 22+50	20 Right	-	Vignes/Temple	271	61/210	82	Yes(32.06)
PE-30	Rotary Wash	CR 19+26	60 Right	-	Vignes/Commercial	275	56/219	80.8	Yes(32.86)
PE-31	Rotary Wash	CL 28+50	-	25 Left	Jackson cul-de-sac	270	75/195	83	Yes(32.06)

NOTES: (1) DD-2 terminated prior to reaching planned penetration depth (80') due to encountering free hydrocarbon product in the groundwater.

(2) DD-3D terminated prior to reaching planned penetration depth (90') due to caving conditions/loss of circulation of drill mud.

(3) DD-5 terminated prior to reaching planned penetration depth (75') due to encountering caving conditions in bucket auger boring.

(4) DD-6 terminated prior to reaching planned penetration depth (80') due to plugging of Becker hammer casing in the Fernando/Puente Formation bedrock.

(5) SD-2 terminated prior to reaching planned penetration depth (85') due to encountering free hydrocarbon product between 26 feet and 34.5 feet BGS.

(6) SD-3 terminated prior to reaching planned penetration depth (125 feet) due to excessive loss of fluid.

* Borings along tunnel alignment are identified by the prefix DD-;borings at station locations are identified by the prefix SD-; borings drilled during the preliminary investigation are identified by the prefix PE-, DD-8 and DD-11. ?

*2 11' [illegible]

*1 20' [illegible] (DD-6 WAS 26 [illegible])

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feet or more below the tunnel inverts, and about 30 and 60 feet below the station inverts as determined from the plan and profile drawings for the Eastside Extension alignment provided by EMC in August, 1995. Two rotary borings (DD-3D and SD-3) terminated prior to reaching the planned depths due to severe caving/loss of mud circulation conditions. Soil samples of the encountered alluvial soils were obtained at approximately 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.

In Borings DD-4, DD-7, and DD-9, relatively thick layers of Fernando/Puente Formation bedrock were encountered. Within these borings, semi-continuous, relatively undisturbed samples of the Puente/Fernando Formation were obtained using a Pitcher-barrel sampler and a drive sampler. The Split spoon sampler was also occasionally used to assess the penetration resistance of the Fernando/Puente Formation bedrock.

3.2.2 Bucket Auger Boring

A 30-inch diameter bucket auger boring (DD-5 in Table 3-1) was drilled to estimate the extent and size distribution of cobbles and boulders in the project area. Bulk samples were obtained at approximate 5-foot depth intervals and at stratum changes. The boring was terminated and backfilled after a depth of 32.5 feet below the ground surface was reached (as compared with a planned penetration depth of 85 feet) due to persisting caving conditions. Within the penetration depth of the boring, cobbles up to 12 inches in size were encountered. No boulders were encountered within the penetration depth of this boring.

3.2.3 Becker Hammer Borings

Three Becker hammer borings (DD-2, DD-6 and SD-2) were drilled using a Link Belt 180 Becker drill rig with an AR-1000 diesel hammer to evaluate the penetration resistance of the coarse grained gravelly and cobbly alluvial soils in the project area for equivalent N-Value and liquefaction potential assessment purposes. Penetration resistances (Becker blow counts) and bounce chamber pressures were measured, corrected and correlated to standard penetration test (SPT) blow counts in accordance with the procedures recommended by Harder and Seed (1986). Also, in conformance with the procedures recommended by Harder and Seed, all borings were advanced using a 6.6-inch outer diameter (O.D). closed end drill bit.

Drilling was advanced by switching to a 6.6-inch O.D. open bit when very hard drilling conditions (450 to about 600 blows per foot or more) by the closed bit were encountered.

Borings DD-2 and SD-2 were terminated at 62 feet and 34.5 feet respectively due to encountering hydrocarbon contamination. After encountering hard driving conditions at 56 feet BGS, Boring DD-6 was advanced further using a 6.6-inch O.D. open-bit and subsequently terminated at 69 feet BGS due to plugging of the open bit which prevented further advance.

Becker blowcounts at the three boring locations were correlated to equivalent corrected SPT blowcounts. The results are shown in Figures 3-1a and 3-1b.

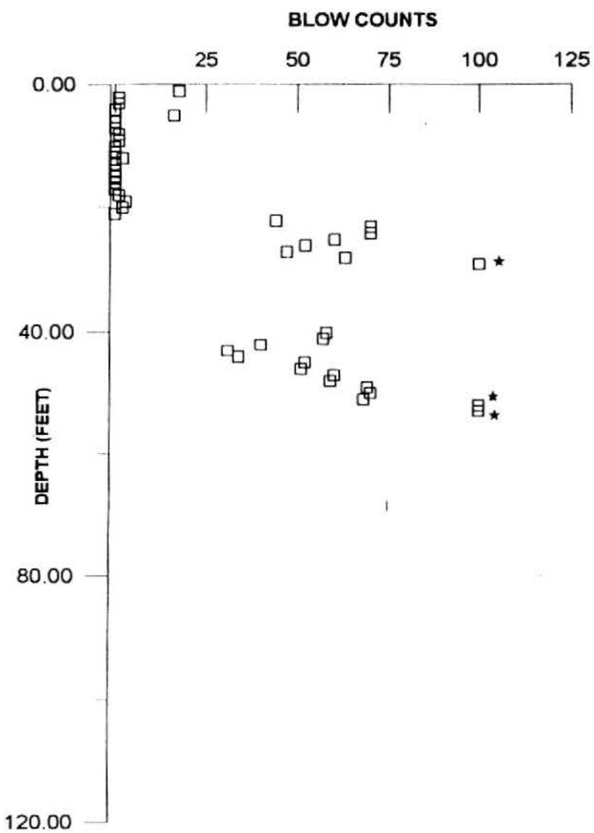
3.2.4 Monitoring Well Installation and Groundwater Level Monitoring

Ten monitoring wells were installed in Borings DD-1, DD-2, DD-3, DD-3C, DD-4, DD-7, DD-9, DD-10, DD-12 and SD-5. Well installation diagrams are presented in Appendix A. Groundwater levels of these piezometers and other existing piezometers (PE-18, PE-29, PE-30, and PE-31) installed during the preliminary engineering program (GeoTransit Consultants, 1994a) were periodically monitored using an electronic water-level indicator. Table 3-2 presents a summary of these readings.

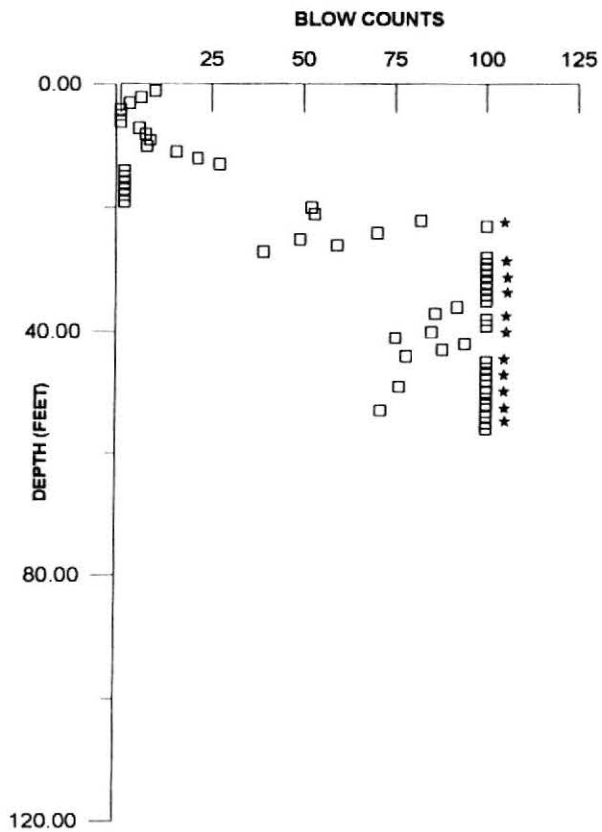
3.3 FIELD TESTS

3.3.1 Aquifer Pump Testing

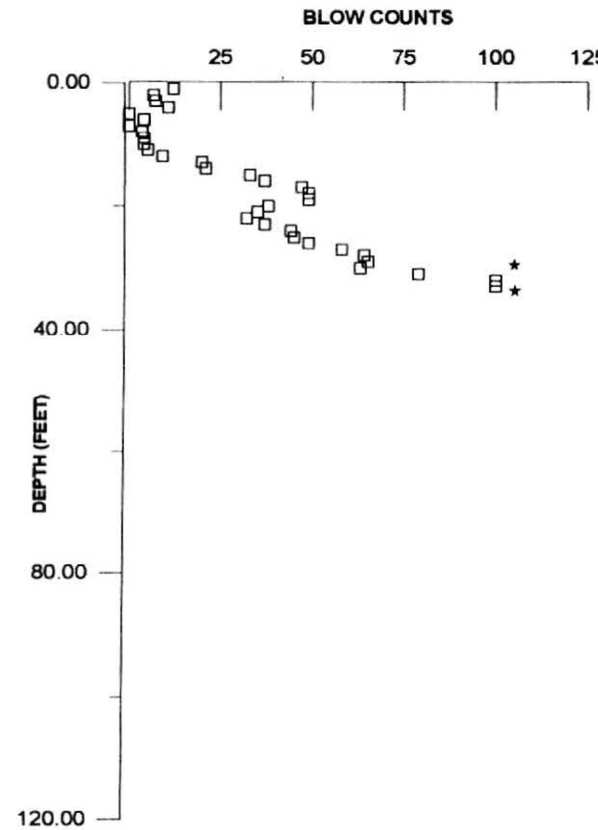
An aquifer pump test was performed at Boring/Pump Well DD-3C and monitored at two nearby observation wells (DD-3 and PE-31 located approximately 39 and 13 feet, respectively from DD-3C). The pump test was conducted at the east end of Jackson Street (about 295 feet east of the center line of Center Street). The aquifer pump test was performed to evaluate the hydraulic characteristics of the coarse alluvium. The aquifer pump test included a step drawdown test at pump rates of 10, 25, 50, 74, and 77 gallons per minute (gpm), a constant discharge rate test at a flow rate of 60 gpm for about 22 hours, and water level recovery monitoring for about 25 hours. At the test location, the groundwater level at the start



A. Boring DD-2



B. Boring DD-6



C. Boring SD-2

Note:

Equivalent SPT (N) blow counts were correlated from Becker hammer blow counts based on Harder and Seed (1986).

Explanation:

□ SPT (N)

□* SPT (N) > 100

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SPT (N) Based on Becker Hammer Blow Counts

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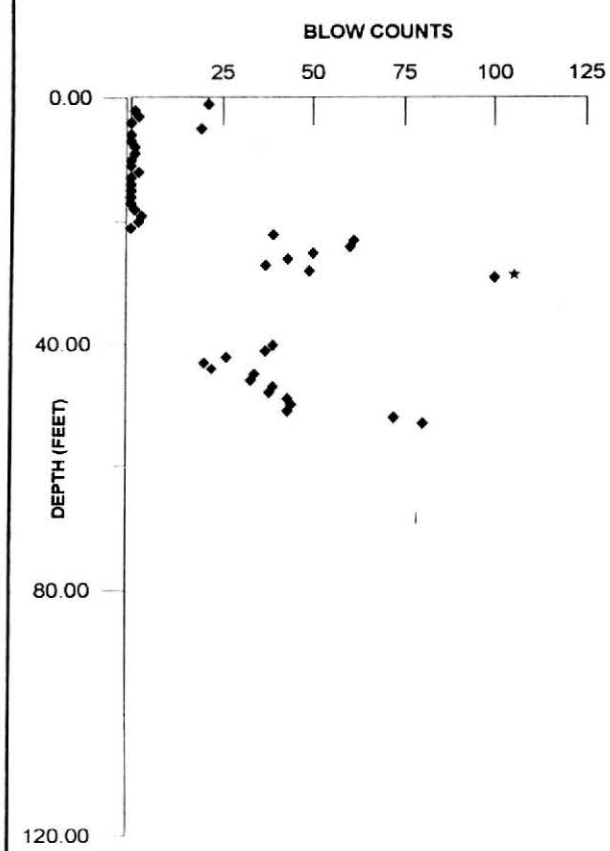
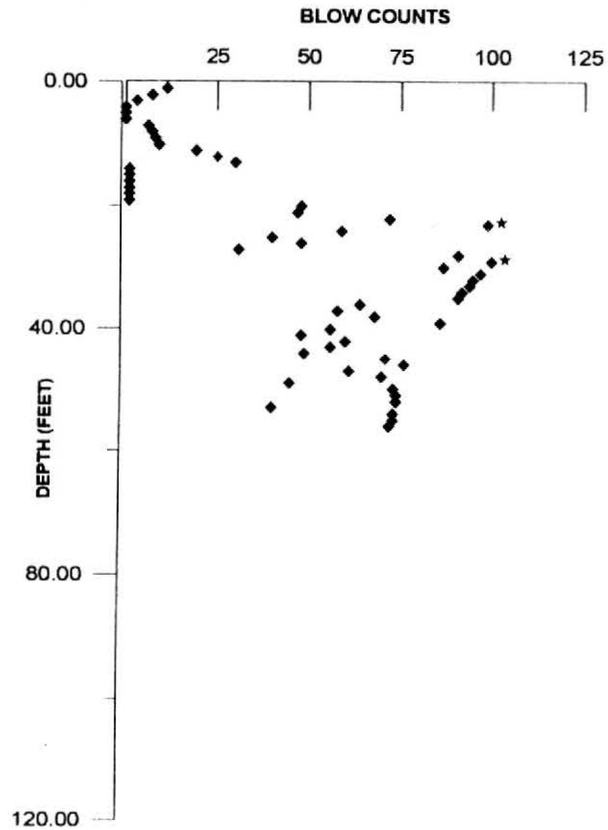
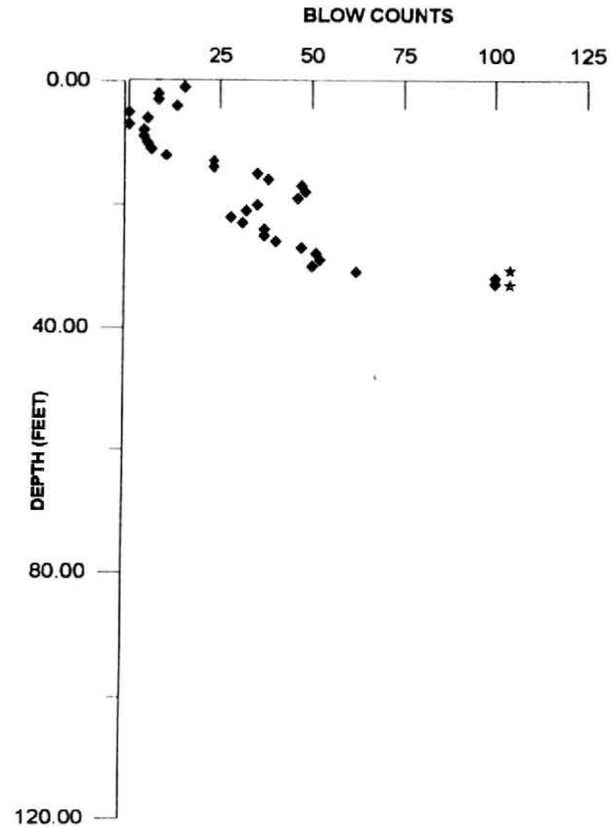
Figure 3-1a



A. Boring DD-2



B. Boring DD-6



C. Boring SD-2

Note:

Equivalent (N1)60 blow counts were corrected from equivalent SPT (N) values for overburden and rod length (Seed et al., 1985).

Explanation:

◆ Normalized SPT (N1)60

◆* Normalized SPT (N1)60 > 100

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Normalized SPT (N1)60
Based on Becker Hammer
Blow Counts

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Figure 3-1b

3-6

TABLE 3-2 SUMMARY OF PROJECT-SPECIFIC GROUNDWATER LEVEL DATA

BORING #	APPROXIMATE STATION ALONG CENTER LINE OF CR TRACK	APPROXIMATE STATION ALONG CENTER LINE OF CL TRACK	APPOXIMATE GROUND SURFACE ELEVATION (FT)	APPROXIMATE DEPTH OF TUNNEL/ STATION INVERT (FT)	GROUNDWATER LEVEL DATA											
					DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)	DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)	DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)	DATE	APPROXIMATE DEPTH (FT)	APPROXIMATE ELEVATION (FT)
DD-1	-	21+70	273.5	62.5	9-22-95	31.12	244.88	7-27-95	33.02	242.98						
DD-2	-	25+50	272.5	73.5	9-22-95	31.64	240.36	7-27-95	32.80	239.20						
DD-3	-	28+68	270	71	9-22-95	35.56	234.44	7-27-95	36.45	233.55						
DD-3C	-	28+43	270	71	9-22-95	35.68	234.32	7-27-95	36.40	233.60						
DD-4	-	35+60	270	69	9-22-95	42.48	226.52	7-27-95	42.61	226.39						
DD-7	32+05	-	269	71	9-22-95	41.30	228.70	7-27-95	41.46	228.54						
DD-9	34+75	-	266	65	9-22-95	54.68	211.32	7-27-95	54.83	211.17						
DD-10	36+35	-	266	64	9-22-95	74.92	193.08	7-27-95	75.50	192.50						
DD-12	39+50	-	267	63	9-22-95	DRY	UNKNOWN	7-27-95	DRY	UNKNOWN						
SD-5	-	46+80	264	63	9-22-95	78.30	185.70	7-27-95	78.55	185.45						
PE-18	42+90	-	265	62	9-22-95	78.24	186.76	7-27-95	78.60	186.40	1-11-94	79.2	185.8	12/9	78.7	186.3
PE-29	22+50	-	271	61	9-22-95	32.06	243.94	7-27-95	33.13	242.87	1-11-94	34.9	235.1	12/9	33.8	236.2
PE-30	19+26	-	275	56	9-22-95	32.86	242.14	7-27-95	34.43	250.57	1-11-94	36.5	238.5	12/9	36.0	239.0
PE-31	-	28+50	270	75	9-22-95	36.02	233.98	7-27-95	36.66	233.34	1-11-94	38.3	231.7	12/9	37.5	232.5
EB-22	22+60	-	276	68	1-22-94	34.30	235.10									

[illegible] (WATER LEVEL APPEARS TO BE 16' BELOW STA. INVERT)

3-7

of the test was about 36 feet BGS. As shown in the logs of Borings DD-3C, DD-3 and PE-31 (Appendix A), the top of the Fernando/Puente Formation bedrock in the vicinity of the pump well is about 73 feet BGS. Thus, the aquifer thickness is about 37-feet. This aquifer consists of about 17½ feet of gravel with sand, and sand with silt and gravel overlying about 19½ feet of less permeable silty sand (sand with more than 12% fines) and sand with silt (sand with 5% to 12% fines).

Details and results of the aquifer pump testing are presented in a separate report titled "Results of Aquifer Pump Testing at Jackson Street, Eastside Extension, Metro Red Line, Los Angeles, California," by GeoTransit Consultants (1995). In summary, the results of the aquifer pump testing indicate the following:

- The aquifer is unconfined to semi-confined with an estimated transmissivity of about 9,000 to 10,000 gallons per day per foot (gpd/ft) and a storage coefficient of about 0.005 to 0.01.

- Considering an aquifer thickness of 37 feet, the estimated transmissivity corresponds to an average (averaged over 37 feet) hydraulic conductivity (coefficient of permeability) of about 1.2×10^{-2} cm/sec.

- Most of the water derived from pump well DD-3C likely came from the more permeable 17.5-foot thick layer of gravel with sand and sand with silt and gravel. Assuming that all of the water from Pump Well DD-3C was drawn from this more permeable layer, the average (averaged over 17.5 feet) hydraulic conductivity of this layer was calculated to be about 2.5×10^{-2} cm/sec.

3.3.2 Downhole Seismic Velocity Survey

Two downhole seismic velocity surveys were performed in PVC-cased Borings DD-4 (located on the tunnel alignment) and SD-5 (just south of Little Tokyo Station). Procedures used for seismic velocity measurements were in general accordance with those described by U.S. Army Corps of Engineers (1979) and Mooney (1984). In general, downhole compressional (P) and shear (S) wave velocities were measured at approximately 5-foot intervals, using three-mutually perpendicular geophones (one vertical and two horizontal) mounted in a 1.75 inch steel cylinder in the borehole. P and S wave sources were generated by hitting a vertical hammer against a metal plate and a horizontal hammer on a wooden beam, respectively. Both metal plate and wooden beam were located on the ground surface about 10 to 12 feet away from the borehole. The results of the seismic velocity surveys are summarized in Table 3-3.

3.4 LABORATORY TESTING PROGRAM

3.4.1 Geotechnical Laboratory Testing

All drive, split spoon, Pitcher barrel, and bulk samples obtained during the subsurface exploration were brought to EARTH TECH's soil mechanics laboratory where they were visually examined to verify field classifications. 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 in situ moisture and density, gradation, shear strength, uniaxial compressive strength, permeability and consolidation characteristics, slake durability, 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 3-4. Laboratory test results are summarized in Table 3-5 and included in Appendix B. In situ 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 4.4.

3.4.2 Analytical (Chemical) Testing

3.4.2.1 Groundwater

An analytical (chemical) test program was performed on the following five samples of groundwater:

- Three groundwater samples from Pump Well DD-3C, including one obtained after the well was developed, one at about the mid-point of the aquifer pump test and one after completion of the aquifer pump testing.

TABLE 3-3 RESULTS OF DOWNHOLE SEISMIC VELOCITY SURVEYS AT TWO BORING LOCATIONS

Boring No.	Depth Range (ft)	Measured Compressional Wave Velocity (ft/sec)	Measured Shear Wave Velocity (ft/sec)	Calculated Dynamic Modulus and Poisson's Ratio			
				Shear Modulus 10^3ksf	Young's Modulus 10^3ksf	Bulk Modulus 10^3ksf	Poisson's Ratio
DD-4	0-10	1500	730	2.15	5.8	6.2	0.35
	10-30	1700	570	1.31	3.77	9.93	0.44
	30-60	4600	1950	15.4	42.8	65.1	0.39
	60-95	4600	1480	8.86	25.5	73.8	0.44
SD-5	0 - 10	990	490	0.97	2.6	2.67	0.34
	10 - 40	2350	930	3.5	9.84	17.7	0.41
	40 - 80	3150	1250	6.32	17.8	31.7	0.41
	80 - 95	5800	1250	6.32	18.6	127.0	0.48

TABLE 3-4 GEOTECHNICAL LABORATORY TEST PROGRAM

TEST TYPE	NUMBER OF TESTS	TEST PROCEDURE
Visual Soil Classification	Every Sample	ASTM D2487 / D2488
Moisture Content	158	ASTM D 2216
Dry Density	134	ASTM D 2937
Grain Size Distribution	75	ASTM D 422
Percent Passing #200 Sieve	37	ASTM D 1140
Atterberg Limits	21	ASTM D 4318
Specific Gravity	3	ASTM D 854
Direct Shear (3 Points)	10	ASTM D 3080
Unconfined Compression	12	ASTM D 2166
Triaxial Compression	9	ASTM D 4767
One Dimensional Consolidation	7	ASTM D 2435
Triaxial Permeability	3	ASTM D 5084
Slake Durability Test	2	ASTM D 4464
pH	14	EPA Method 9045
Chloride Content	17	CALTRANS Test 422 and EPA 300
Sulphate Content	17	CALTRANS Test 417-B and EPA 300
Electrical Resistivity	14	CALTRANS Test 532

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 • ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-E (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ Insitu(saturate) Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4464 (%)
DD-1	S-1	5	SM	Af	8																							
	D-1	10	SP-SM	Qya	(22)	11.7	107.1																					
	S-2	15	SP-SM		21																							
	D-2	20	CL/SM		(30)	32.5	91.9																					
	S-3	25	SP-SM		>100																							
	D-3	30	SP-SM •		>(100)	12.5	124.0	30:63:7																				
	S-4	35	SP-SM		46	14.1				9	2.63																	
	D-4	40	SW-SM •		(56)	10.0	124.8	29:64:7																7.40	52	95	2168	
	S-5	45	SP-SM		84					10																		
	D-5	52	(SP-SM •)		(61)	13.1	123.3	42:52:6																				
	S-6	55	(SP-SM)		91					9																		
	D-6	60	(SP-SM •)		(73)	10.5	128.4	44:50:6																				
	S-7	65	ML		42				32,25,7																			
	D-7	70	SM		(40)	24.9	98.9	0:62:38					800	35														
	S-8	72	SM		>100					33																		
	D-8	75	SP-SM •		(43)	20.8	105.4	0:90:10																				
	S-9	80	SW		>100																							
	D-9A	85	ML	Tf/Tp	(33)	32.3	88.6																					
DD-2S	D-1	5	GP	Qoa	(4)																							
	S-2	10	SP		6																							
	D-3	15	(SP)		(37)	5.9	119.8																					
	D-4	20	SP		>100																							
	D-5	25	SP		(100)	12.2	120.3																					
	S-6	30	SW-SM		98			26:62:12																				
	D-8	38	GP		(100)																							
	S-10	40.5	(SM)		>100			29:52:19																				
	D-12	43.5	SP-SM		(92)																							
	S-14	46	SW-SM		>100			12:76:12																				
	D-15	49	SP		(92)	17.8	112.3																					
	D-18	55	(SP)		>(100)	15.4	117.5																					
	S-20	58	SM		>100			5:77:18																				
	D-21	61	CL		>(100)	25.9	99.5		37,20,17															8.25	170	400	1165	
	S-23	64	CL		>100																							
	D-24	67	SM		>(100)	25.1	101.0																					
	S-26	72	SW-SM		>100			27:64:9																				
	D-27	75.2	MH	Tf/Tp	>(100)	29.5	91.4		50,35,15																			
	D-30	81	MH		>(100)	28.9	91.0																					
	P-32	84	MH						59,31,28			8.781																
	D-33	87	MH		>(100)	30.9	90.2																					
	S-37	95.5	MH		>100																							
DD-3	D-1	5	ML	Af	(7)																							
	D-2	10	SW/GP	Qya	(18)	9.7	115.0																					
	D-3	15	GP		(23)																							
	D-4	20	SW-SM		(31)	6.1	128.5																					
	S-6	25	GP-GM		49																							
	D-6	30	SP		(64)	14.4	104.1																					
	D-7	35	SW-SM •		(90)	7.1	130.5	29:63:8																				
	D-7A	40	GP		>(100)																							
	D-7B	45	GP		(62)																							
	D-8	50	SP-SM		(68)	14.0	120.0	6:82:12																				
	D-9	55	SP-SM		(46)					6																		
	D-10	60	ML •		(26)	22.3	105.4	0:21:79	30,25,5				600	36														
	D-11	65	SW-SM •		>(100)	14.0		8:85:7																7.27	20[illegible]	116	5263	
DD-3C	S-1	5	SP-SM	Qya	20																							

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 • ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ insitu(saturate) Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4464 (%)
DD-3C	S-2	7	SP-SM	Qya	14																							
	D-2	10	(SP)		(15)	12.5	125.0																					
	S-3	15	SP		46																							
	D-4	20	GP		(35)	8.1	123.3																					
	S-4	25	SM/GM		88																							
	D-6	30	SP-SM		(32)	13.3	104.6																					
	S-7	35	SM/GM		>100																							
	D-8	40	GP •		>(100)	12.8		72:21:4																				
	S-6	45	GP		>100																							
	S-10	50	(SW-SM •)		>100			34:56:10																				
	D-11	54	SM		(29)	20.4	107.8			18												40	6.0E-07					
	S-12	60	ML		60			0:21:79																				
	D-13	65	SM		(54)	22.6	104.0			44												50	1.9E-06					
	S-14	69	SW-SM •		>100			11:80:9																				
	D-15	75	ML	Tf/Tp	(36)	23.3	100.0															60	1.2E-08					
DD-3D	S-1	5	SM	Qya	33																							
	D-2	10	SP		(14)																							
	S-2	15	SP/GM		54																							
	D-4	20	GP		(39)	10.0	124.8																					
	S-5	25	GP		53																							
	D-6	30	GP		(20)	11.2	97.7																					
	S-7	35	GW-GM •		96			57:35:8																				
DD-4	S-1	6	ML	Af	9																							
	D-2	10	SM	Qya	(23)	10.8	122.3																					
	S-3	15	SM/GM		99																							
	D-4	20	GP-GM		(37)	12.8	120.4																					
	S-5	25	GP-GM		45																							
	D-6	30	(SW-SM)		>(100)	9.6	130.6	46:46:8																				
	S-7	35	SP-SM		>100					11																		
	D-8	40	GP-GM •		(93)	9.0	127.3	68:26:6																				
	S-9	45	GP-GM		100						2.61																	
	D-10	50	(SW-SM •)		(73)	13.2	121.7	17:73:10																				
	S-11	52.5	SM		>100	12.1																		5.52	215	68	2143	
	D-12A	55	SM		>(100)	10.5		31:48:21																				
	D-13	57	ML	Tf/Tp	(22)																							
	D-13A	62	ML		(26)																							
	S-14	63	ML		73																							
	D-15	66.5	ML		(39)																							
	T-16	67.5	ML		-	27.4	95.4					14.009																
	D-17	71	ML		(28)																							
	D-19	74.5	ML		(25)																							
	T-20	75.5	ML			25.2	100.0		47,34,13								0.0920	0.0220	8.5E-04	1	+0.82							17.5
	T-21	78.5	ML			18.7	107.7		37,31,6			9.434					0.063	0.012	-	10	0			6.45	86	169	1256	
	T-24	87.5	CL			26.7	95.8		41,22,19			13.147			5800	25												
DD-4-1	D-2	10	SP	Qya	(45)	8.2	125.5																					
	S-3	15	SP-SM		>100																							
	D-4	20	SP-SM		(29)	16.0	111.4																					
	S-5	25	SW-SM		79			13:79:8																				
	D-6	30	SP-SM		(43)	15.7	115.4																					
	S-7	35	(GP-GM)		69			51:41:8																				
	D-8	40	GP		>(100)																							
	S-9	43	GP		>100																							
	D-10	46	(SP-SM)		(84)	12.3	116.6	37:57:6																				
	S-11	49	(SP-SM)		>100			40:52:8																				

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 + ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ Insitu/(saturate) Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4464 (%)
DD-4-1	D-12	52	SP-SM	Qya	(34)	19.9	107.0			7																		
	S-13	55	CL/ML	Tf/Tp	37																							
	P-14	58	CL/ML			25.1	97.8																		(940)	(180)		
	D-15	61	CL/ML		(25)																							
	P-16	64	ML			23.9			39,36,13			3.035												7.63	92	890	1601	
	D-17	68	CL/ML		(26)	23.7	102.3																					
	P-18	70	CL/ML																						(690)	(150)		
	D-19	73.3	CL/ML		(32)	23.7	101.4																					
	P-20	76	ML						46,28,18			3.045																
	S-21	79	CL/ML		53																							
	P-22	82	CL/ML																						(800)	(210)		
	D-23	84.5	CL/ML		(27)	24.1	100.8																					
	P-24	88	CL/ML																									
	D-25	91	CL/ML		(25)	25.6	98.3																					
	P-26	94	CL/ML																									
	D-27	97	CL/ML		(28)	24.6	100.0																					
DD-5	2	10	SP-SM •	Qya				16:79:5																				
	3	15	SW-SM •					28:66:6																				
	5	25	SW-SM •			1.9		17:78:5																7.18	107	72	103,550	
	6	30	SW-SM •					19:73:8																				
DD-7	S-1	5	ML	Af	5																							
	D-2	10	(SP-SM)	Qya	(9)	21.5	100.1																					
	S-3	15	SW		33																							
	D-4	20	(SW-SM)		(26)	13.3	109.9																					
	S-5	25	SP		72																							
	D-6	30	(SP-SM)		(69)	10.7	129.1																					
	S-7	35	(SP-SM)		75																							
	D-8	40	(SP-SM •)		>(100)	14.2		19:74:7																				
	S-9	45	GP-GM		81	6.0																		3.60	188	1085	2887	
	D-10	50	GM		>(100)	10.8	126.4	48:37:15																				
	S-11	55	ML	Tf/Tp	49																							
	T-14	63	ML			23.9	102.6					12.990																
	T-17	72	ML			25.3	101.2					10.336																
	T-18	75	CH			24.9	96.1		55,28,27								0.066	0.019	-	10	+0.01							
	T-19	78.5	ML			28.2	93.3		47,30,17																			
	T-20	81	CH			28.1	97.8		55,24,31			18.095			4600	25												
DD-9	S-1	5	SP-SM	Qya	14																							
	D-2	10	SW-SM		(22)	7.4	123.7																					
	S-3	15	GP-GM		30																							
	D-4	20	GP-GM		(37)	14.2	108.4																					
	S-5	25	GP-GM		>100																							
	D-6	30	SP		(38)	12.2	115.0																					
	S-7	35	SP-SM •		>100			35:53:12																				
	D-7	40	GW-GM		>100																							
	D-8	45	(SC •)		(61)	17.3	107.4	14:63:23	31,18,13																			
	D-9	48	(SM)		(92)	18.4																						
	S-10	50	GM/SM		>100																							
	D-11	55	(SP-SM)		(92)	11.4	119.8			7														7.61	186	81	5273	
	S-12	59	GM/SM		41																							
	D-13	61	ML	Tf/Tp	(26)	29.0	94.4																					
	S-14	62	ML		41																							
	T-15	64	ML			25.1	101.2					9.444																
	D-16	66.5	ML		(26)	24.5	100.5		38,27,11				800	34														
	S-17	67.5	ML		43						2.72																	

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 - ASTM D2486	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ insitu/saturate; Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4464 (%)
DD-9	T-19	72	CH	Tf/Tp		28.4	97.0		51,23,28			13.16			4000	26												
	T-23	84	ML			26.2	97.0					20.316																
DD-10	S-1	5	SM	Af	11																							
	D-2	7.5	SP	Qya	(9)																							
	D-3	10	SM		(9)	25.6	92.5																					
	S-4	15	SP		84																							
	D-5	20	SP		(24)	10.7	125.3																					
	S-6	25	SP-SM		45																							
	D-7	29	SC		(9)	14.1	107.2																					
	S-8	30	(SP-SM •)		100			43:47:10																				
	D-9	35	SW-SM •		(79)	8.9	129.2	39:54:7																				
	S-10	40	SP-SM		86					8																		
	D-11	45	GW •		(48)	7.6	114.6	73:24:3																				
	S-12	50	GW		116																							
	D-13	52	GW		(9)																							
	S-14	60	SM		>100					14																		
	D-15	65	SC		(52)	13.2	114.6		50,18,32																			
	S-16	70	SM		>100	15.0		7:78:15																8.11	208	82	4810	
	D-17	75	SP-SM		(56)	9.8	121.9			9			450	43														
	S-18	80	SM		>100			6:81:13																				
	D-19	85	SM		>(100)	10.5				14																		
	D-20	92.5	SP-SM •		(92)	15.4	103.1	34:54:12																				
DD-12	S-1	5	ML	Af	5																							
	D-2	7.5	SM	Qya	(2)																							
	D-3	10	SP		(7)	19.3	101.5																					
	S-4	15	SP		52																							
	D-5	20	SW		(11)	14.1	113.7																					
	S-6	25	SW		92																							
	D-7	30	GP •		(16)	3.6		88:10:2																				
	D-8	35	SP		>(100)																							
	D-9	37	GP		>(100)																							
	D-10	40	(SM)		(76)	5.0	134.2	26:57:17																				
	S-11	45	SM		>100					13																		
	D-12	49	GP-GM •		(92)	6.4		50:44:6																				
	D-13	53	SM		(40)					21																		
	D-14	56	SM		(59)	17.5	114.0			46																		
	S-15	60	SM		>100			25:57:18																				
	S-16	62	SP-SM		>100	17.2																		7.35	164	85	4810	
	D-17	65	SP-SM •		(34)	16.4	108.1	0:92:8					450	41														
	S-18	69	SP-SM		>100					12																		
	D-19	70	SP		>(100)	7.9	128.1																					
	S-20	75	SM		>100			13:69:18																				
	D-21	80	(SP-SM)		92	12.1	124.1	18:70:12																				
	D-22	85	SP		>(100)																							
DD-65	D-1	10	SP	Qya	(75)	10.1	104.0																					
	S-2	15	SP		88																							
	D-3	18	SP		(71)	11.4	117.3																					
	S-4	21	SW-SM		63			14:74:12																				
	D-5	24	GP		>(100)																							
	S-6	27	GP		>100																							
	D-7	30	(SW-SM)		>(100)			40:51:9																				
	S-9	33.5	GP		>100																							
	D-10	37	(GP-GM)		>(100)	9.1	130.4	50:43:7																				
	S-12	40.5	(SW-SM)		>100			12:78:10																				

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 - ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ Insitu(saturate) Moisture Content (ohm-cm)	Sand Durability Index ASTM D 4484 (%)
DD-65	D-13	43	SP	Qya	>(100)	11.0	116.4																					
	S-15	46	SM		>100			1:86:13																				
	D-16	49.5	SP		>(100)	19.0	111.1																					
	S-18	52	(SW-SM)		>100			17:71:12																				
	D-19	56	SP		(85)	20.1	110.3																					
	S-21	59	GP		>100																							
	D-22	62	GP		>(100)																							
	D-23	63	SP		(100)	12.6	125.1																					
	S-25	66	(SP-SM)		>100			43:50:7																				
	D-26	71.5	GP		>(100)																							
SD-1	S-2	5	SM	Af	7																							
	D-3	10	SM	Qya	(5)	12.3	114.0																					
	S-4	15	SW-SM •		68			17:74:9																				
	D-5	20	GP		(15)	13.7	117.1																					
	S-6	25	SW-SM •		53			21:71:8																				
	D-7	30	GP		(20)	2.3																						
	S-8	35	SP-SM		>100					8																		
	D-9	40	GP •		>(100)	4.5		71:25:4																				
	S-10	45	SP		100																							
	D-11	50	GP		(79)																							
	D-12	52	ML		(73)	18.0	105.9	0:47:53																				
	S-13	55	SM		114																							
	D-14	60	SM		>(100)	19.8	100.7			31			1000	35														
	S-15	65	SP-SM		>100	17.4																		8.10	131	98	4545	
	D-16	70	SP-SM •		>(100)	12.9	112.1	43:49:8																				
	S-17	75	GP		>100																							
	D-18	76	GP		>(100)																							
	D-19	80	GP		>(100)	8.0	126.3																					
	S-20	85	SP-SM		>100																							
	D-21	90	SP-SM		(85)	15.6	113.3																					
	S-22	95	SP-SM		>100					9																		
SD-3	D-1	5	SP -	Qya	(5)	19.1	96.4	0:96:4																				
	S-2	10	SP		51																							
	D-3	15	SP-SM		(20)	8.8	136.7						0	54														
	S-4	20	SP-SM		76					10																		
	D-5	25	SP-SM		(42)	7.3	123.8																					
	S-6	30	SP-SM		100																							
	D-7	35	GP		-	5.5				1																		
SD-3A	D-1	35	GP	Qya	(36)	4.7																						
	S-2	40	GP-GM •		76			62:32:6																				
	D-3	41.5	GP-GM		(20)	8.8	130.4																					
	D-4	45	GP		(79)																							
	S-5	50	SM		>100																							
	D-6	56	SM		(49)	11.1	114.7																					
	S-7	60	ML		100																							
	S-8	61	SM		100					38																		
	D-9	65	SP-SM •		(64)	18.8	100.5	0:90:10					800	35			0.0340	0.0050	3.5E-04	1	-0.04							
	S-10	70	SP		100																							
	D-11	75	SW-SM		(73)	12.7	116.0	39:55:6																				
	S-12	80	GP		>100																							
	D-13	82	SP		(64)	9.6	110.1																					
	D-14	85	SP-SM •		(54)	27.4	97.1	0:89:11																				
	S-15	90	ML		93																							
	D-16	91.5	ML		(36)	22.2	105.3			76							0.0650	0.010	9.0E-04	1	+0.16							

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

95-8347-3

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 - ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-E (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ Insitu/Saturate, Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4644 (%)
SD-3A	D-17	95	SC-SM •	Qya	(55)	18.0	111.9		28,12,7	34																		
	S-18	100	SW		>100																							
	D-19	102	SW		>(100)	13.0	114.6																					
	D-20	105	SM		>(100)	17.1	116.9																					
	S-21	110	SM		>100																							
	D-22	115	SP-SM		(92)	13.4	110.6			8																		
	S-23	120	SM		>100																							
	D-24	121	SM		>(100)																							
	D-25	125	SM		(62)	20.6	107.9			37																		
SD-4	S-1	5	ML	Af	7																							
	D-2	10	SM	Qya	(5)	13.1	114.6			14																		
	S-3	15	SP		47																							
	D-4	20	(SM)		(18)	15.1	112.7	1:86:13					0	45														
	S-5	25	SW/SM		100																							
	D-6	30	GP		(24)	4.7																						
	S-7	35	SW-SM •		>100			5:88:7																				
	S-8	36	SP-SM		>(100)					12																		
	D-9	40	(SP-SM •)		>(100)	8.9		42:50:8																				
	S-10	45	GP-GM		>100					8																		
	D-11	50	GP-GM		(92)	12.5	124.4																					
	S-12	55	SP		>100																							
	D-13	60	SM		(34)	19.3	108.6			45																		
	S-14	65	SP-SM		>100	15.4																		7.95	161	196	5556	
	D-15	70	SW-SM •		>(100)	16.8	113.0	8:84:8									0.042	0.0[illegible]4	5.0E-04	1	-0.04							
	S-16	75	SM		97					41																		
	D-17	78	SW		>(100)	15.6	113.8																					
	D-18	80	SM		(67)	16.8	93.0	1:76:23																				
	S-19	85	SP		>100																							
	D-20	90	ML		(35)	18.8	112.1		27,20,7																			
	S-21	95	SM		>100																							
SD-5	S-1	5	SM	Af	8																							
	D-1	10	SM	Qya	(10)	13.0	104.8																					
	S-2	15	SM/GM		50																							
	D-2	20	(SP-SM •)		(40)	10.4	118.9	15:79:6																				
	S-3	25	SM/GM		98																							
	D-3	30	SW-SM		(52)	12.9	120.2																					
	S-4	35	(SM)		77					14																		
	S-4A	40	(SW-SM •)		113																							
	D-4	41.5	(SW-SM)		(91)	12.2	133.2	22:68:10																				
	S-5	45	(SP-SM)		>100																							
	D-5	50	GP •		(57)	9.8	131.5	79:18:3																				
	S-6	55	SP		77																							
	D-6	60	CL •		(28)	26.7	100.2		35,21,14	77																		
	S-7	65	SM		85	18.2																		5.05	131	98	2801	
	D-7	70	SP		(99)	15.5	107.8						0	45														
	S-8	75	ML		70					61																		
	D-8	80	SP-SM •		(82)	14.0	113.2	4:84:12																				
	S-9	85	SP-SM		>100																							
	D-9	90	CL		(62)	16.2	114.8		30,22,8	62							0.05	0.0[illegible]	2.5E-04	2	+0.2							
	D-10	92	SM		>(100)																							
	D-11	95	SP		>(100)																							
	D-12	100	SP		(61)	19.3	107.6																					
(B) Previous Preliminary Investigation																												
PE-18	S-1	5	ML	Qya	2																			7.74	73	109	1538/(2857)	

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 - ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-E (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	Electrical Resistivity DOT CA TEST 532 @ Insitu(saturate) Moisture Content (ohm-cm)	Slake Durability Index ASTM D 4464 (%)
PE-18	D-2	10	GW/SW	Qya	(22)	8.2	128.9																					
	S-3	15	(SP-SM)		24			1:92:7																				
	D-4	20	GW		(27)	11.0	121.3																					
	S-5	25	GW/SW		58																							
	D-6	30	GW		(55)	6.7	123.4																					
	S-7	35	(SW-SM)		>100			20:72:8																				
	D-8	40	GW		(>100)	11.9	129.3																					
	S-9	45	GW		>100			59:37:4																				
	D-10	50	GW		(>100)	7.4																						
	S-11	55	SM		>100					20																		
	D-12	60	SM		(43)	17.0	110.3	0:75:25	28,22,6		2.72						0.07	0.01	0.0014	4	-0.01							
	S-13	65	SM		>100			2:77:21																7.82	115	97	4000	
	D-14	70	SW-SM/GW-GM		(>100)	12.6	123.8																					
	S-15	78.5	SM		>100			0:84:16																				
	D-16	85	ML		(33)	20.1	107.0																					
PE-27	D-1	5	SP-SM	Qya	(4)	9.5	98.5			12																		
	S-2	10	(SP-SM)		22			33:62:5																				
	D-3	15	GP/GW		(29)	2.4																						
	S-4	20	GP/GW		40																							
	D-5	25	GP/GW		(34)	9.9	121.4																					
	S-6	30	(SP-SM)		56			23:71:6																				
	D-7	35	GP/GW		(46)	9.5	122.7																					
	S-8	40	GP		>100			80:17:3																				
	D-9	46	SW/GW		(>100)	10.7																						
	S-10	50	ML	Tf/Tp	32																							
	D-11	55	ML		(33)	25.6		0:3:97	36,29,7				31	1150														
	S-12	60	ML		29				47,30,17															7.52	183	119	7692	
	D-13	65	ML		(36)	24.6			29,28,1			8.841																
	S-14	70	ML		51																							
	D-15	75	ML		(30)	24.6			47,27,20			10.699																
PE-28	D-1	5	SM/SC	Qya	(2)	22.0	100.6																					
	S-2	10	(SP-SM)		46			30:60:10																				
	D-3	15	SP		(30)	9.2	129.5	46:51:3																				
	S-5	25	SP-SM		>100			2:86:12		12																		
	D-6	30	SP		(58)	14.5	116.4																					
	B-7	36.5	GW/GP		>100																							
	D-9	45	CL/CH	Tf/Tp	(40)	31.7	91.6		50,20,29				29	700														
	S-10	50	ML		>100				29,23,6																			
	D-11	55	ML		(35)	25.5	100.1		38,26,12			7.848																
	B-12	62	CL		>100																							
	D-13	65	CL		(80)	22.0	102.8		42,22,20			4.853																
	S-14	70	CL		100				49,23,26															7.71	701	777	1333	
	D-15	75	CL		(70)	23.9	101.2	0:2:98	42,25,17				31	1250														
	S-16	80	CL		>100																							
PE-29	D-1	5	SM	Qya	(17)	17.7	106.4																					
	S-2	10	SM		14																							
	D-3	15	GP/GW		(16)	5.0																						
	S-4	20	(SM)		62			2:77:21																				
	D-5	25.5	GP/GW		(>100)																							
	S-6	32	SP-SM		>100			20:69:11																				
	D-7	35	GP		(73)	7.0	128.0																					
	S-8	41	GP		>100			95:4:1																				
	D-9	45	GP-GM		(34)	9.0	123.6																					
	S-10	50	GP		>100																							

TABLE 3-5 SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Sample Number	Sample Depth (ft)	USCS Classification ASTM D2487 • ASTM D2488	Geologic Unit	Equivalent SPT Value	Moisture Content** ASTM D2216 (%)	Dry Density ASTM D2937 (pcf)	Grain-Size Distribution ASTM D422 GR:SA:FI (%)	Atterberg Limits ASTM D4318 LL,PL,PI	Percent Passing #200 Sieve ASTM D1140 (%)	Specific Gravity ASTM D 854	Unconfined Compressive Strength ASTM D2166 (ksf)	Direct Shear ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Direct Shear ASTM D3080 (Peak Strength) Effective Friction Angle (DEGREES)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Cohesion (PSF)	Triaxial Compression ASTM D3080 (Peak Strength) Effective Friction Angle (DEGREES)	Consolidation Characteristics ASTM D2435 Cc	Cs	Cx	Vertical Stress (ksf)	Swell(+)/ Collapse(-) (%)	Triaxial Permeability ASTM D5084 Effective Confining Stress (psi)	(K20) (cm/sec)	Soil pH USEPA Method 9045	Sulfate Content DOT CA Test 417-B (EPA 300) (ppm)	Chloride Content DOT CA Test 422 (EPA 300) (ppm)	DC ⊕ M
PE-29	D-11	55	GP-GM	Qya	(>100)	8.8	98.2																				
	S-12	61	(SW-SM)		>100			21:68:11																2.37	1645	203	
	D-13	65	SW-SM/GP-GM		(68)	11.6	124.7																				
	S-14	70	CH	Tf/Tp	58				53,22,31																		
	D-15	75	CH		(30)				54,22,32			17.208															
	S-16	80	CH		59																						
PE-30	S-2	10	(SP-SM)	Qya	22			22:71:7																			
	D-3	15	SP-SM/GP-GM		(17)																						
	S-4	25	GP-GM		>100																						
	D-5	30	SP		(77)																						
	D-9	50	(SP-SM)		(67)	16.3		0:89:11																6.74	163	674	
	S-10	55	GW-GM		>100																						
	S-14	75	CH/MH	Tf/Tp	58				69,33,36																		
	S-15	80	CH/MH		>100																						
PE-31	D-1	5	SP	Qya	(7)	14.8	90.3																				
	S-2	10	GP/SP		20																						
	D-3	15	GP/SP		(21)	10.7	121.7																				
	S-4	20	GP/SP		26																						
	D-5	25	GP		(40)	25.3		85:13:2																			
	S-6	30	GP/SP		66																						
	D-7	35	GP/SP		(73)	8.8																					
	S-8	40	GP/SP		>100																						
	D-9	45	GP/SP		(>100)	13.4	119.0																				
	S-10	50	(SP-SM)		69			41:51:8																			
	D-11	55	SM		(46)	17.2	111.3	0:79:21																			
	S-12	60	ML/CL		>100																						
	D-13	65	ML		(23)	27.3	99.7	0:44:56					31	950													
	S-14	75	ML/CL	Tf/Tp	50																						
	S-15	80	ML/CL		59																						

NOTES:

1. USCS Classifications are based on the visual-manual procedure (ASTM 2488) and laboratory test results (ASTM D2487). USCS Classifications based on the laboratory test results (ASTM 2487) are identified by th
2. Some of the material classifications (identified in parantheses) shown in column 3 of this table are not consistent with the general classification of that interval in the boring logs. This generally occurs in gravelly and where due to presence of gravels and cobbles larger than sampler size, laboratory gradations and classifications only reflect the finer matrix materials in alluvium.••
3. For the same reasons presented above, results of gradation, fines content, in situ moisture content and in situ dry density tests in layers identified as gravels, silty/clayey gravel, gravel with sand and sand with gravel, may not be truly representative.
4. For California Drive Samples, Equivalent SPT values were obtained by applying the appropriate corrections for different hammer weights, hammer drop and sampler dimensions, Equivalent SPT values corrected from drive sampler blowcounts are shown in parantheses.
5. Equivalent SPT values in alluvium may not be representative of material density/consistency due to the presence of gravels, cobbles and boulders.
6. Cc and Cs are based on vertical strain-log stress plots, Cx is based on vertical strain-log time plot
7. Explanation of symbols:
   GR:SA:FI = Gravel: Sand : Fines (percent passing #200 sieve)
   LL,PL,PI = Liquid Limit, Plastic Limit, Plasticity Index

Table 3-6: Summary of Laboratory Results for Groundwater Samples

Constituent	Method of Analysis	Sample Identification and Collection Date					MCL
		7/7/95	7/15/95	7/16/95	8/31/95	9/7/95	
		DD-3C Test Production Well			DD-1	DD-2	
		DD-3C Sample: At the End of Well Development	DD-3C-1 Sample: After Pumping Well for 700 Minutes at 60 gpm	DD-3C-2 Sample: After Pumping Well for 1300 Minutes at 60 gpm	Monitoring Well After Bailing and Surging Development and Purging	Monitoring Well After Bailing and Surging Development and Purging	Maximum Contaminant Level
		mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
Organic Lead	DHS LUST	<0.05	<0.05	<0.05	<0.05	<0.05	NA
Total Petroleum Hydrocarbons	8015M	0.6	4.9	5.6	13	0.3	NA
Oil & Grease	413.2	<1	<1	<1	4.5	<1	NA
Bicarbonate Alkalinity	310.1	NA	648	485	NA	NA	NA
Calcium	3005/6010	259	239	214	NA	248	NA
Chloride	300	185	**259**	**271**	**461**	**261**	250 (secondary)
Iron	3005/6010	0.166	<0.05	0.062	NA	0.142	0.3 (secondary)
Magnesium	3005/6010	83.8	75	69.8	NA	84.2	NA
Manganese	3005/6010	**2.3**	**1.76**	**1.56**	NA	**2.38**	0.05 (secondary)
Potassium	3005/6010	7.57	6.48	6.53	NA	8.33	NA
Sodium	3005/6010	181	244	252	NA	187	NA
Total Sulfide	376.1	<0.2	45.66	53.8	80	18	NA
Sulfate	300	**544**	**385**	**340**	77.8	**394**	250 (secondary)
Title 22 Metals		ug/L	ug/L	ug/L	ug/L	ug/L	ug/L
Arsenic	3005/6010	<100	<100	<100	<60	<100	50
Barium	3005/6010	100	148	148	881	76.9	1000
Cadmium	3005/6010	<5	<5	<5	<5	<5	10
Chromium	3005/6010	<10	<10	<10	<10	<10	50
Copper	3005/6010	<10	<10	<10	<10	<10	1,000 (secondary)
Lead	3005/6010	<100	<100	<100	<100	<100	5,000 (secondary)
Nickel	3005/6010	<20	<20	<20	<20	<20	100 (EPA)
Silver	3005/6010	<10	<10	<10	<10	<10	50
Zinc	3005/6010	<10	<10	<10	<10	<10	15 (EPA)
Organic Compounds		ug/L	ug/L	ug/L	ug/L	ug/L	ug/L
Benzene	602	**97**	**290**	**310**	**500**	**250**	1
Benzene	624	**140**	**300**	**420**	**320**	**200**	1
Toluene	602	<1	31	40	90	<5	1,000
Toluene	624	<1	30	53	150	1.8	1,000
Ethylbenzene	602	4.1	350	290	160	<5	680
Ethylbenzene	624	2.8	310	460	400	2.3	680
Total Xylenes	602	4.5	230	280	400	300	1,750
Total Xylenes	624	<1	270	380	240	<1	1,750
1,1-Dichoroethane	624	1.8	<5	<5	2.6	2.3	5
2-Methylnaphthalene	625	<10	71	90	699	<10	NA
Acenaphthene	625	<10	<10	<10	20	<10	NA
Acenaphthylene	625	<10	<10	<10	140	<10	NA
Anthracene	625	<10	<10	<10	11	<10	NA
Carbazole	625	<10	<10	<10	110	<10	NA
cis-1,2-Dichloroethene	624	**17**	**27**	**40**	**36**	**22**	6
Dibenzofuran	625	<10	<10	<10	15	<10	NA
Fluorene	625	<10	<10	<10	84	<10	NA
Naphthalene	625	15	3,100	2,400	2,930	<10	NA
Phenanthrene	625	<10	<10	<10	75	<10	NA
trans-1,2-Dichloroethene	624	3	<5	<5	3.2	4.6	10
Trichloroethene	624	<1	<5	<5	1.6	<1	5
Vinyl Chloride	624	<1	**23**	<10	**55**	**41**	0.5

DHS LUFT: California Department of Health Services Leaking Underground Storage Tank Program (Flame-atomic absorption spectrophotometer)
NA: Not Available or Not Analyzed
Secondary: State of California Secondary Drinking Water Quality Criteria - Not a Legal Standard
Bold number indicates MCL is exceeded for that constituent.

- One groundwater sample each from Monitoring Wells DD-1 and DD-2.

The test program, relevant test standards and the results of the analytical testing of groundwater are summarized in Table 3-6 and presented in Appendix C.

3.4.2.2 Soils

An analytical (chemical) test program was performed on selected soil samples with high OVA readings from Borings DD-2S, DD-4-1, and DD-65. The test program and test results are summarized in Table 3-7. Detailed results are presented in Appendix C.

3.5 FIELD OBSERVATIONS

Table 3-8 summarizes various field observations noted in this investigation during drilling sampling and development of monitoring wells for groundwater quality samples. These observations include the following:

- Locations and approximate sizes of cobbles and boulders
- Occurrence of caving and/or loss of circulation of drill mud
- Depth intervals of potential hydrocarbon contamination
- Depth intervals of sulfurous and/or hydrocarbon odor
- Measured hydrogen sulfide during well development and groundwater quality sampling
- Locations and concentrations of volatile organic vapor (OVA) of soil samples with concentrations at least 10 ppm higher than the corresponding background readings.

Table 3-7. Summary of Chemical Laboratory Test Results for Soil Samples

Constituent	Method of Analysis	Sample Identification and Collection Date									Regulatory Goals	
		12/6/95	12/6/95	12/6/95	11/11/95	11/11/95	11/11/95	12/8/95	12/8/95	12/8/95		
		DD-2S, E-22	DD-2S, E-25	DD-2S, E-36	DD-4-1/ P-14	DD-4-1/ P-18	DD-4-1/ P-22	DD-65, E-17	DD-65, E-20	DD-65, E-24	LA County UST Program Maximum Acceptable Levels (MALs)	US EPA Preliminary Remediation Goals (PRGs)
		mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
Organic Lead	DHS LUFT	-	-	-	<0.05	<0.05	<0.05	-	-	-		NA
Total Petroleum Hydrocarbons *	8015M	2.21	1.97	<0.5	<2.44	<2.48	<2.49	46	35	38	1,000/100/10* (Gasoline) 10,000/1,000/100* (Diesel)	NA
Oil & Grease	413.2	-	-	-	<12.2	<12.4	<12.4	-	-	-		NA
Chloride	300	-	-	-	180	150	210	-	-	-		NA
Total Sulfide	9030	-	-	-	30.49	13.37	13.43	-	-	-		NA
Sulfide	376.1	-	-	-	940	690	800	-	-	-		NA
Title 22 Metals		mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
Antimony	3005/6010	-	-	-	<7.32	<7.43	<7.46	<6	-	-		680i
Arsenic	3005/6010	-	-	-	<12.2	13.2	<12.4	<10	-	-		32r
Barium	3005/6010	-	-	-	207	189	187	52.3	-	-		100,000i
Beryllium	3005/6010	-	-	-	<0.61	<0.62	<0.62	<0.5	-	-		1.1i
Cadmium	3005/6010	-	-	-	<0.61	<0.62	<0.62	<0.5	-	-		850i
Chromium	3005/6010	-	-	-	32.6	34.9	34.7	4.11	-	-		1,600i(Cr VI)
Cobalt	3005/6010	-	-	-	9.83	8.9	9.03	3.22	-	-		NA
Copper	3005/6010	-	-	-	32.9	30.5	33.9	7.7	-	-		63,000i
Lead	3005/6010	-	-	-	<12.2	<12.4	<12.4	<10	-	-		1,200i
Mercury	7471	-	-	-	<0.12	0.12	<0.12	-	<0.1	-		510i
Molybdenum	3005/6010	-	-	-	<6.1	<6.19	<6.22	<5	-	-		8,500i
Nickel	3005/6010	-	-	-	25.7	26.5	26.4	3.08	-	-		34,000i
Selenium	3005/6010	-	-	-	<24.4	<24.8	<24.9	<20	-	-		8,500i
Silver	3005/6010	-	-	-	<1.22	<1.24	<1.24	<1	-	-		8,500i
Thallium	3005/6010	-	-	-	<0.61	<61.9	<62.2	<50	-	-		150i (compds)
Vanadium	3005/6010	-	-	-	43.8	43	44.9	13	-	-		12,000i
Zinc	3005/6010	-	-	-	75.6	72.8	75.7	41.4	-	-		100,000i
Organic Compounds		ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg	ug/kg
Benzene *	8240	410	160	<1	<1.22	<1.22	<1.22			7	1/0.3/NA	4,400i
Toluene *	8240	80	28	<1	<1.22	<1.22	<1.22			<1	50/0.3/NA	2,700,000i
Ethylbenzene *	8240	3,800	800	<1	<1.22	<1.22	<1.22			23	50/1/NA	3,100,000i
Total Xylenes *	8240	32,000	660	<1	<1.22	<1.22	<1.22			29	50/1/NA	980,000i
Acenaphthylene	8270	<330	-	-	<402	<408	<410	380	-	-		NA
Benzidine	8270	<800	-	-	<976	<990	<995	14,000	-	-		
Fluorene	8270	<330	-	-	<402	<408	<410	650	-	-		NA
2-Methyl-naphthalene	8270	<330	-	-	<402	<408	<410	510	-	-		NA
bis (2-Ethylhexyl) phthalate	8270	<330	-	-	<402	<408	<410	<330	-	-		NA
di-n-Butyl-phthalate	8270	<330	-	-	<402	<408	<410	<330	-	-		NA
Naphthalene	8270	3,300	-	-	<402	<408	<410	2,000	-	-		
Phenanthrene	8270	<330	-	-	<402	<408	<410	620	-	-		NA

mg/kg: milligrams per kilogram
ug/kg: micrograms per kilogram

* Total Petroleum Hydrocarbon or BTEX Content Ranges for MAL Assessment Based Largely on Depth to Groundwater
MAL Assessment Criteria for Groundwater >100 feet / 51 to 100 feet / 25 to 50 feet below ground surface

DHS LUFT California Department of Health Services Leaking Underground Storage Tank Program (Flame-atomic absorption spectrophotometer)

i industrial area PRG r residential area PRG Cr VI Hexavalent Chromium

TABLE 3-8 SUMMARY OF VARIOUS FIELD OBSERVATIONS DURING DRILLING SAMPLING AND DEVELOPMENT OF MONITORING WELLS

Boring/ Well No.	Approximate Station Along CR/CL Track	Approximate Depth to Tunnel Invert/Station Bottom (ft)	Groundwater Depth measured on 9/22/95 (ft)	Boulder Size/Depth	Cobbles Size/Depth	Depth of Caving or Loss of Circulation	Depth of Detected Sulfur Odor (ft)	Depth of Detected Hydrocarbon Odor (ft)	OVA Reading Depth/OVA(1) (ft/ppm)	REMARKS
DD-1	CL 21+70	62.5	31.1		6"-8"/33.5'-34'		35	20	5/78	Monitoring well installed
					*/36.5'-39'		45	40	25/120	
					7"/42.5'			52	55/85	
					8"/47.0'			55	65/940	
					8"/49.0'			65	72/440	
					4"/64.0'			70	75/400	
								84.5-88	80/120	
									85/18	
DD-2	CL 25+50	73.5	31.6					30.5		Boring terminated prior to reaching planned penetration depth of 62 feet
								62		due to encountering free hydrocarbon products in the ground water
										between 39 feet and 61 feet.
DD-2S	CL 25+51	74			*/13'		30		55/15	No monitoring well installed.
					*/17'		35		58/18	
					*/23'		51		61/460	
					*/28'		58		61/480	
					*/34'		60.5		67/320	
					6"/34'-37'		65.5		72/78	
					6"/40'		68.5		75/180	
					*/70'-72'		79		80/170	
					*/74'		80		87/340	
							96		95.5/>1000	
DD-3	CL 28+68	71	35.6	small/36'	4"/15'		50		55/142	Monitoring well installed.
					3"/25'		55		60/480	
					9"/36'		60		65/130	
					4"-8"/42'-43'					
					4"-6"/46'					
					6"/49'					
					*/72'					
DD-3C	CL 28+43	71	35.7		*/14.5'					Monitoring well installed.

TABLE 3-8 SUMMARY OF VARIOUS FIELD OBSERVATIONS DURING DRILLING SAMPLING AND DEVELOPMENT OF MONITORING WELLS

Boring/ Well No.	Approximate Station Along CR/CL Track	Approximate Depth to Tunnel Invert/Station Bottom (ft)	Groundwater Depth measured on 9/22/95 (ft)	Boulder Size/Depth	Cobbles Size/Depth	Depth of Caving or Loss of Circulation	Depth of Detected Sulfur Odor (ft)	Depth of Detected Hydrocarbon Odor (ft)	OVA Reading Depth/OVA(1) (ft/ppm)	REMARKS
DD-3C					*/32.5'		53.5	33	60/920	
					8"/36.5'		69	40	65/130	
					4"-8"/38'				64/69	
					8"/40'				75/320	
					*/45'					
					4"/49'					
DD-3D	CL 28+80	71	-	1 1/2'/25'	*/14.5'	caving at 12'				Terminated at 36.5 feet prior to reaching planned penetration depth
					*/16'-19'	caving at 13'				of 90' due to caving conditions and loss of circulation of drill mud.
					*/23'-25'	caving at 36.5'				
					*/30'-31'					
					*/33'-35'					
DD-4	CL 35+60	69	42.5	13"/38'	*/8'			1	6/160	Monitoring well installed.
					5"/28.5'			6	20/54	
					3"-7"/30'			15	81.5/120	
					4"/35'				84.5/140	
					6"/44.5'				87.5/180	
					4"/54'				90.5/160	
DD-4-1	CL 31+95	75			*/13'		72	32	25/36	Monitoring well installed
					*/19'			34.5	30/78	
					*/23'			42	35/330	
					*/28'			45	40/64	
					*/32'			46	43/54	
					6"/32'-33'			49	46/62	
					6"/34'-34.5			52	49/140	
					7"/39'			55	52/>1000	
					*/42'				55/>1000	
									58/>1000	
									61/72	
									64/>1000	

TABLE 3-8 SUMMARY OF VARIOUS FIELD OBSERVATIONS DURING DRILLING SAMPLING AND DEVELOPMENT OF MONITORING WELLS

Boring/ Well No.	Approximate Station Along CR/CL Track	Approximate Depth to Tunnel Invert/Station Bottom (ft)	Groundwater Depth measured on 9/22/95 (ft)	Boulder Size/Depth	Cobbles Size/Depth	Depth of Caving or Loss of Circulation	Depth of Detected Sulfur Odor (ft)	Depth of Detected Hydrocarbon Odor (ft)	OVA Reading Depth/OVA(1) (ft/ppm)	REMARKS
DD-4-1									68/>1000	
									70/>1000	
									73/620	
									76/>1000	
									79/>1000	
									82/>1000	
									84.5/740	
DD-5	CL 43+70	65	-		6"/4.5'	Caving below 30'				Bucket Auger
					**/12.5					Boring terminated at 32 5 feet before reaching planned penetration depth of 75 feet
					**/28.5'					No monitoring well installed. No GW observed during drilling.
					12/29.5'					
DD-6	CR 25+60	73	-		3.5"/60'				Not measured.	Becker Hammer Drilling. No soil sampling from 0-58 feet.
					3.5"/62					
DD-7	CR 32+05	71	41.3		4"/29'		43.5		20/50	Monitoring well installed.
					4"/33'				40/17	
					4"-6"/39'				45/66	
					4"/54'					
DD-9	CR 34+75	65	54.7		6"/37'				5/110	Monitoring well installed.
					*/44"				10/16.5	
					6"/51'				55/16	
									59/18	
									61/16	
									62/28	
									63.5/88	
									66.5/84	
									67.5/80	
									69/72	
									72/100	
									75/74	
									78/180	

TABLE 3-8 SUMMARY OF VARIOUS FIELD OBSERVATIONS DURING DRILLING SAMPLING AND DEVELOPMENT OF MONITORING WELLS

Boring/ Well No.	Approximate Station Along CR/CL Track	Approximate Depth to Tunnel Invert/Station Bottom (ft)	Groundwater Depth measured on 9/22/95 (ft)	Boulder Size/Depth	Cobbles Size/Depth	Depth of Caving or Loss of Circulation	Depth of Detected Sulfur Odor (ft)	Depth of Detected Hydrocarbon Odor (ft)	OVA Reading Depth/OVA(1) (ft/ppm)	REMARKS
DD 9									81/40	
									84/180	
									87/27	
DD-10	CR 36+35	64	74.9	1'/33'	*/17'	Loss of circulation at 47'		40	7.5/96	Monitoring well installed
					6"-7"/30'			44	40/90	
					*/34'-35'			52	45/460	
					*/37'				70/18	
					*/41'-44'				75/18	
					*/48'-50'				92.5/16	
					*/55'-59.5'					
					*/64'					
					*/82'-84'					
					*/91'					
DD-12	CR 39+50	63	Dry		*/36'					Monitoring well installed
					6'-8"/42'					
					*/52'					
DD-65	CL 19+10	57			*/5'	36'	47		27/19	No monitoring well installed.
					5"/7'		54		40.5/22	
					*/22.5'		57		43/76	
					6"/26'		60		49.5/43	
					*/29'		67		52/24	
					*/32'		71.5		56/72	
					5"-6"/36'-37'				63/39	
					3.5/39'				71.5/18	
					*/55'-56'					
					5"/59'					
					7"/68'-71.5'					
					*/75'					
SD-1	CL 40+60	65	-		8"-10"/33'					No monitoring well installed
					*/39'					Groundwater encountered at 91 feet during drilling.
					*/41'-42'					

TABLE 3-8 SUMMARY OF VARIOUS FIELD OBSERVATIONS DURING DRILLING SAMPLING AND DEVELOPMENT OF MONITORING WELLS

Boring/ Well No.	Approximate Station Along CR/CL Track	Approximate Depth to Tunnel Invert/Station Bottom (ft)	Groundwater Depth measured on 9/22/95 (ft)	Boulder Size/Depth	Cobbles Size/Depth	Depth of Caving or Loss of Circulation	Depth of Detected Sulfur Odor (ft)	Depth of Detected Hydrocarbon Odor (ft)	OVA Reading Depth/OVA(1) (ft/ppm)	REMARKS
SD-1					*/44'-45'					
					*/68'					
					*/73'					
					*/76'					
SD-2	CL 42+45	63	-							Boring terminated at 34.5 feet prior to reaching planned penetration depth at 85'
										due to encountering free hydrocarbon product between 26 feet and 34.5 feet.
SD-3	CL 43+95	63	-		*/35.5'	Loss of circulation at 34' and 35'				Boring terminated at 35.5 ' due to excessive mud loss.
SD-3A	CL 43+55	63	-		6"-8"/24'					No monitoring well installed.
					10"/35'					Groundwater encountered at 88 feet during drilling.
					12"/39'					
					8"-10"/52'-54'					
SD-4	CR 45+55	63	-		8"/38'				36/60	No monitoring well installed
					*/41'-45'					
					*/52'-53.5'					
SD-5	CL 46+80	63	78.3		**18.5'				20/78	Monitoring well installed.
					*/31'				30/18	
					*/35'					
					6"/38'					
					6"/41.5'					
					7"/50'					

NOTES:

(1) OVA Concentration >= 10 ppm than the corresponding background readings.

* Drill rig chatter probably encountered gravels/cobbles.

** Trace cobbles.

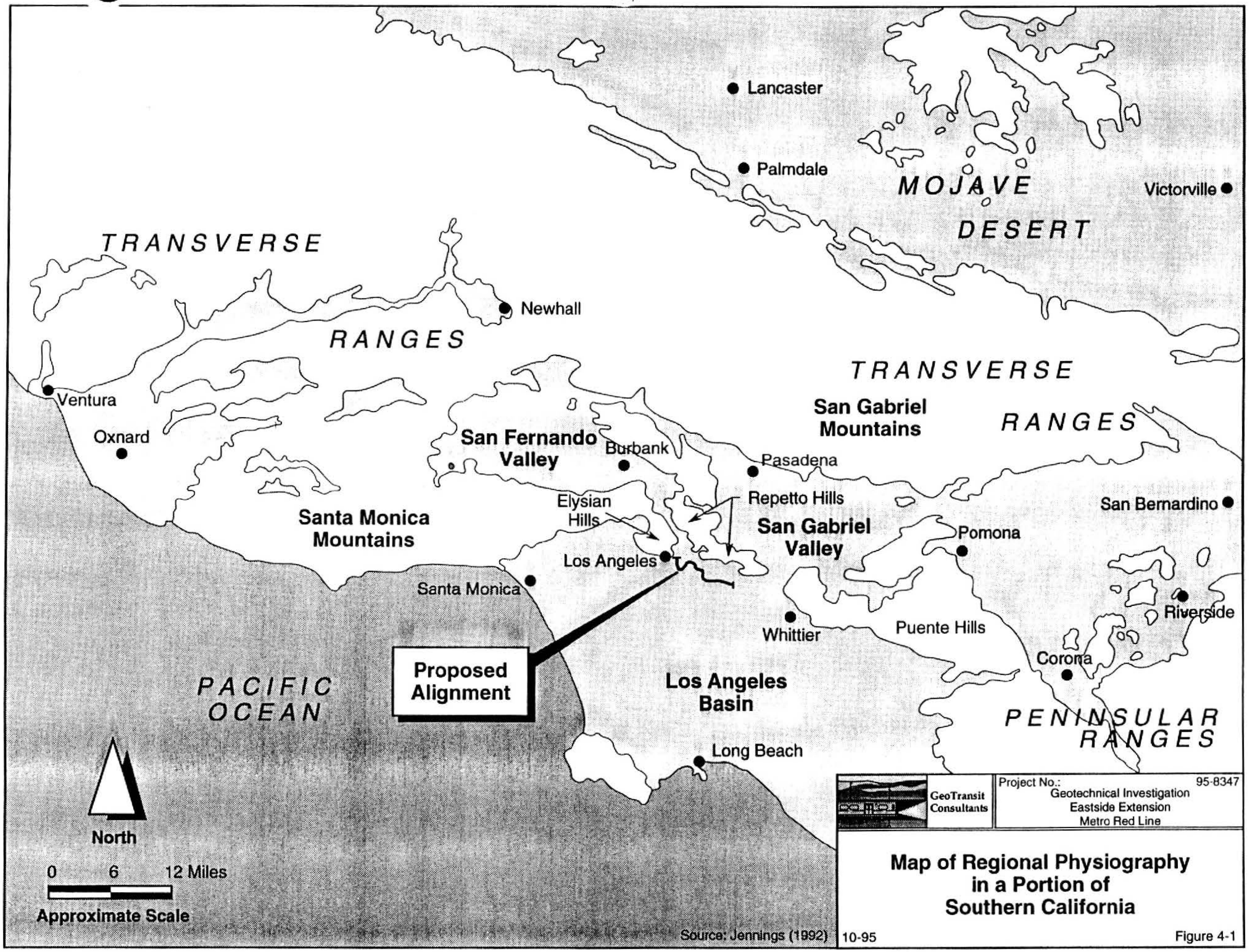
4.0 GEOLOGIC AND GROUNDWATER CONDITIONS

4.1 REGIONAL SETTING

4.1.1 Regional Geology

The proposed Metro Rail Eastside Extension alignment is on the northern edge of the Los Angeles coastal plain and the underlying structural basin, at the junction between the Transverse Ranges and Peninsular Ranges 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. The western Transverse Ranges are uplifted by northward-dipping thrust faults along their southern margin. The hilly terrain of the Eastside Extension 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 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.



Map of Regional Physiography in a Portion of Southern California

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.

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.

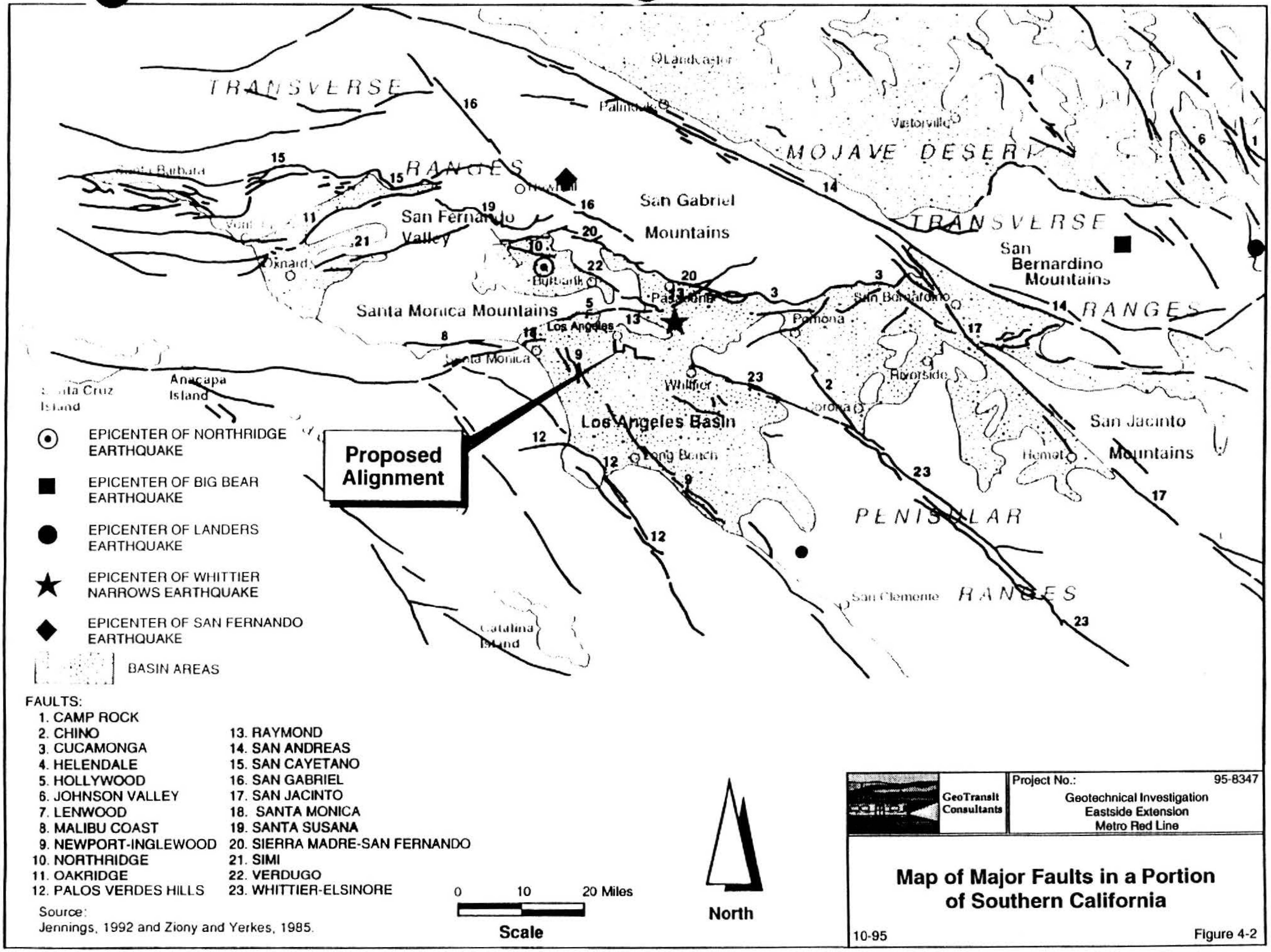
4.1.2 Regional Faulting and Seismicity

4.1.2.1 Regional 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 system, 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 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 documented active faults closest 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



Map of Major Faults in a Portion of Southern California

Figure 4-2

Monica Mountains about 5 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 5 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 8 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 1). An east-west-trending linear escarpment in alluvium that crosses the alignment at three locations (approximate Stations CR 35+50, CR 108+00 and CR 154+00) east of the project area, 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 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 a separate report (GeoTransit Consultants, 1996).

4.1.2.2 Regional 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 7.9 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

TABLE 4-1. ESTIMATED SEISMIC CHARACTERISTICS OF PRINCIPAL FAULTS

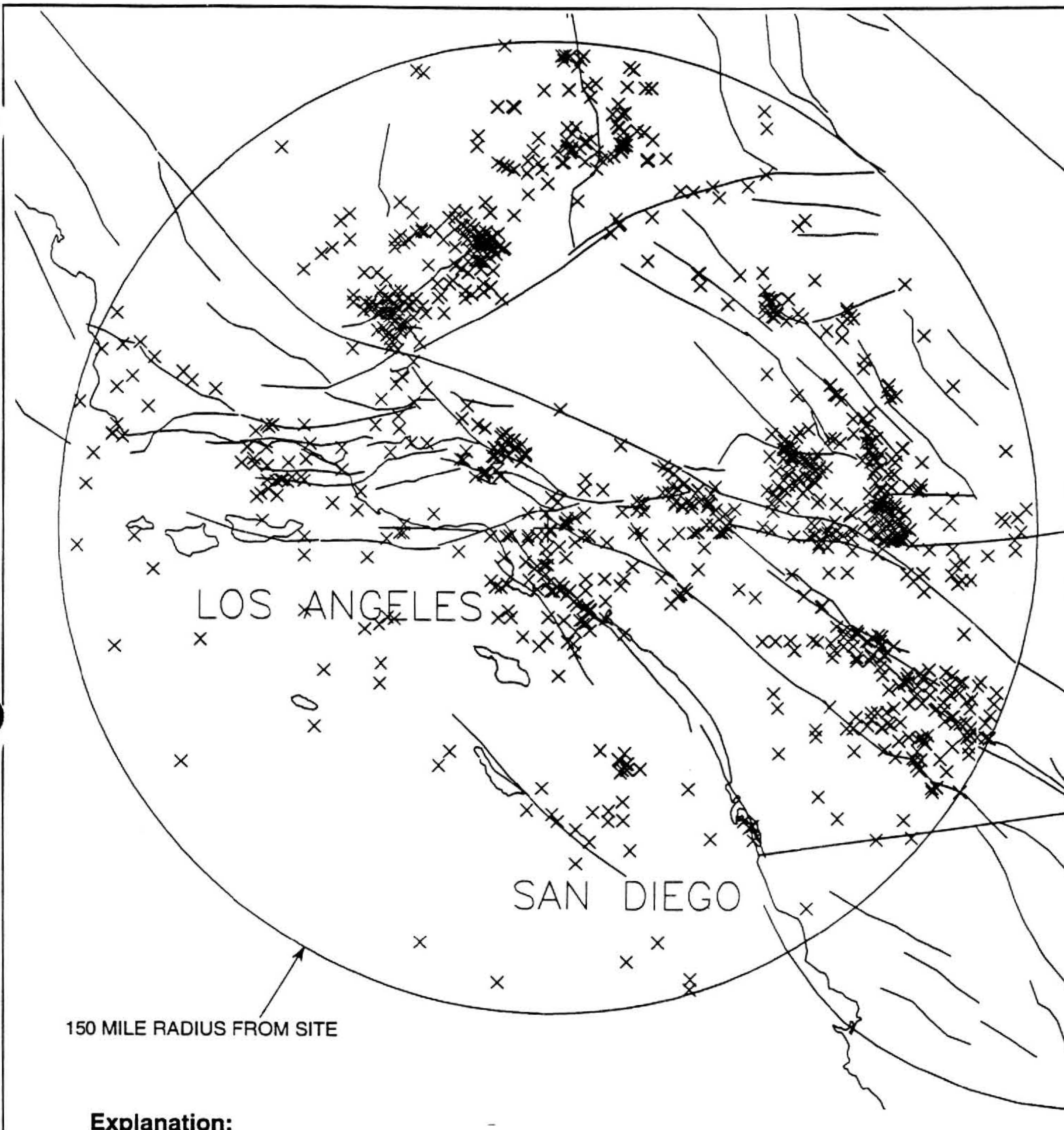
Fault	Approximate Distance from Station[1] (miles)	Magnitude of Maximum Credible Earthquake[2]	Age of Most Recent Displacement[3]
Chino	27	7 1/2	Late Quaternary
Cucamonga	29	7	Holocene
Hollywood	7	7 1/2	Holocene
Malibu Coast	24	7 1/2	Holocene
Newport-Inglewood	9	7	Historic (1933)
Northridge	23	7 1/2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



Explanation:

× M = 4.0–4.9

Site Location(+):

Latitude - 34.0340 N
Longitude - 118.1920 W

North

0 50 100 Miles
Scale

Source:
Epicenters from Blake, 1992.

GeoTransit Consultants

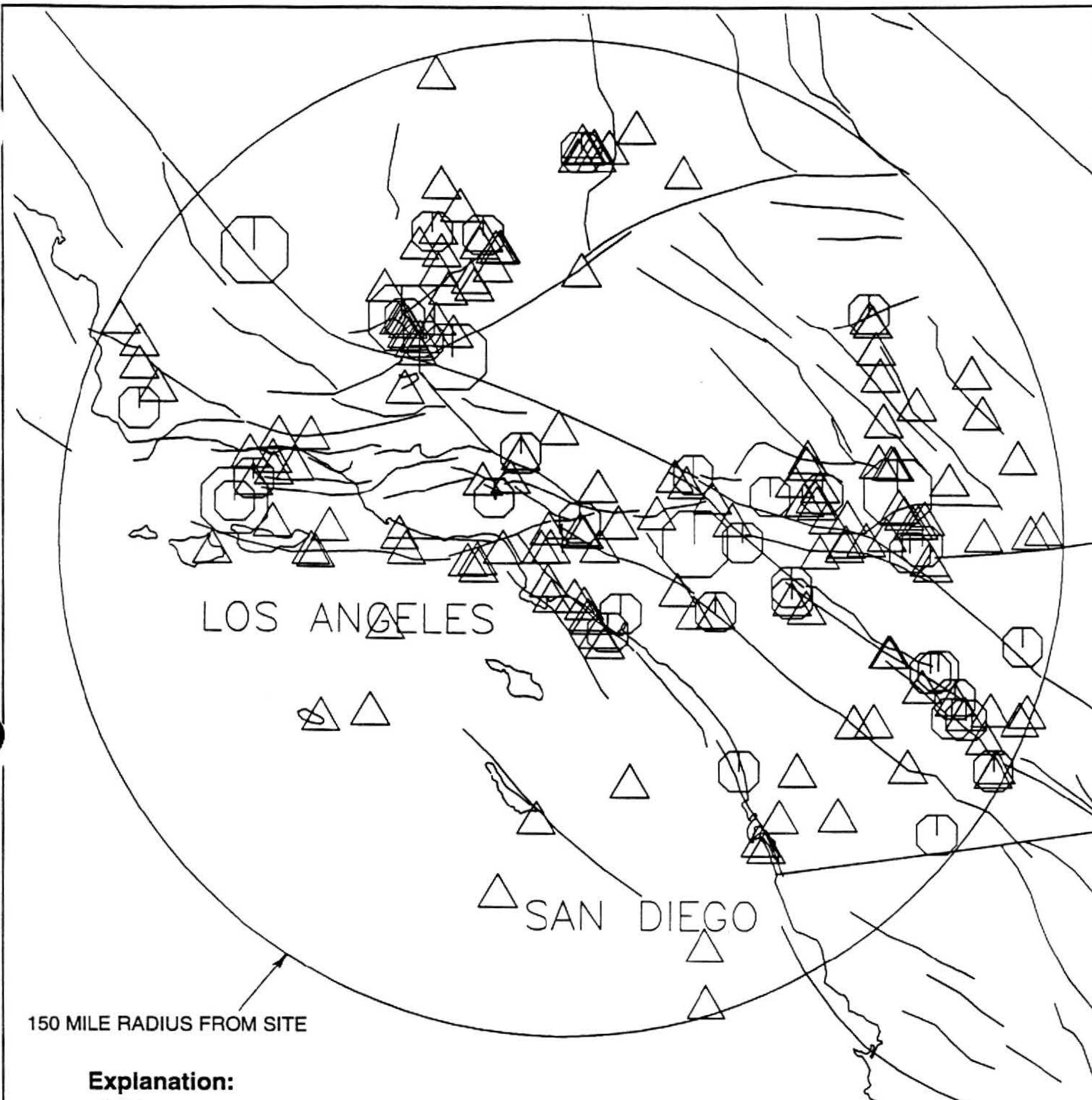
Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

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

10-95

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

Source:
Epicenters from Blake, 1992.

GeoTransit Consultants

Project No.: 95-8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

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

10-95

Figure 4-4

closest moderate-sized historical earthquake was that of the 1987 Whittier Narrows earthquake (M 5.9), with an epicenter about 10.5 miles east-northeast of the subject 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 northwest of the alignment. Records of ground accelerations released by the California Division of Mines and Geology for a strong ground motion instrument located in the vicinity of City Terrace (approximately 3 miles northeast of the project area) indicates maximum free field accelerations of 0.32g horizontal and 0.13g vertical for the January 17, 1994 earthquake.

4.1.3 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 have supplied 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. The Central Basin of the coastal plain is further subdivided into the Los Angeles Forebay, Montebello Forebay, Whittier, and Central Basin Pressure Areas. The subject segment of 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 subject segment of the alignment include the Semiperched, Gaspur, Exposition, Gage, and Gardena aquifers at increasing depths in the Holocene and Pleistocene sediments (CDWR, 1961). Because bedrock occurs at relatively shallow depths along the subject segment, only the upper Semiperched and Gaspur aquifers appear to be present (CDWR, 1961). The semiperched aquifer consists 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 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.

4.2 LOCAL SETTING

4.2.1 Local Geologic Conditions

Unconsolidated to weakly consolidated Holocene alluvial sediments, and consolidated bedrock of the Miocene and Pliocene age Puente and Fernando formations will be encountered during construction of the tunnels and station in the project area. Plates 4 and 5 illustrate the subsurface conditions along the alignment based on the results of available information.

4.2.1.1 Surficial Deposits

The subject tunnels and station will be mostly in alluvium deposited by the Los Angeles River. Both granular and fine-grained intervals occur in the 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 observed clasts range up to 4 feet in size (Converse and others, 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 alluvium in the Narrows area is characterized by a zone of boulders and cobbles [Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants (CCI/ESA/CRC), 1984]. This condition was found in Borings DD-3, DD-3D, DD-4, DD-10, PE-28, PE-29, and PE-30, where clasts up to 3 feet in size were encountered.

4.2.1.2 Bedrock

Bedrock strata of the Pliocene Fernando Formation crop out 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 1, 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). Also, engineering properties of both formations are considered similar. 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 the project area. Within the borings, the bedrock material consists of very poorly bedded to distinctly bedded siltstone and claystone with cemented beds and concretions locally present. The bedrock is typically dark olive gray and generally appears to be fresh or unweathered.

Where observed in the borings, bedding planes have variable inclinations, ranging from less than 20 degrees (Boring PE-29) to 45 degrees (Boring PE-28). 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 ½-inch thick) was encountered in Boring PE-28, and a thinner cemented sandstone zone was encountered in Boring DD-7. Drilling through this interval resulted in continuous rig chatter and slow progress. Although Borings PE-28 and DD-7 were the only borings in which cemented materials were encountered during the investigations, the available literature indicates that cemented beds, lenses and nodules, locally up to 12 feet thick, are present (Lamar, 1970; Converse, Davis and Associates, 1975; LeRoy Crandall and Associates, 1979; Converse and others, 1981 and 1984).

4.2.2 Local Faulting and Folding

An east-west trending topographic escarpment (the Coyote Pass escarpment) forms the southern margin of the City Terrace area in the Repetto Hills and is as much as 80 feet high. It 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 1). The escarpment is highest along the southern edge of the heights of City Terrace in East Los Angeles and diminishes to an indistinct feature that is less than 20 feet high near its intersection with the tunnel alignment east of the proposed Little Tokyo Station and approaching tunnels. A second topographic escarpment occurs approximately 1 mile south of the Coyote Pass escarpment. The southerly escarpment has an east-northeast to northeast trend, and its surface expression is relatively subdued when compared to the Coyote Pass escarpment.

Geologic studies following the 1987 Whittier Narrows earthquake (M 5.9) attribute these and similar escarpments in the Elysian 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). Concurrent investigations carried out by GeoTransit Consultants to evaluate the Coyote Pass escarpment conclude that the escarpment as well as the southerly escarpment are primarily the result of fold deformations associated with faulting at some unknown depth. These investigations also suggest that the Coyote Pass escarpment projects from the heights of East Los Angeles, west across the Los Angeles River floodplain, and possibly into downtown Los Angeles. The projected trace of the escarpment intersects the subject tunnels in the vicinity of the First Street bridge.

No published active faults trend toward or cross the project site and there are no Alquist-Priolo earthquake fault zones identified by the State in the area.

4.2.3 Coyote Pass Escarpment, Bedrock High and Groundwater Barrier

Preliminary geotechnical explorations revealed an apparent bedrock high and possible groundwater barrier aligned with the projection of the Coyote Pass escarpment from the heights of East Los Angeles across the Los Angeles River area.

Six of the geotechnical borings on South Santa Fe Avenue (Plate 1) were used to help understand the subsurface configuration of the bedrock surface across the escarpment's projection. The spacing between these borings was limited to an approximate average of 150 feet to evaluate if the apparent bedrock high is abrupt or gently sloped, and if the groundwater gradient was just as abrupt or steeply sloping.

Borings DD-7 through DD-12 were drilled to depths of approximately 14 and 30 feet below the planned tunnel invert (Plate 4). They encountered granular alluvial deposits overlying dark gray siltstone and claystone of the Pliocene Fernando and Puente formations. The location of the deposits in the former floodplain of the Los Angeles River suggest that they are mostly Holocene in age (about 10,000 years or younger), with only the base of possible late Pleistocene age. Similar conditions were encountered by an alignment of borings recently drilled along Mission Road across the escarpment projection directly east of the Los Angeles River channel (GeoTransit Consultants, 1996). The Los Angeles River Channel was scoured to its greatest depth during the latest glacial period, which ended about 15,000 to 18,000 years ago.

Subsurface data from these borings and all previous explorations were incorporated in the profile presented in Plate 4. It is apparent from the profile that the bedrock high is located in the vicinity of Borings DD-7, DD-8, PE-27 and PE-28. In the vicinity of Boring PE-28, the bedrock surface is at an elevation of about 225 feet, or 45 feet BGS. From that area, the bedrock surface gradually drops to an elevation of 220 feet, or 50 feet BGS at Boring DD-8, and to an elevation of 207 feet, or 60 feet BGS at Boring DD-9. South of Boring DD-9, the bedrock surface drops with some abruptness to an elevation below 175 feet and greater than 92.5 feet (maximum depth of boring) BGS at Boring DD-10. This represents a change in the bedrock surface elevation of more than 32 feet over a length of about 160 feet along the alignment. At

Boring DD-10, only alluvium was encountered to the explored depth of 92.5 feet. The bedrock surface was not encountered by the geotechnical borings drilled south of Boring DD-10. It should be noted that the horizontal projection of the Coyote Pass escarpment lies between Borings DD-9 and DD-10 (Plate 4) in the vicinity of the First Street bridge crossing of Santa Fe Avenue.

Observations made during drilling and subsequent measurements of groundwater levels in piezometers suggest that the bedrock high is causing groundwater to pond in the floodplain north of the bedrock high. The groundwater surface has an elevation of about 240 feet, or about 40 feet above the bedrock surface, north of the bedrock high. To the south, the elevation of the groundwater surface ranges from approximately 187 feet to 193 feet. Between these two areas, the groundwater level varies in elevation from approximately 229 feet at Boring DD-7 to approximately 211 feet at Boring DD-9. These elevations are within 10 to 15 feet of the bedrock surface. Based on these observations, the change in groundwater level across the bedrock high is not abrupt but, in general, conforms with the slope of the bedrock surface.

4.3 SUBSURFACE STRATIGRAPHY AND GROUNDWATER CONDITIONS

4.3.1 Subsurface Stratigraphy

Based on the subsurface information obtained from this investigation and previous investigations in the project area, Plates 4 and 5 present the following two plans and generalized subsurface cross-sectional profiles showing the subsurface stratigraphy and tunnel and station profiles in the project area.

- Along the CR track from the Union Station to the vicinity of the southern terminus (Station CR 46 + 56) of the Little Tokyo Station.

- Along the CL track from the Union Station to the vicinity of East First Street (approximate Station CL 40+00) where the south heading CL and CR tracks become parallel.

The subsurface stratigraphy along the proposed tunnel alignments and in the vicinity of the Station consists of shallow surficial fills (for pavement and structure subgrade) overlying alluvium and Fernando/Puente Formation bedrock. The bedrock was encountered in borings along portions of the Union-Little Tokyo Tunnels and was not encountered in any of the borings drilled in the vicinity of the Little Tokyo station.

At the boring locations, the fill underlying existing roadway pavements varies up to about 5 feet in thickness and consists predominantly of base course, and subbase materials including gravel, sand and silty sand. Locally thicker layers of fill to 15 feet or more may exist, especially in the Busway and US101 Freeway area near the Union Station, and in the vicinity of underground utilities (gas, water, electricity, telephone, oil pipelines, etc.).

The alluvium below the fill is heterogeneous. Within the depths of exploration the alluvium is predominantly granular and consists of loose to very dense gravel (with and without sand and/or silt), sands (with and without gravel and/or silt) and silty sand (with and without gravel), and firm to hard sandy silt with cobbles and boulders. Unified Soil Classification System (USCS) classifications of granular alluvium encountered along this portion of the alignment include GP, GW, GW-GM, GP-GM, GM, SW, SP, SW-SM, SP-SM, SC-SM, SM and SC. The predominantly coarse grained alluvium is occasionally interlayered with fine-grained alluvium consisting of silt, lean clay and silty clay with sand, especially near the southern portion of the Little Tokyo Station (i.e., in the vicinity of Borings SD-4 and SD-5). The equivalent standard penetration test (SPT) blowcounts in the alluvium range from 3 per foot to values in excess of 100 per foot. Loose alluvium with SPT blowcounts of less than 10 per foot are generally located less than 10 to 20 feet BGS. The SPT blowcounts for gravel and sand/silt with gravel are generally high (in excess of 100). These high SPT blowcounts are indicative of the presence of gravel and/or cobbles and may not reflect the density/consistency of these soils. Based on interpretation of Becker Hammer blowcounts in SD-2, DD-2 and DD-6, the cobbly and gravelly soils have equivalent SPT blowcounts ranging from 30 to 100 with predominant equivalent SPT values more than 50. Thus the consistency of the gravelly cobbly alluvium ranges from dense to predominantly very dense.

As summarized in Table 3-8, cobbles (3 to 12 inches in size) were encountered in most of the borings drilled during the current investigation as evidenced by the samples from the bucket auger boring (DD-5) and a combination of factors including rock fragments in cuttings, zero or low sample recovery and drill rig chatter. Cobbles were also encountered in the preliminary investigation program (Borings PE-18 and PE-28 through PE-31, GeoTransit Consultants, 1994a). Within the project area the presence of boulders estimated to range in size, from 12 inches to up to 3 feet was indicated in Borings DD-3, DD-3D, DD-4 and DD-10 drilled in this investigation, and PE-28 and PE-29 drilled in the preliminary investigation. The cobbles and boulders are primarily composed of very hard to extremely hard granitic and metamorphic rock types that are weathered and durable. The possibility of encountering larger boulders up to 4 feet or more exists due to the proximity of the project site to the Los Angeles River channel where 4-foot diameter boulders were observed at the surface prior to lining the Los Angeles River at the Macy Street (Cesar Chavez Avenue) crossing as reported by Converse Consultants, Inc./Earth Sciences Associates/Geo Resource Consultants (CCI/ESA/GRC, 1981).

A significant portion of the granular layers within the tunnel zone and above the station invert are susceptible to raveling and running/flowing conditions during tunneling and station excavation. A number of incidents of caving and loss of circulation were observed in Borings DD-3D, DD-5, DD-6, and SD-3 of this investigation (Table 3-8) and in various borings drilled in previous non-project specific investigations summarized in Table 2-1.

The bedrock of the Puente/Fernando Formation in the project area consists predominantly of weak, slightly weathered to fresh, thinly laminated to massive siltstone and claystone interbedded with occasional hard, well-cemented calcareous nodules and beds, and conglomeratic sandstone layers. As described in Section 4.2.1.2, hard, cemented siltstone beds (each up to ½ inch thick) were encountered in zones up to 5 feet and 1 foot thick in Boring PE-28 drilled in the preliminary investigation program and Boring DD-7 drilled for this investigation, respectively. Based on available as-built geologic logging for the nearby City Terrace trunk sewer tunnel in the Puente/Fernando Formation, these occasional well-cemented zones were less than two percent of the strata that were tunneled and ranged in size from 20-inch nodules to lenses and beds up to 12 feet thick.

Except for the local presence of well cemented zones, the Puente/Fernando Formation bedrock is expected to behave in a manner similar to that of hard and dense soil. The unconfined compressive strengths of this bedrock obtained from laboratory tests on Pitcher barrel samples (Table 3-5) range from about 9440 psf (65.5 psi) to about 20,300 psf (141 psi). Due to material variability and potential sample disturbance, actual in situ maximum strength of the Puente/Fernando Formation bedrock is anticipated to be higher.

4.3.2 Groundwater Levels

Groundwater levels in the project area were monitored in 14 piezometers/monitoring wells including 10 installed during this investigation and four (PE-18 and PE-29 through PE-31) installed during the preliminary investigation (GeoTransit Consultants, 1994a). The observed groundwater levels are summarized in Table 3-2. The most recent groundwater level readings are also shown in Plates 4 and 5. Other available groundwater level data and time of readings reported in other investigations (GeoTransit Consultants 1994b; CCI/ESA/GRC, 1981 and 1984, and Earth Technology, 1987) are also shown in these plates.

In general, the current groundwater levels are approximately 30 to 40 feet BGS from the Union Station area to the vicinity of DD-4 and DD-7 near Banning Street where a shallow buried bedrock ridge (bedrock high) was encountered. South of Banning Street the groundwater table dips south with an average gradient of about 5 percent to about 80 feet BGS at the northern terminus of the Little Tokyo Station. At the Little Tokyo Station the groundwater levels are relatively flat and are about 78 to 80 feet BGS.

Significant differences exist in groundwater levels between the data recorded during this and previous project-specific investigations (GeoTransit Consultant, 1994a and 1994c), and those recorded in 1983 by CCI/ESA/GRC(1984), especially in the Little Tokyo Station area. The measured groundwater levels in 1983 were higher than those measured in the current investigation and the 1994 preliminary investigation (GeoTransit Consultants, 1994a) by the following amounts:

- About 55 feet higher in the Little Tokyo Station area
- About 20 feet to 55 feet higher between Banning Street and Little Tokyo Station
- About 0 to 20 feet higher between the Union Station and Banning Street.

We have searched the water well records in the general area. There exist records of three water wells (2765, 2775 and 2809 shown in Table 2-1 and Plate 1) in the vicinity of the project area that are maintained by the Los Angeles County Department of Public Works. As shown in Table 2-1, the water levels in these wells show the following fluctuations:

Well No.	Water Level Depth (Date)	Fluctuation in Water Level Depth (Feet)
2765	90 ft. (10/38) 113.6 ft (4/72)	26.6 ft
2775	40.2 ft (11/34) 70.8 (6/40)	30.6 ft
2809	126.6 ft (3/35) 286.2 (4/60)	159 ft

The above records and the observed groundwater level fluctuations in the project area indicate that the groundwater levels in the project area are time-dependent and can significantly fluctuate. Potential fluctuation in groundwater levels should be considered in the design and construction of the tunnel and station facilities in the project area.

The presence of occasional less permeable clayey sand, silt, silty clay and lean clay interbeds within the coarse alluvium indicate potential existence of local perched groundwater zones in the project area. This is further evidenced by the floating hydrocarbon products encountered between about 26 and 34.5 feet BGS in Boring SD-2 and between about 39 and 62 feet BGS in Boring DD-2.

4.3.3 Soil and Groundwater Contamination

An assessment of the soil and groundwater contamination for the project area has been presented in the Stage II site assessment report for the Eastside Extension prepared by GeoTransit Consultants (1994). The results of the Stage II site assessment together with the data obtained from the current investigation and available data from other previous investigations were utilized in evaluating the potential for soil and groundwater contamination.

4.3.3.1 Data from the Environmental Stage II Site Assessment Investigation

Appendix D presents summaries of data obtained during the Environmental Stage II site assessment investigation for the project area. These summaries are presented in tabulated formats to describe the field exploration program, significant observations, and analytical test program and results on selected soil, groundwater and gas samples.

4.3.3.2 Data from Current Investigation

The scope of environmental monitoring and testing performed in this investigation included screening soil samples with the OVA for the potential presence of volatile organic compounds (VOCs), monitoring selected groundwater samples for hydrogen sulfide (H_2S) using a multiple gas indicator, and chemical testing of selected soil and groundwater samples. The results of chemical testing and H_2S monitoring are presented in Tables 3-6, 3-7, and 3-8. 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 also summarized in Table 3-8.

4.3.3.3 Groundwater Contamination

Groundwater samples collected from monitoring wells DD-1, DD-2, and DD-3C from this investigation (Table 3-6) and PE-29, PE-30, and PE-31 from our previous investigation (Appendix E) are contaminated with hydrocarbons above California Department of Health Services (CDHS), Maximum Contaminant Levels (MCLs) for drinking water and other published threshold concentrations. As can be concluded from Table 3-6, the hydrocarbon contaminants in groundwater that require treatment prior to disposal include, but are not limited to total petroleum hydrocarbons (TPH), volatile organic compounds (VOCs including cis-1,2-dichloroethene, trans-1,2-dichloroethene, benzene, toluene, ethylbenzene, vinyl chloride etc.), and a number of semi-volatile organic components (acenaphthene, naphthalene, etc).

Groundwater samples also contain several constituents of concern including hydrogen sulfide, sulfide, sulfate and chloride. Total sulfide was found to exceed 50 mg/l in groundwater samples collected from DD-1, DD-3C, PE-29 and PE-30. Sulfate and/or chloride contents exceed the secondary MCL of 250 mg/l in groundwater samples from DD-1, DD-2, DD-3C, PE-29 and PE-31.

The detected total sulfides in groundwater samples are indicative of the presence of dissolved H_2S in groundwater. Sulfur odor was noticed during the development of monitoring wells PE-29, PE-30, PE-31, EB-22, DD-1, DD-2 and DD-3. Prior to well development, the airspace above groundwater registered H_2S concentrations of 46, 2.9, 11.5 and 64 ppm at Wells PE-29, PE-30, PE-31 and DD-1, respectively. During well development/groundwater sampling, maximum H_2S concentrations of 150, 1,012 and 193 ppm were measured in the well casing near well heads at Wells EB-22/, DD-1 and DD-2, respectively. These measurements provide further evidence of the release of H_2S from groundwater in the area.

As shown in Table 3-6, groundwater contamination appears to be transient in nature and varies from location to location as evidenced by the contamination levels of most constituents in groundwater samples from Pump Well DD-3C at the end of well development (Sample DD-3C) being generally much less than the groundwater samples from the same well obtained approximately midway and at the end of the aquifer pump testing (Samples DD-3C-1 and DD-3C-2 in Table 3-6). This indicates that more contaminated groundwater from the vicinity of the pump well was drawn into the pump well during aquifer pump testing. Thus, it can be anticipated that groundwater contamination in the project area will change with time and location.

No evidence of hydrocarbons or H_2S contamination was found in groundwater samples from Boring/Monitoring Well PE-18 in the Little Tokyo Station area. Thus, groundwater contamination in the project area is likely limited to portions of the tunnel alignment from the Union Station to somewhere north of the Little Tokyo Station.

Based on current groundwater levels and the planned tunnel profiles, the portion of the Union-Little Tokyo Tunnels between the Union Station and the vicinity of Boring DD-10 will be below groundwater. Groundwater from dewatering will require treatment before disposal. The treatment method that was successfully used for the disposal of pumped water from the aquifer pump test performed in this investigation included the following basic steps:

- Addition of chemicals to raise the pH to 9.5
- Addition of hydrogen peroxide to oxidize the sulfide in water to sulfate
- Addition of chemicals to lower the pH to 4 to dissolve scale
- Filtering the water through granular activated carbon to remove the hydrocarbon compounds,
- Addition of chemicals to raise the pH to above 6.1.

4.3.3.4 Soil Contamination

The chemical test results of soil samples from nine borings performed in the Stage II site assessment investigations (GeoTransit Consultants, 1994b) are shown in Appendix D. This data together with available field observation data from this investigation and other site-specific investigations (Geotransit Consultants, 1994a and 1994c) were utilized in assessing soil contamination in the project area.

Based on available data, the following evidence of soil contamination within the project are noted:

- Flame Ionization Detector (FID) and Photo Ionization Detector (PID) readings of more than 100 ppm above background level were obtained mostly in the headspace of soil samples near or below groundwater.

- Except for localized TPH contamination above groundwater, most of the detected hydrocarbon contamination was in soil samples near or below the groundwater level. Soil contamination that exceed threshold level are summarized in Appendix D. The detected concentrations of contaminants are, in general, below the threshold levels except in fine grained materials (fine-grained alluvium and Fernando/Puente Formation bedrock materials). One possible explanation for this observation is that the predominantly coarse-grained alluvial soils within the project area have not retained significant amounts of contaminants from the groundwater. However, due to their high surface adsorption capacity, portions of the bedrock (especially siltstone and claystone) and locally present fine-grained alluvium in the project area have absorbed and contain TPH, VOCs, and SVOCs above the threshold levels.

- The above field observations and laboratory test results all seem to indicate that most soil contamination in the project area is probably related to the groundwater contamination described in the previous section.

- Most of the localized soil contamination is likely related to (1) crude oil contamination (as evidenced by the oil and tar like substance described in Table 2-2 related to the Union Station Oil field and (2) leakage from existing or abandoned oil pipelines in the project area as evidenced by the detection of free product between 26 and 32 feet BGS in Boring SD-2.

Moisture contained in subsurface soils, especially below or near the groundwater table, will be a likely source for H_2S gas as evidenced by sulfur odors and detected concentrations of total sulfides in the soil samples. In addition to being potentially present above the groundwater table, H_2S may also be present within the previously saturated zones that become unsaturated upon dewatering. Release of H_2S from soils could occur during construction operations associated with tunneling/excavation and muck handling.

4.3.3.5 Gassy Conditions

A Portion of the project area is located within known boundaries of the Union Station oil field. The potential accumulation of methane and other gases within oil fields in Los Angeles Basin is well known.

The potential for gassy conditions within the project area was investigated by sampling and testing of gas samples from Monitoring Wells PE-29, PE-30 and PE-31, and Nested Wells EB-22/1 and EB-22/2 (screened below and above the groundwater table, respectively); and field observations during drilling, well development and groundwater sampling, and gas sampling. Laboratory test results on gas samples are summarized in Appendix D.

Based on the field observations and laboratory test results, the following conclusions can be made:

1. High methane concentrations were detected in gas samples collected from the tops of the well casings of Monitoring Wells PE-29 (55,000 to 110,000 ppmv), PE-30 (26,000 to 360,000 ppmv) and EB-22/1 (20,000 to 720,000 ppmv) while insignificant amounts of methane were present in gas samples collected from the tops of the well casings of Monitoring Wells PE-31 (57 ppmv) and EB-22/2 (8.2 ppmv). These results and available field observation data (Appendix D) appear to indicate the following:

- Methane is lighter than air and hence will rise to the top of the casing in monitoring wells. High concentrations of methane were detected in gas samples collected from the tops of the closed monitoring well casings.

- Varying amounts of methane are likely to be present in the area between Union Station and north of Little Tokyo Station. Methane could potentially exceed its LEL (55,000 ppmv), in small confined spaces.

- The difference in methane concentration levels between EB-22/1 (screened below the groundwater table) and EB-22/2 (screened above the groundwater table) indicate that methane is released from or through groundwater. This is also evidenced by the field observations that hydrocarbon odors and high FID and PID readings are mostly encountered in soil samples below the groundwater table.

2. Laboratory tests on gas samples collected from Well EB-22/1 indicated H_2S concentrations of 10,600 to 19,000 ppmv while H_2S was not detected in gas samples from Well EB-22/2. These observations suggest that H_2S is also released from or through groundwater. As described in Section 4.3.3.4, the observed high H_2S concentrations during development of Monitoring Wells EB-22/1, DD-1 and DD-2 provide further evidence to support this finding.

3. All sulfurous odors were detected in soil samples (headspace readings) near or below groundwater levels in Wells PE-29 through PE-31. This indicates that H_2S released from groundwater may be either immediately dissipated through the coarse granular alluvium in the area or stays in the vadose zone because H_2S is heavier than air and methane. On the other hand, all the gases present in a closed space, such as the well casing of EB-22/1, will come to equilibrium concentrations due to the diffusion process, given sufficient amount of time. It is postulated that the maximum detected concentrations of methane (720,000 ppmv) and H_2S (19,000 ppmv) in the gas sample collected on February 1, 1994, in Well EB-22/1, is probably near the equilibrium concentrations for the volume of casing above the water in that well (approximately 0.75 cu. ft.).

4. The results of a limited isotopic analysis (GeoTransit Consultants, 1994b) indicate that methane in the project area may be biogenic. Isotope ratios show that the methane is generated by bacteria

through the reduction of CO_2 with hydrogen gases. H_2S may be diffused up from a deeper aquifer or from some deep horizon where it has been trapped from some previous generation event.

5. The above findings indicate a high potential for accumulation of high concentrations of methane and H_2S during tunnel construction between the Union Station and north of Little Tokyo Station, especially between the Union Station and the vicinity of Boring DD-10 where groundwater exists within the proposed tunnel envelope. However, the rate and amount of accumulation of these gases in a tunnel (larger volume) will be significantly less than the rate and amount of accumulation observed in the well casings (smaller volume). Proper ventilation and monitoring of methane and H_2S will be necessary to conform to the U.S. Occupational Safety and Health Administration (OSHA) requirements and to provide a safe working environment.

6. Accumulation of other VOCs such as benzene and vinyl chloride at concentrations near the allowable industrial exposure limits is also possible in tunnels, as evidenced by the compounds detected (benzene, 2.8 ppmv; vinyl chloride, 2.3 ppmv) in the gas sample from Nested Well EB-22/1. OSHA Short Term Exposure Limit (STEL, a 15-minute time weighted average exposure limit) for benzene is 5 ppmv. The OSHA ceiling exposure limit for vinyl chloride is 5 ppmv.

4.4 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

The engineering properties of subsurface materials, as evaluated from results of field and laboratory tests, are summarized in Table 3-5. Blowcount data from drive sampling and standard penetration tests ((SPT N-Values) are shown in the borehole logs and presented in Table 3-5.

Table 4-2 presents a summary of the measured/interpreted ranges of relevant geotechnical parameters for the various materials types encountered within the project area. For purposes of presentation, the alluvium has been broadly categorized into fine-grained and coarse-grained alluvium. The alluvium within the project area is predominantly coarse-grained. The high blowcounts recorded within the course grained alluvium 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. However, as described in Section 4.3.1, the gravelly/cobbly alluvium is generally dense to very dense in the project area based on an interpretation of the results of three Becker hammer borings (DD-2, DD-6 and SD-2).

No engineering properties are presented for the locally present surficial fill which is considered to have little or no effect on the station design. Station and tunnel excavation and construction will be primarily within the alluvium and bedrock. Based on available geotechnical data and engineering evaluation, the static and dynamic engineering properties for the alluvium and Fernando/Puente Formation bedrock are summarized in Tables 4-2 and 4-3. The engineering properties are presented and used in the engineering analysis presented in Section 5-0.

The following section provides a description of relevant engineering properties of the subsurface materials in the project area.

4.4.1 Alluvium

4.4.1.1 Grain Size Distribution

The alluvium in the tunnel envelope and station excavation area is heterogeneous and is predominantly coarse-grained. Results of grain size distribution and fines content (percentage passing #200 sieve) tests are summarized in Table 3-5. The bulk of the gradation and fines content tests were performed on selected granular samples in the vicinity of the tunnel envelope and above the Station invert elevation in station borings. This was done primarily to evaluate areas of cohesionless sands and gravels which may be susceptible to raveling/running/flowing conditions. It should be noted that in layers classified as gravel, clayey gravel, silty gravel, gravel with sand and sand with gravel, because of small diameter boreholes and limited sample size, some of the laboratory gradation curves presented may not be truly representative of the entire deposit and may only reflect gradations of the finer matrix materials.

Results of gradation tests and field observations show the presence of significant zones of granular alluvium with low fines content (poorly to well graded sands and gravels) within tunnel and station excavations. Such zones exist within a major portion of the tunnel and station area.

TABLE 4.2. SUMMARY OF ESTIMATED ENGINEERING PROPERTIES

ENGINEERING CHARACTERISTICS	FINE-GRAINED ALLUVIUM		COARSE-GRAINED ALLUVIUM		Fernando/Puente Formation Bedrock	
USCS Classification	CL,CH,ML,MH CL-ML		SP,SW,GP,GW,SM,SC GP-GM,GW-GP SP-SM,SW-SM,SC-SM		ML,MH,CL,CH	
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
Equivalent SPT Blow Counts	26-95	-	2->100	-	22->100	-
Insitu Moisture Content (%)	16-33	21	2-26	11	19-32	25
Void Ratio	0.48-0.91	0.62	0.26-0.76	0.42	0.58-0.92	0.62
Insitu Dry Density (pcf)	89-115	105	93-130	115	92-108	105
Fines Content(% passing #200 Sieve) (%)	50-79	-	1-46	-	84-98	-
Specific Gravity	2.72	2.72	2.61-2.63	2.62	2.72	2.72
Liquid Limit (%)	27-37	35	-	-	29-69	40
Plasticity Index (%)	5-17	12	-	-	1-31	15
Peak Shear Strength: Cohesion (psf)	600-1000	600	0-1000	0	700-5900	5000
Friction Angle, (degrees)	32-35	32	35-45	35	25-34	25
Uniaxial Compressive Strength (psf)	-	-	-	-	4900-20300	12000
Poissons Ratio	0.3-0.45	0.4	0.3-0.4	0.35	0.4-0.49	0.49
Young's Modulus (10^3 psf)						
0-10'	-	-	210-680	300	-	-
10-40'	-	-	400-1350	600	-	-
Below 40'	-	-	1400-4200	2000	1500-2700	1800
pH	7.74-8.25	-	3.6-8.11	-	6.45-7.71	-
Chloride Content (ppm)	109-400	-	68-1085	-	119-890	-
Sulphate Content (ppm)	73-170	-	52-1645	-	86-940	-
Electrical Resistivity (ohms-cm)	1165-1538(2857)	-	625-5556	-	1256-7692	-
Compression Index- Cc	0.05-0.07	0.07	0.034-0.07	0.04	0.063-0.092	0.09
Swelling Index-Cs	0.008-0.011	0.01	0.004-0.01	0.005	0.012-0.022	0.015
Rate of Secondary Compression-Cx	0.00025-0.0013	0.001	0.00035-0.0014	0.0004	0.0008-0.0009	0.0008
Swelling (+)/Collapse (-), %	+0.2-(0.16)	-	-0.01-(-0.04)	-	0-(+0.82)	-
Slake Durability Index (%)	-	-	-	-	1.1-17.5	-

NOTES:
1. Equivalent SPT Blow Counts in alluvium may not be representative of material density/consistency due to the presence of gravels,cobbles and boulders.
2. 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.
3. Shear strength parameters (cohesion and friction angle) were obtained by direct shear tests and triaxial compression tests.
4. Electrical resistivity tests, in general were conducted at in situ moisture content. The value of paranthesis () correspond to saturated condition of the samples.
5. Cc and Cs are based on vertical strain-log stress plots. Cx is based on vertical strain-log time plot.

TABLE 4.3. ENGINEERING PROPERTIES FOR DYNAMIC ANALYSIS

MATERIAL PROPERTY	ALLUVIUM						Fernando/Puante Formation Bedrock	
	0 to 10 feet		10 to 40 feet		Below 40 Feet			
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
Shear Wave Velocity (ft/sec)	450-800	550	600-1100	750	1100-2000	1300	1100-1500	1250
Poissons Ratio	-	0.3	-	0.35	-	0.4	-	0.49
Shear Modulus [1] (10^3 ksf)	0.8-2.6	1.2	1.5-5	2.2	5 to 15	7	5-9	6
Youngs Modulus [1] (10^3 ksf)	2.1-6.8	3	4-13.5	6	14-42	20	15-27	18
Damping	See Note (2)							

Notes:

[1] Values correspond to small strains (shear strain <= 0.001%).
Apply the reduction factor for other strain values:

Shear Strain (%)	Reduction Factor
1.0E-02	0.65
1.0E-01	0.2
1.0E+00	0.05

[2] Recommended damping value is strain-dependent as follows:

Shear Strain (%)	Damping (%)
2.0E-03	5
1.0E-02	10
5.0E-02	15
2.0E-02	20

4.4.1.2 In Situ Conditions and Index Properties

Laboratory test data indicate that the dry density of the alluvium ranges from 90 to 131 pcf, and the in situ moisture content ranges from 2 to 33 percent. The relatively wide ranges of density and moisture content values and the large number of soil types encountered in the alluvium are indicative of the nonuniforn and heterogeneous nature of the alluvium in the project area.

Atterberg limit tests on the limited fine-grained alluvium encountered indicate that these materials are predominantly lean clays, silty clays and silts, with relatively low plasticity (liquid limits ranging from 27 to 35 and plasticity indices ranging from 5 to 14).

4.4.1.3 Shear Strengths

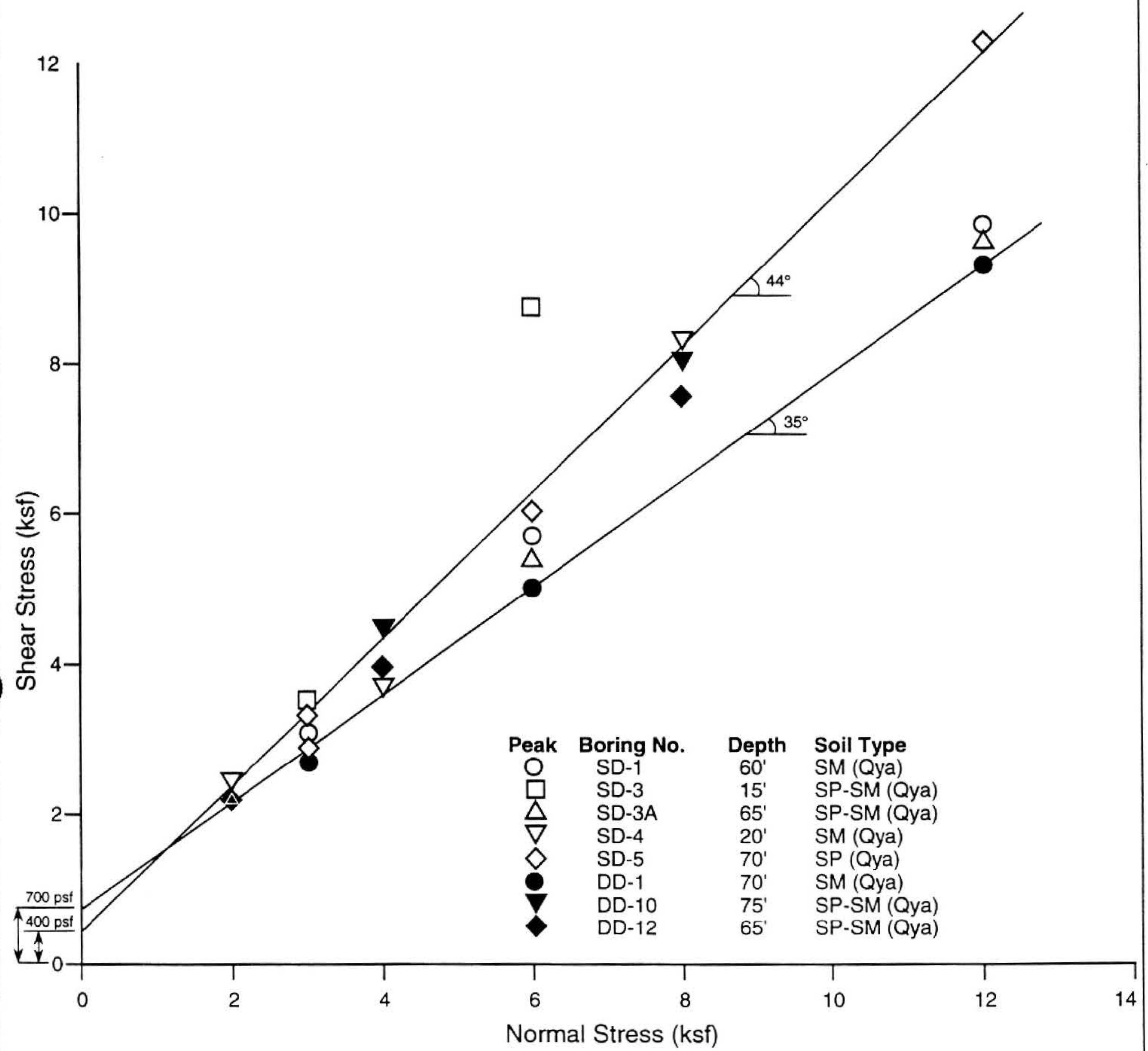
Shear strength parameters for the alluvium were derived based on the results of direct shear tests on selected samples from the current investigation and results presented in the preliminary investigation (GeoTransit Consultants, 1994a). Results of these direct shear tests are graphically presented in Figures 4-5 and 4-6.

4.4.1.4 Static and Dynamic Modulus

The dynamic modulus and Poisson's ratio for the alluvium were estimated based on results of two downhole geophysical surveys, available literature on similar materials, available correlations with SPT data and engineering judgement. Modulus of soil is strain-dependent. The modulus value determined from seismic velocity surveys correspond to small strain ranges appropriate for dynamic loading situations (shear strain about 10^{-3} percent or less). Under static loadings shear strains are expected to be considerably larger and the corresponding moduli should be reduced in accordance with the recommended reduction factors shown in Table 4-3.

4.4.1.5 Compressibility

Results of three consolidation tests on relatively undisturbed samples of relatively fine grained alluvium from the 65- to 92- foot depth range within the proposed station area are presented in Table 3-5.



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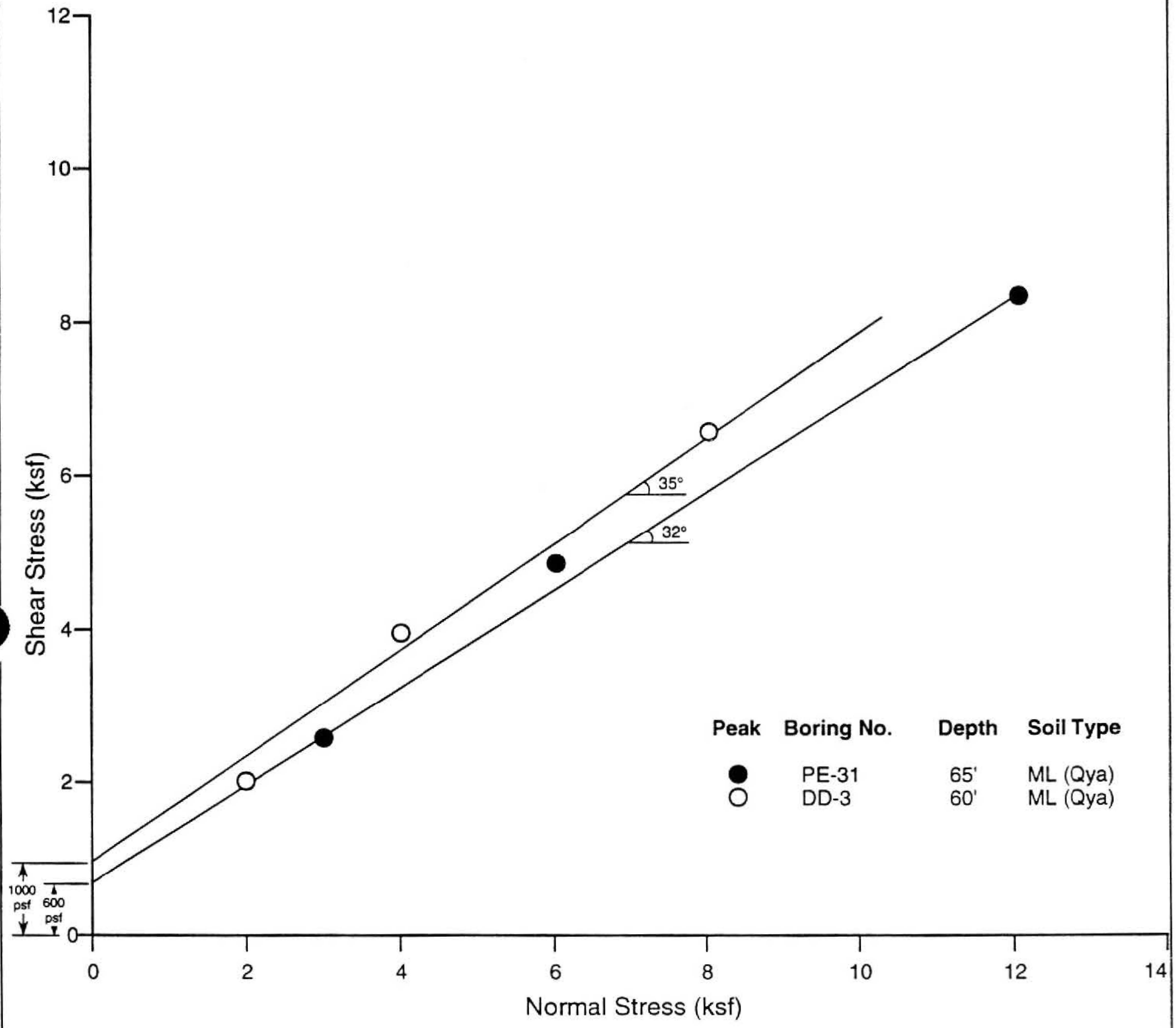
Direct Shear Test Results on Coarse-Grained Young Alluvium

10-95

Figure 4-5



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Eastside Extension
Metro Red Line

Direct Shear Test Results on Fine Grained Young Alluvium

10-95

Figure 4-6

4.4.1.6 Corrosion Potential

Results of soluble sulfate content tests (52 to 1645 ppm) summarized in Table 4-2 indicate that alluvial materials are mildly to moderately corrosive to concrete. Results of electrical resistivity tests (625 to 5556 ohm-cm) indicates that these martials are moderately corrosive to very corrosive to metals.

4.4.2 Fernando/Puente Formation Bedrock

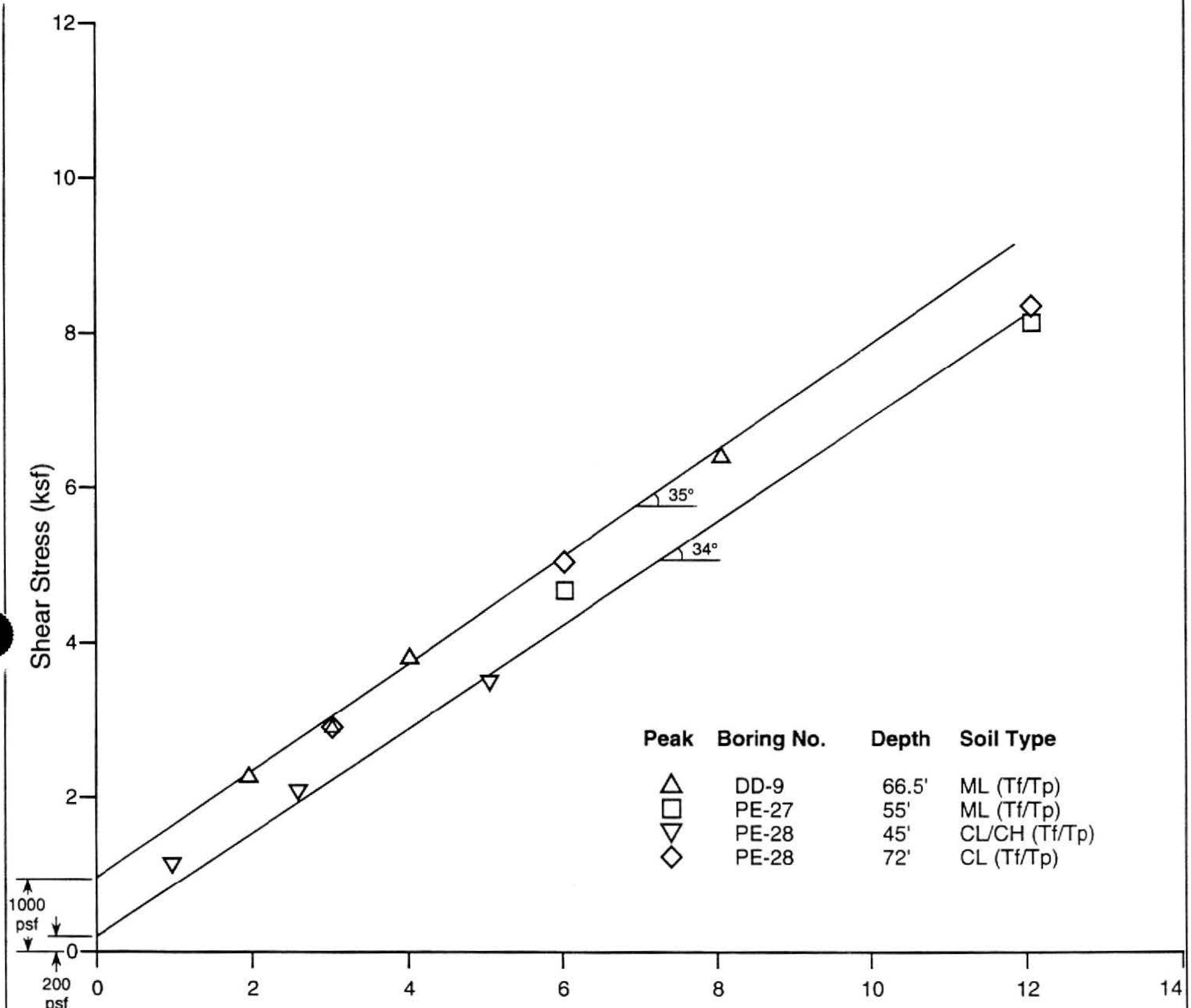
4.4.2.1 Index Properties

The Fernando/Puente Formation bedrock in the project area consists predominantly of very soft to soft (based on Bureau of Reclamation, Engineering Geolon Manual Classification) claystone, clayey siltstone and siltstone. Various engineering properties from field and laboratory tests on selected bedrock samples are summarized in Table 4-2.

In situ dry density of the bedrock ranges from 89 to 108 pcf, and in situ moisture content ranges from 19 to 32 percent. The fine grained soils derived from the bedrock are predominantly silts, and lean (medium to low plasticity) and fat (high plasticity) clays with liquid limits ranging from 29 to 69 and plasticity indices ranging from 1 to 31.

4.4.2.2 Shear Strength Characteristics

Shear strength characteristics of bedrock materials are based on the results of uniaxial compression tests, direct shear tests, and consolidated undrained triaxial compression tests with pore pressure measurements on selected bedrock samples (Tables 3-5 and 4-2). The range of uniaxial compressive strength of the bedrock is provided in Table 4-2. Results of direct shear tests and triaxial compression tests are presented in Figures 4-7 and 4-8, respectively.



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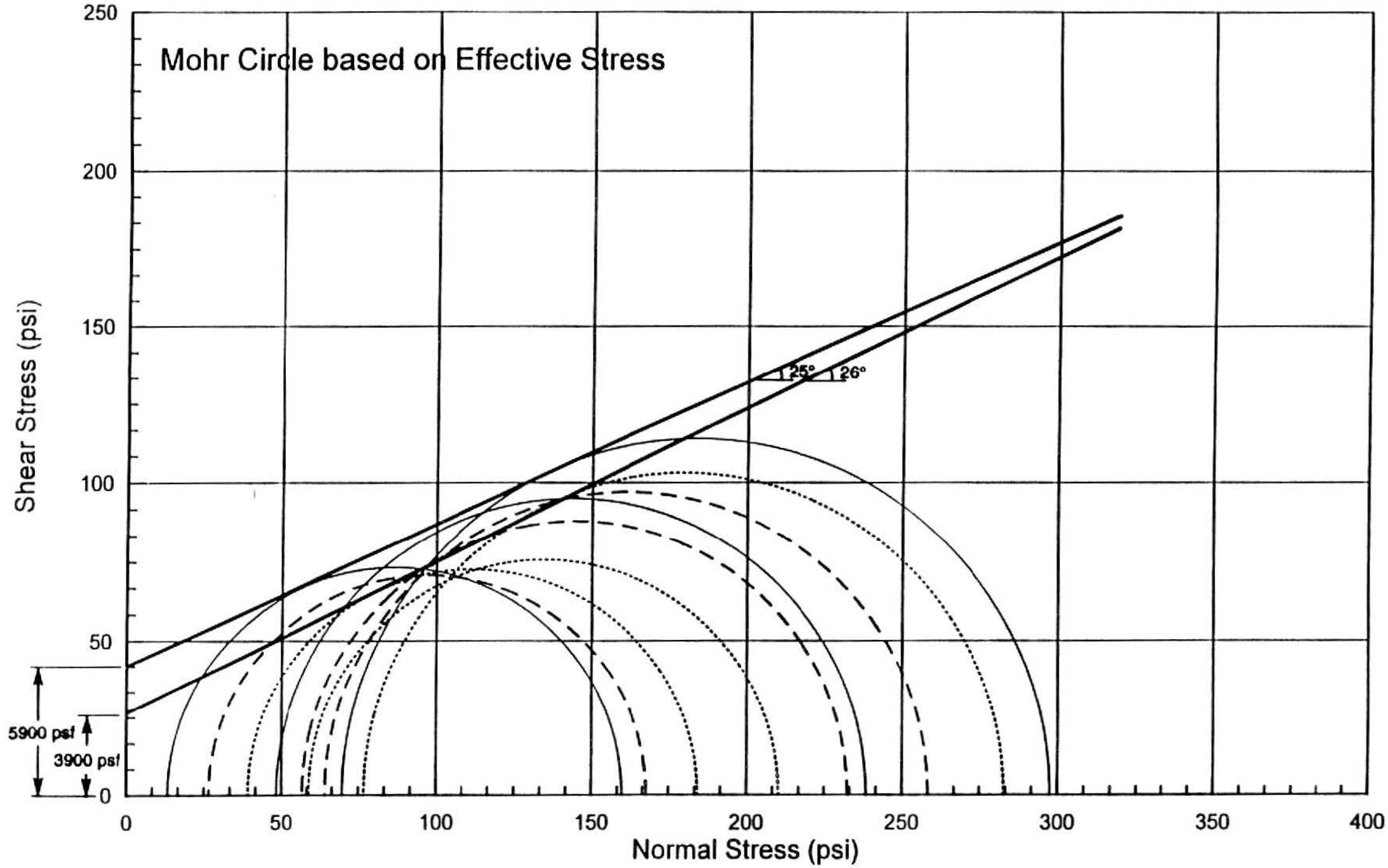
Metro Red Line

Direct Shear Test Results on Fernando/Puente Bedrock

10-95

Figure 4-7



Boring No.	Sample No.	Depth (ft.)	Eff. Conf. Pressure (psi)
DD-4	T-24	87.5	20,40,80
DD-7	T-20	81.0	20,40,80
DD-9	T-19	72.0	20,40,80

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Geotechnical Investigation
Eastside Extension
Metro Red Line

Triaxial Compression Test Results on Fernando/Puente Bedrock

10-95

Figure 4-8

4.4.2.3 Static and Dynamic Modulus Characteristics

Dynamic modulus characteristics of the Fernando/Puente Formation bedrock were estimated based on interpretation of the results of the downhole seismic velocity surveys and engineering judgement. Results are presented in Table 4-3. Static modulus characteristics of the Fernando/Puente Formation bedrock were estimated by applying a strain dependent reduction factor on the dynamic modulus.

4.4.2.4 Slake Durability

Two slake durability tests were performed on selected Fernando/Puente Formation bedrock samples from Borings DD-4 and DD-7. The slake durability index (second cycles) were found to be 1.1 and 17.5 percent, indicating high susceptibility to slaking. Thus, the Fernando/Puente Formation bedrock has a high potential to slake and ravel if subject to moisture changes due to prolonged exposure.

5.0 DESIGN AND CONSTRUCTION

5.1 GENERAL

This section provides a description of geotechnical evaluations and recommendations and key geotechnical issues for the design and construction of the Union-Little Tokyo Tunnels and the Little Tokyo Station.

5.2 SUMMARY OF SUBSURFACE STRATIGRAPHY

Based on current plans and profiles, the Union-Little Tokyo Tunnels will be within alluvium and the Fernando/Puente Formation bedrock, while the Little Tokyo Station will be located predominantly in the alluvium. The alluvium is heterogeneous and non-uniform and consists predominantly of gravels, gravelly sands, sands and silts with local cobbles and boulders and occasional and localized layers of clayey sand, lean clay and clayey silt. Boulders up to 4 feet in size have been encountered in this area. The granular alluvium over a large portion of the project area consists of sands and gravels with low fines content. The boulders and cobbles encountered are typically very hard to extremely hard unweathered granitic and metamorphic rock types. Within the tunnel depths, the bedrock materials, when encountered, are expected to consist predominantly of very low-strength siltstone, claystone and sandstone, except for local zones of hard, well-cemented calcareous interbeds with an estimated maximum thickness on the order of about 5 feet. Except for such local hard and well-cemented interbeds and nodules, the Fernando/Puente Formation bedrock, for tunneling purposes, is expected to behave in a similar manner as hard and dense soils.

5.3 GROUNDWATER LEVEL FLUCTUATIONS

Groundwater levels are an important input for assessing dewatering needs during construction, and to determine appropriate hydrostatic pressure on the design of tunnel and station structures. As described in Section 4.3.2. the groundwater levels in the project area appear to fluctuate significantly as evidenced by the most recent measured groundwater levels being up to 55 feet lower than those indicated in a 1983 investigation by CCI/ESA/GRC (1984). This significant groundwater fluctuation indicates that it is difficult to predict the groundwater levels during construction or the maximum groundwater levels that the station structures may experience during their design life. For design and construction purposes, the following groundwater levels were assumed:

- Groundwater levels in the project area before and during tunnel and station construction will be the same as the most recent measured groundwater levels obtained from this investigation.

- The design groundwater level corresponding to the maximum sustained groundwater level over the design life of the structure can be represented by the groundwater level data measured in the 1983 investigation.

It is recommended that groundwater levels be monitored prior to and during the construction. The geotechnical evaluation and recommendations, especially those related to groundwater dewatering, should be examined and, if needed, modified to reflect the measured groundwater level at that time.

We understand that EMC applies a load factor of 1.7 on groundwater pressures in tunnel and station design in accordance with the current ACI code requirements, to account for potential future groundwater fluctuations. The maximum design groundwater level recommended above is based on actual groundwater levels measured in 1983, and represents a reasonable estimate. The potential exists for groundwater levels to periodically rise higher than the recommended design value. To account for this possibility and at the same time avoid over conservatism in design, the following are recommended:

1. Use the design groundwater level recommended above in conjunction with the current ACI code requirements (i.e. applying a load factor of 1.7).

2. Check the design by using a much lower load factor (say 1.0) and assuming groundwater levels to be at the ground surface.

5.4 SEISMIC DESIGN CONSIDERATIONS

No project-specific seismic hazard evaluations were performed as part of this investigation. As per EMC's request, the seismic criteria established in a 1983 report titled "Seismological Investigation & Design Criteria" prepared by CCI/ESA/GRC (1983) for the Metro Rail Project, was adopted in our analyses. For geotechnical analyses and design purposes, the maximum design earthquake (MDE) with a maximum horizontal ground acceleration of 0.60g as recommended in the above report, was used. The magnitude of the corresponding design earthquake was estimated at 7.0. An earthquake corresponding to the Operating Design Earthquake (ODE) specified in the 1983 CCI/ESA/GRC report with a ground acceleration of 0.3g was assumed for estimating the earthquake loading imposed on shored excavations during construction.

There have been significant changes in our understanding of the seismicity of the area, the state of the art in seismic hazard analyses, and local code requirements since 1983. If the seismic design criteria are revised in the light of these changes, the seismic related geotechnical parameters and recommendations presented in this report should be reviewed and modified as appropriate.

5.5 TUNNEL

5.5.1 Excavation Considerations

Based on current plans and profiles, the Union-Little Tokyo Tunnels will be in alluvium except within the following approximate sections where tunnels will be partly (mixed face conditions) or entirely within the Fernando/Puente Formation:

? 29+00

- From the vicinity of Station CR 22+50 (vicinity of Boring PE-29) to the vicinity of Station CR 35 +50 (vicinity of Boring DD-9) along the CR track

- From the vicinity of Station CL 25+50 (vicinity of Boring DD-25) to the vicinity of Station CL 38+50 (vicinity of Boring DD-9) along the CL track.

Tunnel excavation considerations that would impact tunnel design and construction, tunnel face stability, tunnel excavation techniques, advance rates, and the potential for ground loss include the following:

- Mixed face conditions (between alluvium and bedrock) should be anticipated in the intervals identified above.

- Boulders up to 4 feet in size should be anticipated in alluvium within the entire project area.

- The soft bedrock is locally interbedded with hard well-cemented calcareous interbeds up to 5 feet in thickness and hard concretionary nodules up to 18 inches in size as evidenced in Boring PE-28 and DD-7.

- 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 can be anticipated under dewatered conditions or above groundwater in silty sand and clayey sand. 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 raveling and running/flowing conditions are anticipated over a major portion of the tunnel within alluvium.

5.5.2 Groundwater Control

Based on the most recent measured groundwater levels from this investigation, sections of the planned CR and CL tunnel envelopes between the Union Station and the vicinity of Station CR 36+50 (in the vicinity of Boring DD-10) will likely be fully or partially below groundwater.

Dewatering will be necessary to enhance stability, and reduce the potential for ground settlement and for inflows of water during tunnel excavation. It is anticipated that the groundwater level in the above portions of the tunnel segment will be reduced and maintained at least 5 feet below the planned tunnel invert. The Contractor will be responsible for designing, installing and operating a construction dewatering system subject to review and acceptance by the owner. Based on local practice, a possible dewatering system may consist of a series of deep wells placed along the affected tunnel segments. The results of a pump test conducted in the vicinity of Station CL 28+50 are presented in a separate report (GeoTransit Consultants, 1995).

The groundwater in the affected area is contaminated with hydrocarbons, H_2S and a number of other constituents (Section 4.0). The pumped water will require treatment before disposal. One viable treatment method/process has been described in Section 4.0.

Even in areas with construction dewatering provisions or 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 in the project area. This is further evidenced by the perched free hydrocarbon product encountered in Borings SD-2 and DD-2.

5.5.3 Liquefaction Potential

Most of the alluvium along the project alignment is granular in nature and contains variable amounts of gravel and cobbles. In order to evaluate the consistency and liquefaction susceptibility of these gravelly and cobbly layers, Becker Hammer Borings were performed at three locations along the alignment. The Becker hammer blowcount data obtained at Borings DD-2, DD-6, and SD-2 were then correlated to SPT blowcounts (Figure 3-1) based on procedures established by Harder and Seed (1986).

The liquefaction potential evaluation was conducted using the simplified procedures developed by Seed and Idriss (1982) and Seed et al (1984). Recent work by Fear and McRoberts (1995) was also considered in our evaluations. The results of our study indicate that liquefaction potential along the underwater portion of the tunnel alignment is very low and the tunnel alignment is not expected to experience any serious impacts due to earthquake-induced liquefaction of the surrounding subsurface materials.

5.6 LITTLE TOKYO/ARTS DISTRICT STATION

5.6.1 General

The planned cut-and-cover construction of the Little Tokyo Station will involve about 70 feet of excavation from the ground surface to the planned station invert (at about elevation 197 feet). The excavation will be primarily in the alluvium underlying surficial shallow fills. A detailed description of the alluvium has been presented in Section 4.3 and a summary provided in Section 5.2. The station structure will essentially be a water-tight, rigid reinforced concrete box structure bearing on the alluvium.

The primary considerations that require geotechnical engineering evaluation for design and construction of the planned station facilities include the following:

- Excavation methods
- Construction dewatering and related issues
- Excavation-related temporary shoring systems
- Foundation design and recommendations for soil, water and earthquake loading on the permanent station structures
- Evaluation of potential for earthquake-induced liquefaction and its effects on the station structure.

5.6.2 Excavation Method

Station excavation will be primarily in alluvium which is granular in nature. Based on the results of this investigation and design and construction experience under similar subsurface conditions, it is anticipated that the cut-and-cover excavation can be achieved using conventional excavations methods. However, suitable excavation equipment to handle the very dense gravelly and cobbly alluvium and potential local large boulders would be required. Most of the granular alluvium in the station area will run and ravel readily. Thus, timely application of ground support is important to prevent ground loss.

5.6.3 Dewatering and Groundwater Control

The most recent observed groundwater level data indicate that the groundwater table in the station area is about 10 feet below the planned bottom of station excavation. No preconstruction dewatering is anticipated provided the groundwater levels during station construction remain at or below current levels. The potential for significant fluctuation of the groundwater levels in the area dictates the need for continuing groundwater level monitoring before and during station construction to verify groundwater levels and to re-evaluate the dewatering needs, as appropriate.

Localized groundwater inflows can be anticipated due to the local presence of perched groundwater conditions. If these inflows are large enough, the accumulated inflow may need to be collected and pumped out of the excavation using ditches and sumps.

5.6.4 Sloped Excavation

Compared to shored excavations, sloped excavations will require more construction space and increase the volume of excavated materials. Sloped excavations can be used for the station's structural components that require shallower excavations, or can be used to reduce the height of shoring if sufficient easements can be obtained.

Temporary slopes in alluvium should be no steeper than 1½H:1V (1½ horizontal to 1 vertical). If heavy loads (stored materials, cranes, etc.) are anticipated at the top of the slopes, the slopes must be modified accordingly by taking into consideration the impact of these loads. The construction and proper maintenance of safe, stable slopes are the responsibility of the contractor. Safe, stable slopes should be based on actual construction conditions and subsurface conditions encountered during excavation.

5.6.5 Shored Excavation and Shoring Support

The planned station excavation will require shoring due to the proximity of the station 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 and lagging, 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 (if applicable), construction procedures, characteristics of nearby buildings, and local experience. Based on local practice in the Los Angeles area in subsurface geotechnical conditions similar to those encountered at the at Eastside Extension, soldier piles and timber 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. Driving of sheet piles may not be feasible due to the presence of gravel, cobbles and boulders.

5.6.5.1 Lateral Pressure

Lateral earth pressure on the shoring system depends on the type of shoring system, construction procedures, and subsurface and groundwater conditions. Based on the available results, anticipated shoring system, and construction procedures, as well as previously stated engineering assumptions, lateral earth pressures on the temporary shoring walls for the following cases are shown in Figures 5-1 through 5-4:

- Braced sheeting above the excavation
- Cantilevered sheeting above and below the excavation
- Surcharges from a sloped excavation, existing buildings, construction loads, and earthquake-induced loads
- Active and passive earth pressures on soldier piles below the excavation.

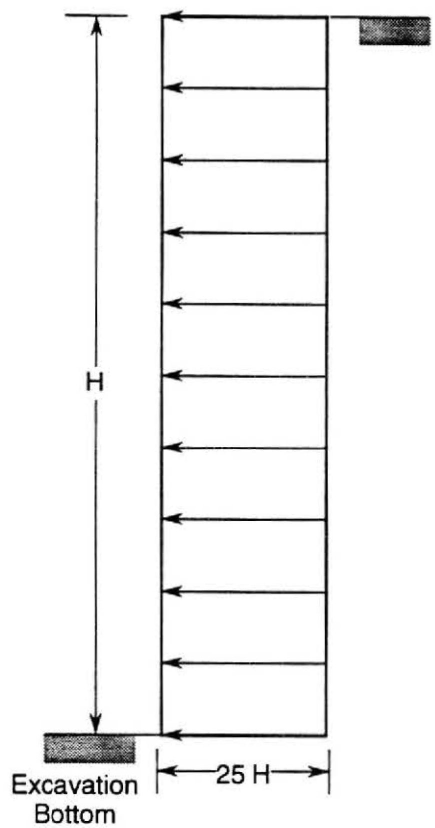
The lateral loading diagrams presented in Figures 5-1 through 5-3 are for use in the design of soldier pile details, tiebacks, or an internal bracing system.

Lateral earth pressures on lagging depend on a number of factors, including subsurface conditions and engineering properties, spacing between soldier piles, and dimensions and configuration of the excavation. For sizing purposes, the lateral earth pressure on lagging can be taken as 50 percent of that recommended for the temporary shoring walls (Figures 5-1 through 5-3), to account for soil arching effects.

It is understood that design and construction of an appropriate shoring system to minimize ground loss and disturbance of the site and adjacent buildings, and to provide a safe worksite is the sole responsibility of the Contractor. Various design considerations are described in the following sections for use as general guidelines.

5.6.5.2 Soldier Piles and Lagging

The soldier piles and lagging walls should be designed adequately resist lateral and vertical loads imposed by the excavation, existing structures, construction loading, environmental loading (such as earthquake loading), and the shoring system itself. Design considerations, including pile sizing, embedment depth, spacing, installation, and lagging provisions, should be in compliance with appropriate building codes and city requirements.



Lateral Earth Pressure
On Braced Sheeting

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Lateral Earth Pressure on Temporary Excavation Supports

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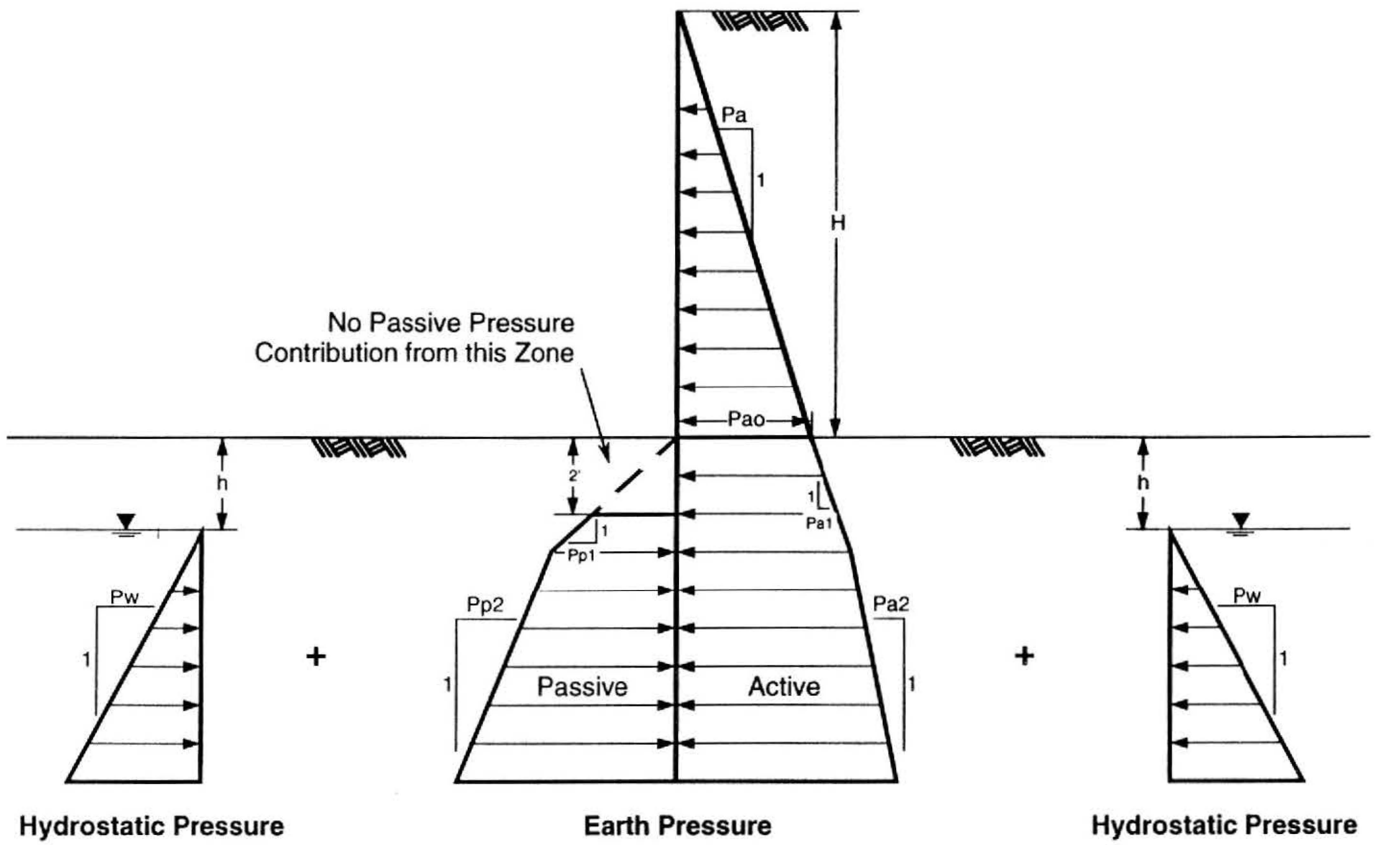
Figure 5-1



Pa = 35 psf/ft
Pao = 35H psf
Pa1 = 35 psf/ft
Pa2 = 18 psf/ft
Pp1 = 450 psf/ft
Pp2 = 225 psf/ft
Pw = 62.4 psf/ft

Note: The penetration obtained by using this pressure diagram should be increased by 20%

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Lateral Earth Pressure on Temporary Excavation Support Below Excavation

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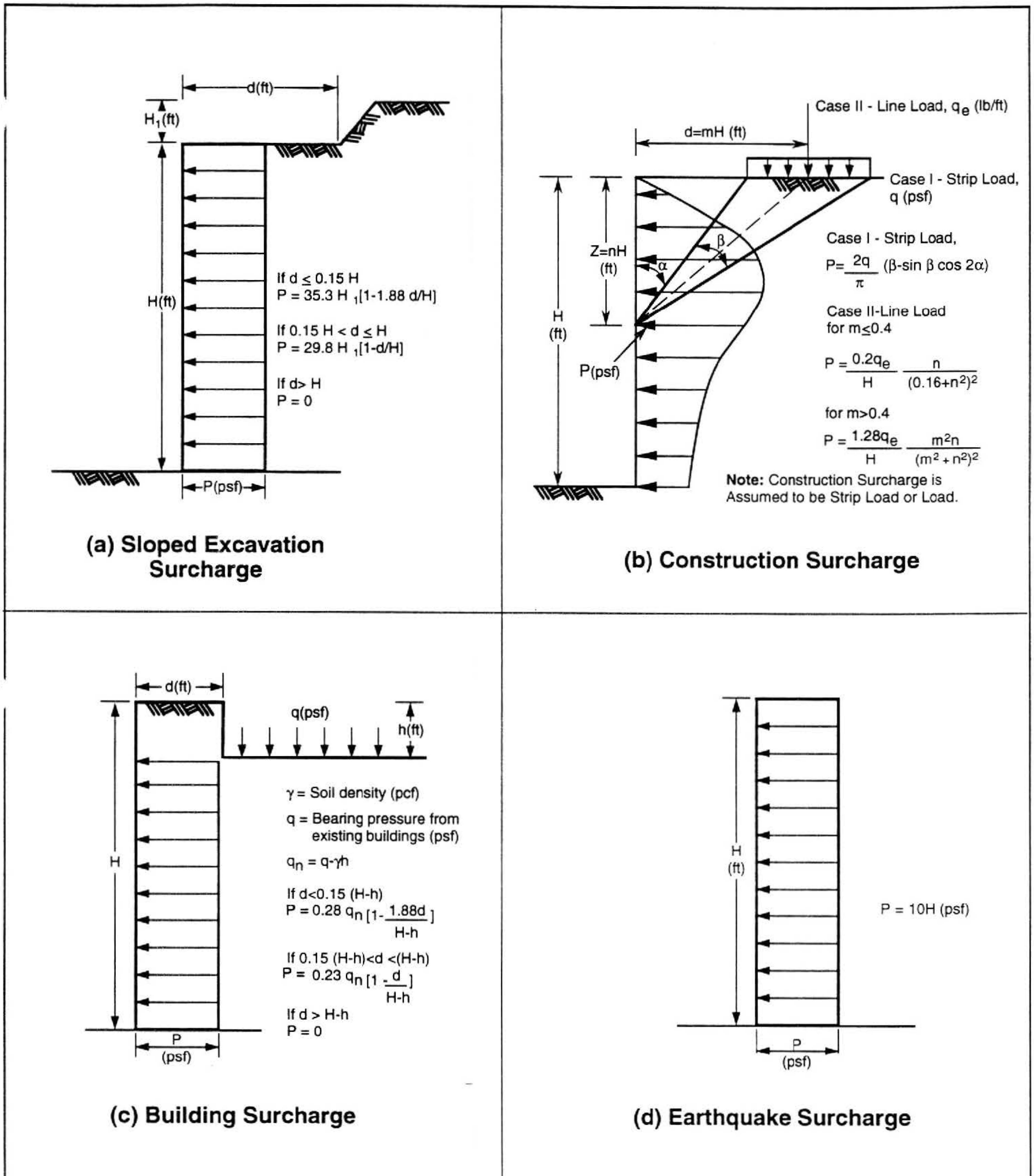
Figure 5-2



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Additional Lateral Earth Pressure on Braced or Cantilevered Sheeting

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Pile sizing includes proper determinations of pile size (diameter or cross section) and type (stiffness) so that stresses in the piles are within allowable limits. All anticipated lateral and vertical loads as well as calculated loads from tiebacks or internal bracing should be applied in calculating the pile stresses. The calculated stresses in the pile can be reduced by 20 percent to account for arching effects due to pile flexibility.

The soldier piles should be sufficiently embedded below the excavation depth to safely resist anticipated lateral and vertical loads. The passive resistance should exceed the imposed lateral loads (active resistance minus the resistance from tiebacks or internal bracing) with a reasonable safety factor. The effective excavation width that each pile can support should be taken as 1-1/2 times the soldier pile diameter or half of the pile spacing, whichever is less. It should be noted that piles may undergo some movement before mobilizing the anticipated capacities. It is recommended that at least one or two pile load tests be performed to verify estimated capacities and movement under design load will be acceptable.

Optimal pile spacing depends on a number of factors, including subsurface conditions and engineering properties of subsurface materials, pile sizing, construction procedures and cost. Considering the need for lagging to alleviate soil raveling and reduce ground loss, a pile spacing of 8 feet or less would be reasonable.

Local noise abatement requirements, and the presence of cobbles and boulders in alluvium 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 large size boulders may be required. Slurry and/or casing will be required to handle potential caving conditions within the granular alluvium.

Lagging between soldier piles will be required. It is understood that the Contractor will be responsible for controlling the temporary height of exposed soil prior to lagging placement to eliminate raveling and ground loss problems.

5.6.5.3 Tieback Anchors

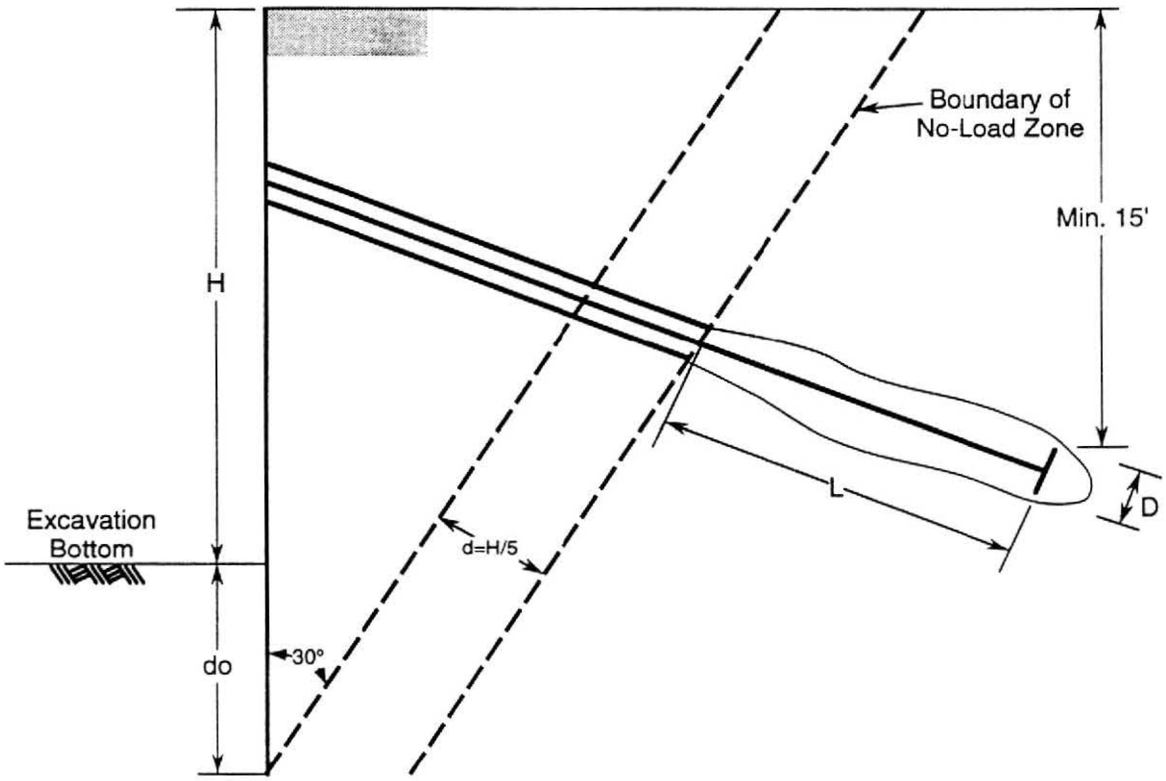
Installing tiebacks in the site area will require permission from the owners of adjacent buildings and avoidance of below-grade obstructions such as basements or foundations of adjacent buildings. Many types of tieback anchors exist, including shaft anchors, belled anchors, anchor blocks, and high-pressure grout anchors. In general, the allowable capacity of the tieback anchor should be determined in the field based on load tests. The following recommendations should be considered in the design and installation of tieback anchors.

Effective friction of a tieback anchor can develop only beyond a no-load zone. Our recommendations for the no-load zones, considering depth of excavation and potential wedge failure planes, are shown in Figure 5-4. The anchors may be installed at inclinations ranging between 20 degrees and 50 degrees below the horizontal. Potential caving conditions in the granular alluvium are possible, so the contractor should use appropriate measures to prevent caving and minimize ground loss. Each tieback anchor should be load tested to 150 percent of the design load in accordance with standard acceptance criteria (FHWA-DP-68-IR, November 1984) or local site-specific experience of the contractor. The load in the tiebacks should be locked off at 100 percent of the design load. The loads in a selected number of tiebacks should be periodically monitored and reloaded to 100 percent of the design load if the load decreases to less than 75 percent of the design load.

5.6.5.4 Internal Bracing

If braced shoring systems are employed, the strut loads should be determined using the full load diagrams shown in Figures 5-1 through 5-3. The vertical spacing between struts should be appropriately designed to reduce ground movements. All struts should be preloaded to eliminate slack and reduce ground movement. A preload of 25 percent of the design load is recommended. However, it should be noted that preload of 25 percent of design load may induce undue loading on basements, if any, of adjacent buildings. This possibility should be analyzed on a case-by-case basis.

Procedures to compensate for the effects of temperature changes on the strut loads should be developed and implemented so that proper strut load levels can be monitored and maintained during construction.



L= Bond length of anchor beyond no-load zone.
do= Depth of penetration required for stability.
do, d, D, H, and L are in feet.

Reference: NAVFAC Design Manual 7.02, 1986

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Anchor Locations for Tieback Walls

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Figure 5-4

5.6.5.5 Combined Shoring System

If a shoring system which combines external tiebacks and internal bracing or struts and wales is selected for support of excavations, the support design by the contractor must account for the variation in stiffness and deflection characteristics of the support elements which may induce substantially different load distributions.

5.6.5.6 Ground Movement

Station excavations will incur ground movements in terms of wall movement and ground heave. The magnitude of wall movement depends on many factors, including the design and construction of shoring systems, construction schedule, specifications, and subsurface conditions. In general, for a well-designed and constructed shoring system, the maximum horizontal wall deflection may be about 0.1 percent to 0.2 percent of the excavation depth. For the Little Tokyo Station the corresponding maximum horizontal wall movement may be about 1 inch to 2 inches. For a shoring system with tiebacks, this maximum horizontal deflection will likely occur near the surface, and will decrease with depth. For a well-designed and constructed internally braced system with struts and wales, the maximum horizontal deflection will probably occur near the bottom of the excavation and decrease to about 0.2 inch to 0.5 inch near the surface. It is estimated that, for a well-designed and constructed shoring system a maximum vertical settlement of about 0.5 inch to 1 inch will probably occur behind the wall to about 25 feet to 50 feet from the wall and will decrease as the distance from the maximum settlement location increases.

5.6.6 Protection of Adjacent Buildings/Structures and Underpinning

No building structures are directly above the station. Within 60 feet of the proposed station excavation, there exist two 3-story buildings (201 and 215 S. Santa Fe Avenue), an abandoned one-story warehouse and a one-story Metro Rail maintenance building in the MTA yard.

Station excavation and construction may cause some ground settlement or angular distortion. The Contractor is responsible for carrying out the excavation with timely placement of an adequate support system to mitigate potential disturbance to the adjacent buildings.

In general, the need for underpinning these adjacent buildings will depend on whether their foundations are adequate and whether the buildings can satisfactorily resist anticipated settlement due to excavation-related deformation. Thus, further structural and foundation evaluations by qualified structural engineers will be necessary for these buildings.

Figure 5-5 presents rough guidelines to estimate whether adjacent buildings require underpinning. The final need for underpinning should be evaluated by a qualified structural engineer on a case-by case basis. These adjacent buildings (whether underpinned or not) should be monitored for settlement and lateral movement on a regular basis during station construction. If the monitored settlements indicate the potential for excessive settlements beyond the maximum allowable limits preset by the engineers, excavation work should be suspended temporarily and immediate remedial measures taken.

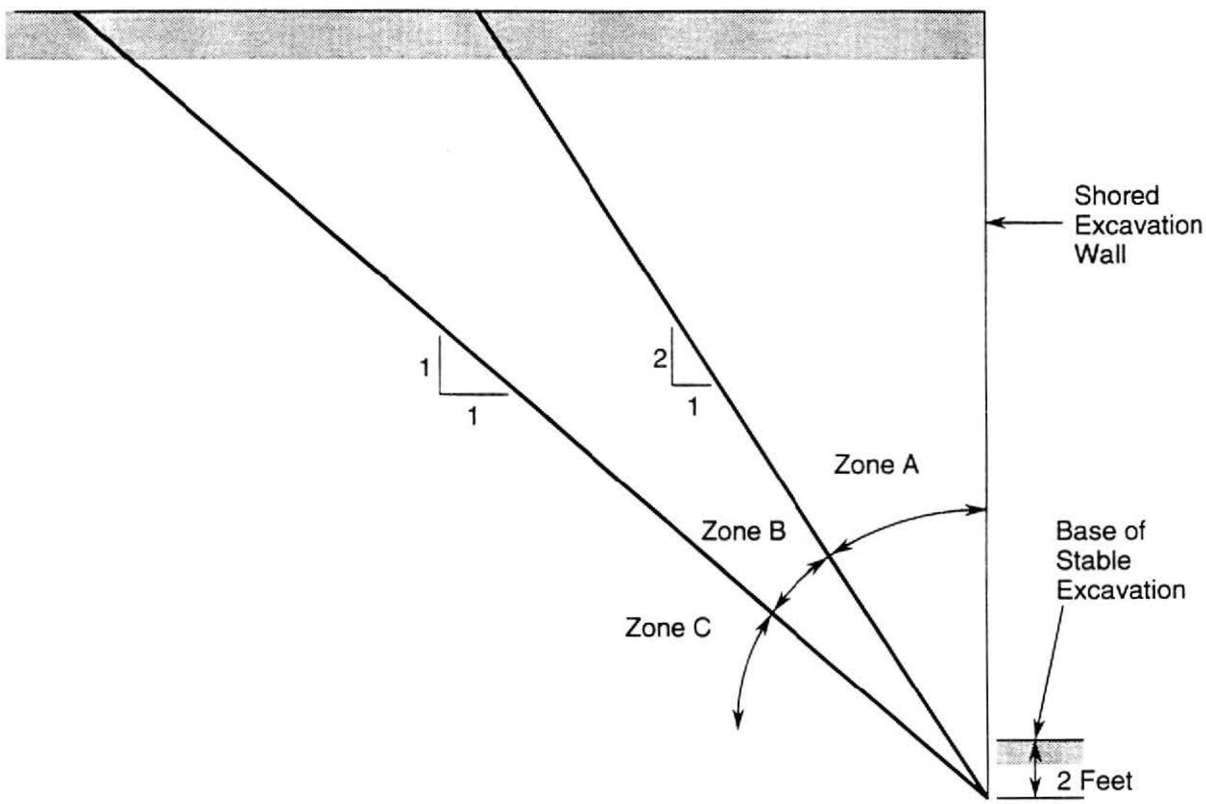
5.6.7 Excavation Heave

The excavation depth of the Little Tokyo Station is approximately 70 feet. This would mean a maximum stress relief of about 9,000 psf at the bottom of the excavation resulting in bottom heave due to elastic and consolidation rebounds. Based on the subsurface conditions and properties of the subsurface soils, it is estimated that the heave due to elastic rebound will be about 1-1/2 to 2 inches and the consolidation rebound will be about 1/2 inch to 3/4 inch. Elastic rebound will take place during the excavation. The bulk of the consolidation rebound is anticipated to occur within approximately one month following the excavation. Rupture of the excavation bottom due to excessive heave is not likely.

5.6.8 Foundation Support

5.6.8.1 Main Station Structure

The overall station will be designed and constructed as a relatively rigid box and the main station housing the rail facilities will be supported on wide, thick slabs that will function as relatively rigid mat foundations. The design of mat foundations are generally governed by settlement considerations rather than by bearing capacity. Available information indicates that the average bearing pressure on the mat foundations from



Notes: The following underpinning guidelines are for structure foundations in Zones A, B, and C.

Zone A: Foundations within this zone generally require underpinning

Zone B: Sensitive structure foundations within this zone generally require underpinning

Zone C: Underpinning may not be required

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General Guidelines for Underpinning

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Figure 5-5

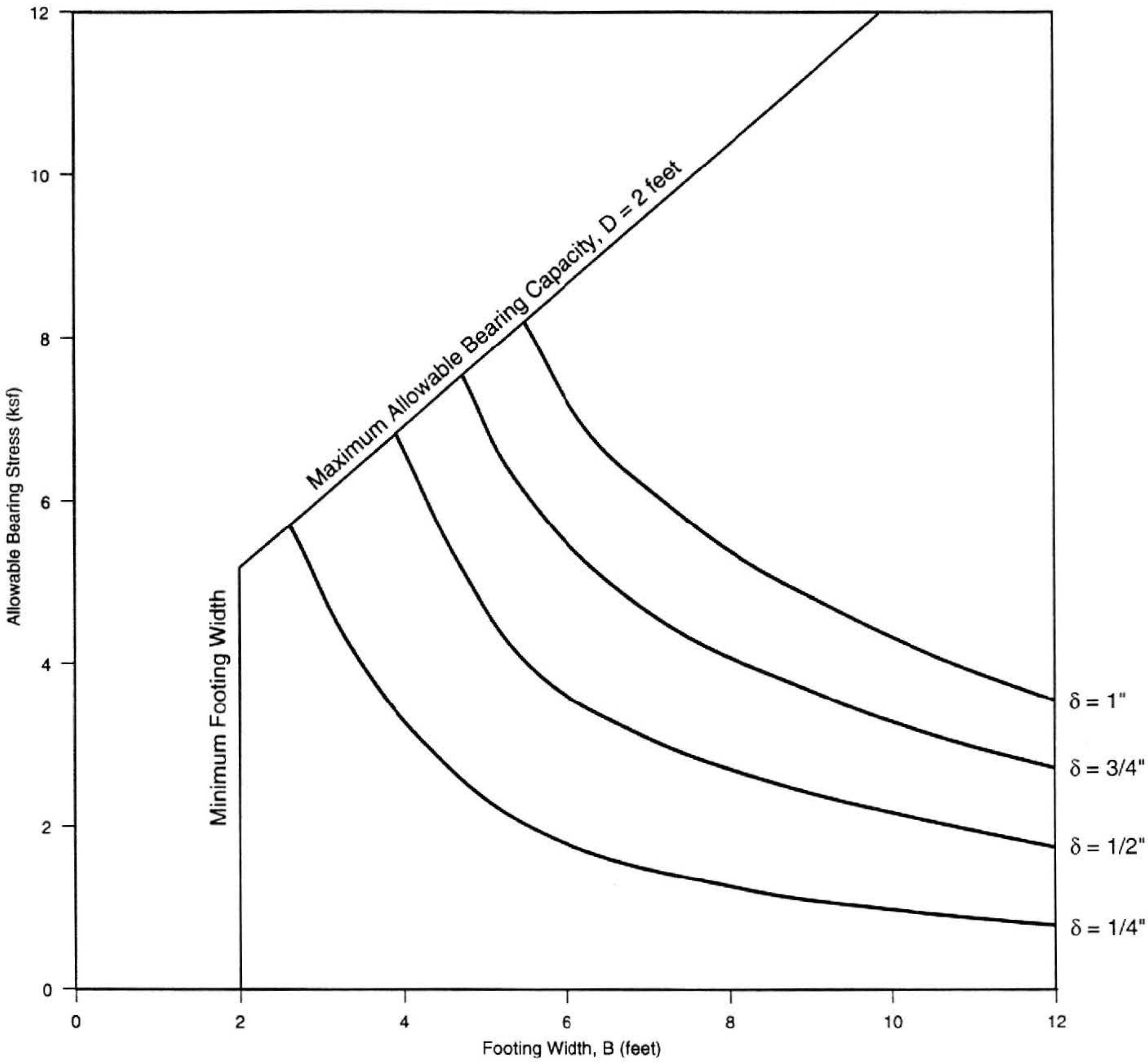
the station and backfill would be on the order of 5,000 psf, which is less than the overburden removed by the excavation. Therefore, this anticipated station load can be adequately supported on the alluvium underlying the station mat foundation. An allowable bearing capacity of 5,000 psf may be used for design of mat foundations provided the settlements estimated below are considered in the design.

It is estimated that elastic settlement due to the station load of 5,000 psf will be on the order of about 3/4 inch to 1-1/4 inch. This settlement would take place during construction. In addition to the immediate elastic settlements, 1/4 to 1/2 inch of consolidation settlement is estimated under the design load of 5,000 psf. Analyses indicates that about 90% of the consolidation settlement due to the station load will take place over a period of about 1 month following construction. Some differential settlement of the structure should be expected due to the nonuniformity of the soils at the site. It is estimated that the differential settlement over the width of the station will be on the order of 3/4 inch.

If the groundwater levels in the station area rise from current levels to the assumed design levels (i.e., about 50 feet above the station invert), the station mat foundation will experience a vertical stress relief of about 3,100 psf, resulting in heave due to elastic and consolidation rebounds. It is estimated that the corresponding elastic rebound will be on the order of about 1/2 to 3/4 inch, and the corresponding consolidation rebound will be on the order of 1/4 inch. In this case, differential heave over the width of the station is estimated to be about 1/2 inch.

5.6.8.2 Surface and Near Surface Structures

Near surface structures can be supported on conventional spread footings founded on properly compacted fill. All spread footings should be a minimum of 2 feet wide and at least 2 feet below the lowest adjacent grade. Subgrade for shallow footings should be overexcavated to a depth equal to the footing width and replaced with properly compacted fill. Overexcavation should extend a minimum distance equal to half the width or 2 feet, whichever is larger, beyond the perimeter of the footing. Fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Procedure D-1557. Allowable bearing capacities and estimated total settlement in terms of footing width and bearing pressures for shallow spread footings are graphically presented in Figure 5-6. Some differential settlement between adjacent footings should be anticipated. This differential settlement between adjacent footings is estimated to be one-half of the average total settlements or the differences in total settlement, whichever is larger.



Explanation:

δ = Immediate Settlement for a Square Footing

D = Footing Embedment Below Adjacent Grade

Note: Factor of safety of 3 is included.

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Allowable Bearing Capacity for Spread Footing in Compacted Granular Fill (Compacted to 95% of Maximum Dry Density)

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Figure 5-6

In the absence of specific information on loads, dimensions and locations of structures, Figure 5-6 is provided as a general guideline for foundation support needs. Structure-specific foundation recommendations should be provided when such information becomes available.

Figure 5-6 can be used for vertical, concentric loading. Bearing capacity will be reduced due to eccentric and/or inclined (combined vertical and horizontal loads) loading conditions. It is recommended that a site-specific and loading specific study be performed for such cases once design loading conditions are known.

Lateral resistance of the footing can be assumed to be provided by passive earth pressure on the side of the footing and friction resistance between the footing and soil. An allowable passive pressure of 250 psf per foot, and an allowable frictional coefficient of 0.4 are appropriate for lateral resistance considerations.

5.6.9 Geotechnical Input for Station Design

5.6.9.1 Geotechnical Parameters

Table 5-1 summarizes recommended geotechnical design parameters for station design based on available data. These design parameters include lateral earth pressure coefficients, unit weight of soil, shear strength, long term maximum groundwater level, modulus of subgrade reaction for shallow foundations, and seismic coefficients.

5.6.9.2 Soil Spring Constants for Main Station Structures

Estimated ranges and best estimates of static and dynamic soil spring constants for design of the main station structure are presented in Figure 5-7. It is recommended that the full estimated ranges be considered in the structural design to account for the potential variability of the subsurface materials in the station area. An upper bound/lower bound approach is recommended for the structural response analysis of the station box structure. The lower bound values of soil spring constants should be used for evaluation of settlement and deformations, while the upper bound values should be used for evaluation of stresses in the structures.

TABLE 5-1. GEOTECHNICAL DESIGN PARAMETERS FOR UNDERGROUND STRUCTURES (LITTLE TOKYO STATION)

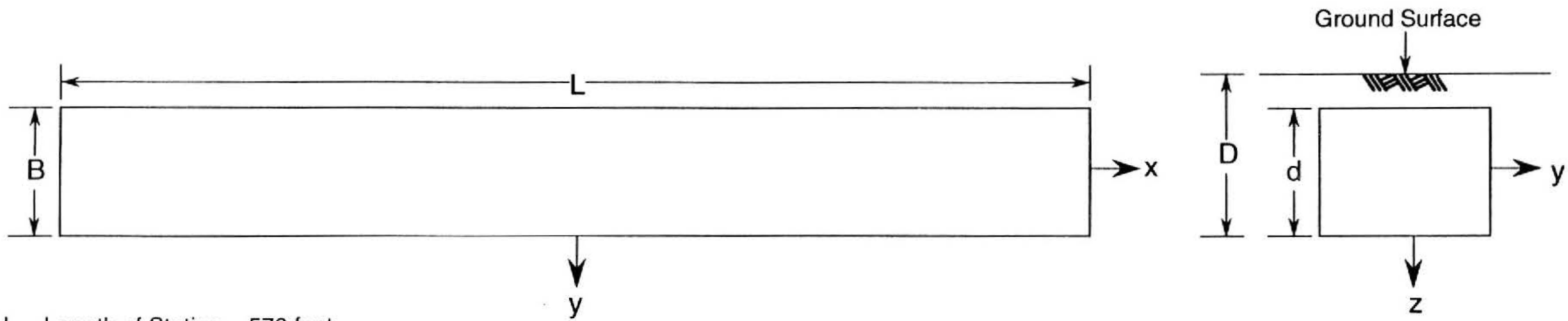
Parameter	Design Value
Geologic Unit	Coarse-Grained Alluvium
Unit Weight of Soil (pcf)	130
Void Ratio	0.42
Cohesion (psf)	0
Angle of Internal Friction of Soil (degrees)	35
Soil Pressure Coefficient	
At Rest (Ko)	0.45
Active (Ka)	0.27
Groundwater Depth(1)	25 feet below ground surface
For Support of Shallow Foundation: Coefficient of subgrade reaction K_1(2) (tons/ft^3)	
0 to 10 feet	50 to 75
10 to 40 feet	75 to 150
Footing of Width B, K_b(3) (tons/ft^3)	$K_b = \frac{K_1}{I}\left(\frac{B+1}{2B}\right)^2$
Seismic Coefficient	
ODE(4) Horizontal (Kh)	0.3g
Vertical (Kv)	0.2g
MDE(4) Horizontal (Kh)	0.6g
Vertical (Kv)	0.4g

Notes: (1) Assumed maximum groundwater level during the design life of the station. Current groundwater levels are below station invert (Elevation 197 feet).

(2) K_1 = Coefficient of subgrade reaction for a 1' x 1' plate.

(3) Kb = Coefficient of subgrade reaction for shallow foundation of width B and length mB. I=1 for m=1, I = 1.12 for m = 2, I = 1.6 for m = 5, I = 2 for m $\geq$10.
An upper bound/lower bound approach is recommended for the structural response analysis of the station structure. The lower bound values of modulus of subgrade reaction should be used for evaluation of settlements while the upper bound values should be used for evaluating stresses in the structure.

(4) Operating Design Earthquake (ODE) and Maxmium Design Earthquake (MDE) are based on Seismic Investigation and Design Criteria for Metro Rail Project prepared by Converse Consultants, Earth Sciences Associates and Geo/Resource Consultants (1983).



Plan View **Cross Section**

L = Length of Station = 570 feet
B = Width of Station = 60 feet
D = Depth of Station Invert = 69 feet
d = Height of Station Wall = 51 feet
k_x, k_y, k_z, = Soil Spring Constants in x, y and z directions, respectively.

Loading Condition	Applicable Condition	Shear Modulus G, 10^6 psf	Poisson's Ratio	Estimated Soil Spring Constant (Stiffness) 10^6 lb/ft		
				k_z	k_y	k_x
Static (high strain)	Estimated Range	0.5 to 1.5	0.3 to 0.4	650 to 2000	750 to 2300	650 to 1950
	Best Estimate	0.7	0.35	910	1090	920
Dynamic* (low strain)	Estimated Range	5 to 15	0.3 to 0.4	6800 to 20500	7900 to 23900	6500 to 19700
	Best Estimate	7	0.35	9600	11200	9200

Note:

* Actual dynamic stiffness is frequency-dependent. Approximate average numbers (over a wide frequency range) are provided for simplification purposes.

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Estimated Soil Spring Constant Values (Little Tokyo Station)

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Figure 5-7

5.6.9.3 Loads on Station Walls and Slabs

We understand that the station will be designed and constructed as a relatively rigid, water-tight box. The recommended lateral earth pressures including hydrostatic pressure and earthquake loading are shown in Figure 5-8. The following are noted:

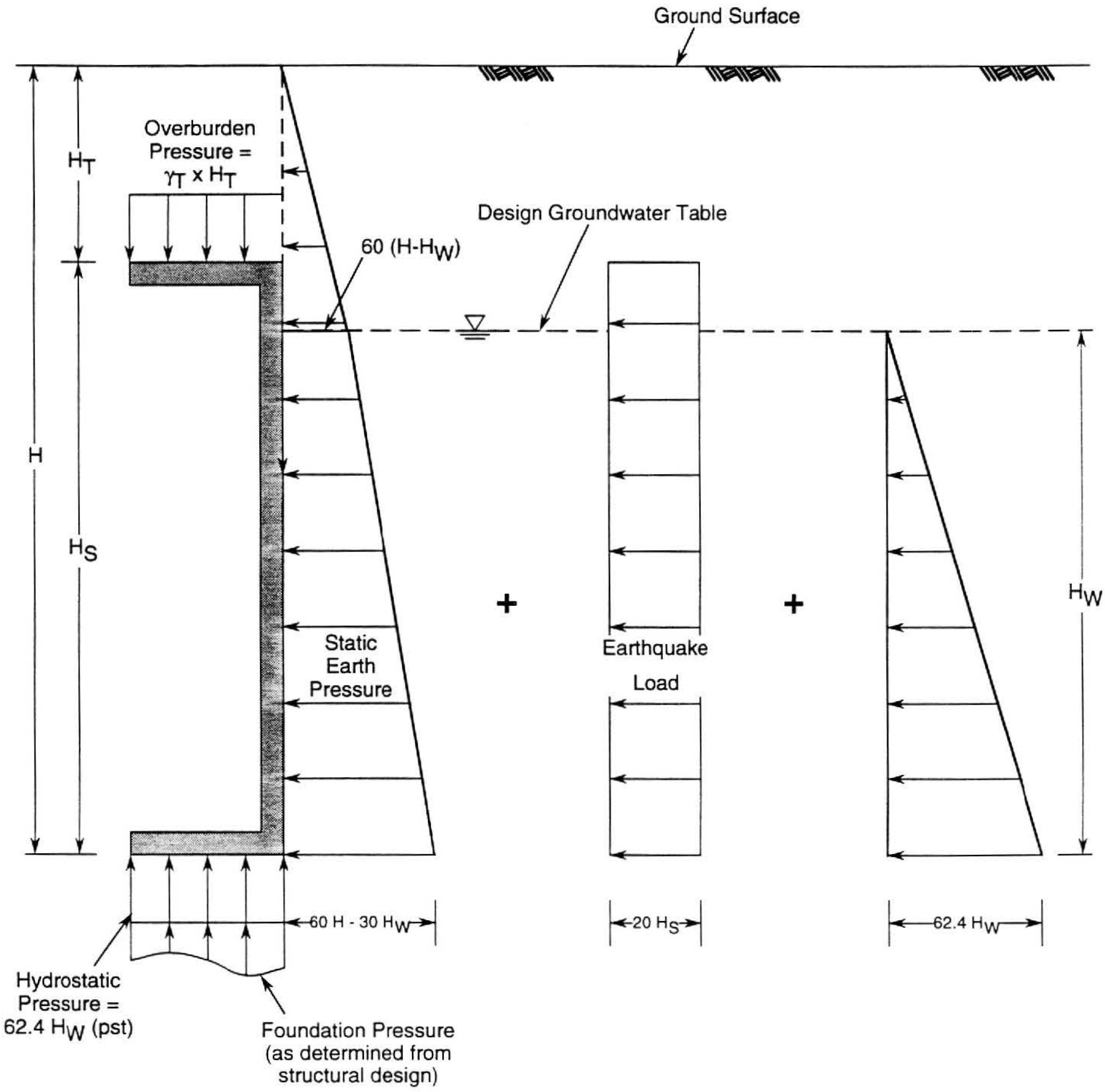
- Assumptions for groundwater levels during the design life of the station and maximum design earthquake (MDE) described in Sections 5.3 and 5.4 were utilized in developing Figure 5-9.

- The Mononobe-Okabe procedure was utilized for developing earthquake-induced lateral earth pressures. This procedure assumes that the wall yields or rigidly moves sufficiently for active conditions to develop during earthquake loading. In developing the earthquake-induced earth pressure recommendations, the potential effects of vertical ground acceleration were ignored to avoid being too conservative.

- Potential surcharge effects from adjacent buildings which are not underpinned should be considered in the wall pressure diagrams. Lateral surcharge loads on walls can be calculated in accordance with the recommendation shown in Figure 5-3c. Vertical loads from anticipated traffic and other live loads should be determined and added to the roof loading.

5.6.10 Liquefaction Potential

Liquefaction potential of the subsurface soils in the station area was evaluated as described in Section 5.5.3. The assumed highest groundwater levels described in Section 5.3 were utilized in the analysis to account for the effects of potential groundwater fluctuations on the liquefaction potential of subsurface materials. The results indicate that liquefaction of the site soils under the maximum design earthquake is unlikely.



Notes:
(1) All pressure in psf
(2) Design groundwater level is at about elevation 241 feet (about 25 feet below ground surface).
(3) H, H_T, H_S, and H_W are in feet.
(4) γ_T - Total density of soils.
(5) Drawing is schematic.

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Earth Pressure on Below-Grade Permanent Structures

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5.6.11 Earthwork

Based on the subsurface conditions encountered in the investigation, the station excavation can be accomplished relatively rapidly by conventional and readily available excavation technology. Earthwork and site preparation activities are expected to consist of an excavation for subterranean structures, subgrade preparation for the station floor, foundation preparation for near-surface structures, excavation for utility trenches, subgrade preparation for pavements, and backfill placement for subterranean walls, footings and utility trenches. Major excavations will need to be provided with temporary shoring according to the recommendations presented in Section 5.6.5. Other minor excavations, subgrade preparation and backfill placement should be done in accordance with the guidelines presented in Appendix E. All work should be in compliance with applicable city (Los Angeles), state (California), and federal (Occupational Safety and Health Act) requirements.

In general, the mat foundation should be underlain by a minimum 2-foot-thick dense granular material with an in situ density of at least 95 percent of the maximum dry density as per ASTM D-1557. In granular alluvium, this may be achieved by proof rolling the excavated subgrade with a heavy vibratory roller, or by overexcavating and compacting the 2-foot zone below the foundation in layers. In fine grained alluvium the subgrade should be overexcavated a minimum of 2 feet below design grade and replaced with granular material compacted to specification. If the mat is placed directly on the native materials, rock fragments larger than 6 inches should not be allowed in the exposed subgrade. The compacted fill blanket should extend a minimum of 5 feet beyond the foundation perimeter.

Materials excavated from non-contaminated granular alluvium (sand, silty sand, gravelly sand, sandy gravel, and gravel) could be stockpiled to be reused as backfill material. Excavated fine-grained alluvium is not suitable as backfill material. If there is insufficient material available for backfill, imported granular material could be used for fill subject to the approval of a geotechnical engineer.

5.7 SUMMARY OF SOIL AND GROUNDWATER CONTAMINATION

5.7.1 Groundwater Contamination

As described in Section 4.3.3.3, groundwater within the tunnel envelope approximately between Union Station and the vicinity of Boring DD-10 is likely contaminated with hydrocarbons, hydrogen sulfide (H_2S) and other constituents (sulfate, sulfide, chloride, etc) with concentration levels exceeding published threshold concentrations. Thus, groundwater from dewatering for the Union-Little Tokyo Tunnels will require treatment prior to disposal.

The remaining areas of tunnel and station construction may encounter local perched groundwater conditions. Some of these perched water inflows may be contaminated with hydrocarbons as evidenced by the encountering of free product in Boring SD-2, probably as a result of leakage from nearby oil pipelines in the project area, and may also require treatment prior to disposal.

5.7.2 Soil Contamination

As described in Section 4.3.3.4 tunnel and station construction will likely encounter the following soil contaminants:

- Soil, especially the fine-grained Fernando/Puente Formation bedrock, near and below groundwater table may be contaminated with hydrocarbons and hydrogen sulfide. These soil contaminants are probably related to groundwater contamination and are especially likely to occur within the tunnel envelope between the Union Station and the vicinity of Boring DD-10.

- Localized soil contamination with hydrocarbons is possible in the remaining tunnel portion and within the Little Tokyo Station area. The sources of localized soil contamination may include leakage from the existing oil pipelines and the existing Union Station oil field.

5.8 GASSY CONDITIONS

As summarized in Section 4.3.3.5, there is a high potential for accumulation of toxic and explosive gases, especially methane and hydrogen sulfide, in the project area. Available data indicates that methane and hydrogen sulfide are released from or through the groundwater. Thus, as in the case of groundwater contamination, there is a higher potential for high concentrations of methane and hydrogen sulfide between Union Station and the vicinity of Boring DD-10 than elsewhere in the project area. Because of the permeable nature of the predominantly coarse alluvium in the project area, which enables the dissipation of the groundwater-released gases, the rate and amount of accumulation will be significantly less than what was observed in the well casings of various monitoring wells (Section 4.3.3.5).

Proper ventilation and monitoring of methane, hydrogen sulfide and other toxic/explosive gases will be necessary to satisfy the U.S. and California Occupational Safety and Health Administration (OSHA) requirements and to provide a safe working environment.

In addition to being potentially present above the groundwater table within the area of concern, hydrogen sulfide may also be present within the previously saturated zones that became unsaturated upon dewatering. The release of hydrogen sulfide from soil would occur during the mixing process associated with tunnel excavation and muck handling. This possibility should be considered in the design, construction and operation of the facilities within the affected area.

5.9 ABANDONED OIL WELLS

Due to the proximity of the project area to the Union Station oil field, 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 move the casings, such abandoned wells, if encountered, may contain large quantities of water or even oil under pressure. The abandoned wells may also contain residual accumulations of hydrogen sulfide, methane or other toxic/explosive gases.

5.10 CORROSIVE SOILS

Results of sulfate content tests (Table 4-2) indicate that the soils in the project area are mildly to moderately corrosive to concrete. Type II cement should be used for concrete in contact with mildly to moderately corrosive soil. Results of available electrical resistivity tests from the preliminary geotechnical investigations (Table 4-2) and the current investigation indicate that most of the subsurface soils are moderately (2000 to 5000 ohm-cm range) to extremely corrosive (less than 2,000 ohm-cm) to metals.

5.11 FAULT CROSSING

As described in Section 4.2.3, the location of the "bedrock high" is approximately aligned with the projection of the escarpment of the Coyote Pass fault. This implies that this fault may potentially cross the tunnel segment in the vicinity of the "bedrock high". The results of project-specific fault investigations performed by GeoTransit Consultants to delineate and characterize this fault and to assess its seismic capability are presented in a fault-investigation report (GeoTransit Consultants, 1996). The results indicate that the "Coyote Pass Escarpment" is active. The recommended magnitudes of fault movement for tunnel design at the fault crossing are provided in the fault investigation report.

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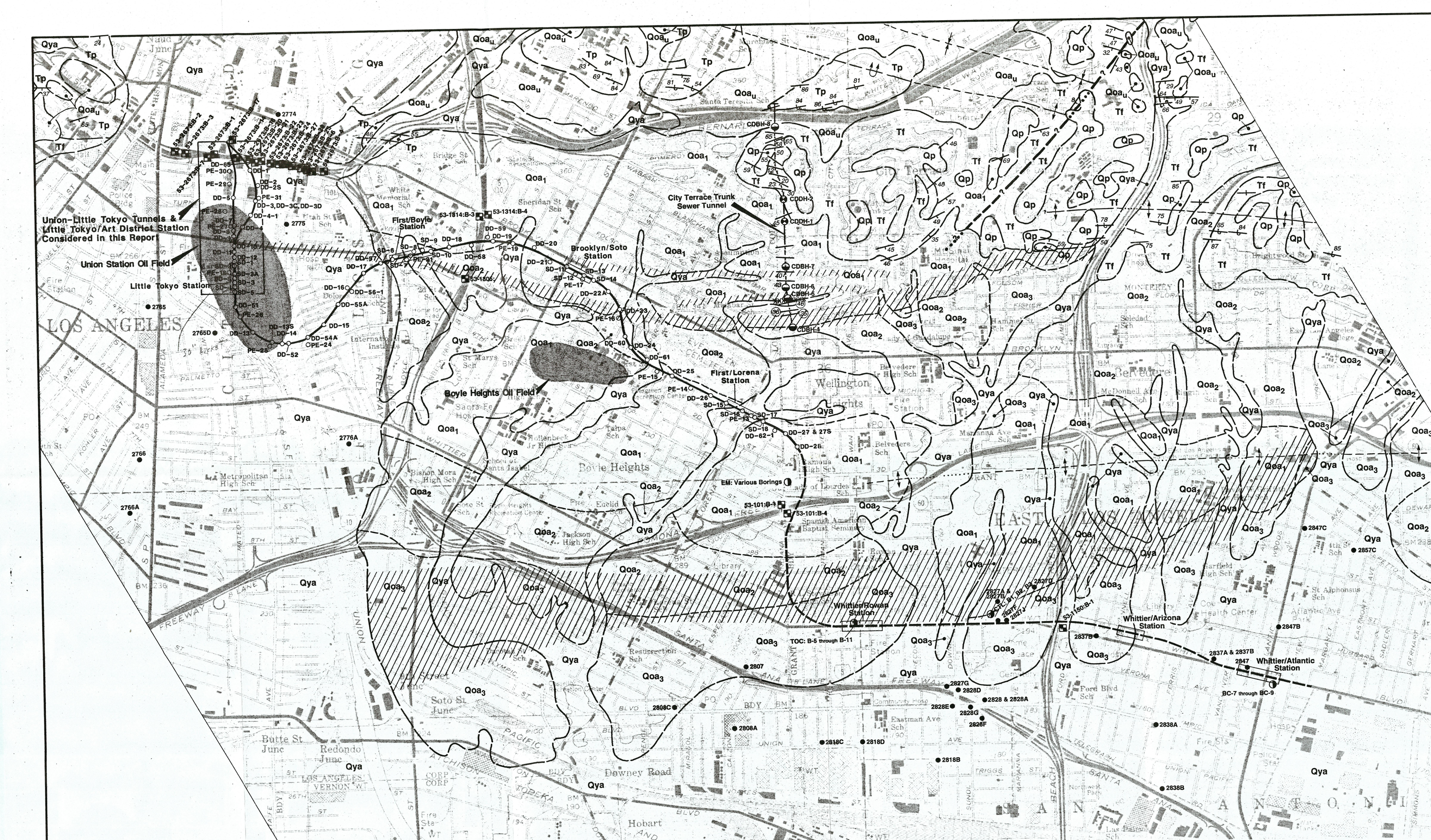
7.0 LIMITATIONS

The conclusions and professional opinions presented in this report were developed by GeoTransit Consultants for Engineering Management Consultant (EMC). GeoTransit Consultants makes no warranty, either expressed or implied, as to its findings, opinions, recommendations, specifications, or professional advice except that these were developed after being prepared in accordance with generally accepted standards of care and diligence normally practiced by recognized consulting firms performing services of a similar nature.

Subsurface conditions are, by their nature, uncertain and may vary from those tested in the laboratory, documented in historical documents or encountered at the locations where visual inspections, borings, soundings, test pits, surveys, or other explorations were made by GeoTransit Consultants. The data, interpretations, and recommendations of GeoTransit Consultants ar based solely on such information or from information obtained by others in the area covered by this report, and or observations from borings in the area covered by this report.

The data and conclusions contained herein should be considered to relate only to the specific project and location discussed herein. GeoTransit Consultants is not responsible for any conclusions that may be made from these data by others unless we have been given an opportunity to review such conclusions and concur in writing. This report has not been prepared for use by parties other than EMC. It may not contain sufficient information for the purposes of other uses. If any changes are made in the project as outlined in this report, the conclusions contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified and approved in writing by Geotransit Consultants.



NORTH

0 500 1000 Feet
Scale

EXPLANATION

GEOLOGIC UNITS

	Qya — YOUNGER ALLUVIUM	Q_4 and Q_5 of Bullard and Lettis (1993)
Qalo of Lamar (1970)	Qoa_u / Qoa_3, Qoa_2, Qoa_1 — OLDER ALLUVIUM	Q_1, Q_2 and Q_3 of Bullard and Lettis (1993)
	Qp — OLDER GRAVEL	
	Tf — FERNANDO FORMATION	
	Tp — PUENTE FORMATION	

SYMBOLS

- Fault/Escarpment
- TUNNEL ALIGNMENT (dashed denotes future alignment)
- 71 STRIKE AND DIP OF BED
- STRIKE OF VERTICAL BED
- 82 STRIKE AND DIP OF OVERTURNED BED
- AXIS OF ANTICLINE; short dashed in older alluvium, dotted where buried, queried where uncertain or continuation unknown
- AXIS OF OVERTURNED ANTICLINE
- AXIS OF SYNCLINE; short dashed in older alluvium, dotted where buried, queried where uncertain or continuation unknown
- GEOLOGIC CONTACT, short dashed between subunits of older alluvium
- POSSIBLE FAULT; dotted where buried

- Project-Specific Geotechnical Borings; Tunnel Borings Denoted by DD-, Station Borings Denoted by SD-, Borings from Preliminary Engineering Program Denoted by PE-
- 53-2673-B2 ROTARY BORING; Caltrans, various dates
- 2818D WATER WELL; monitored by L.A. Co. Dept. Public Works
- BC-, CTL-, EM-, TOC- Selected Subsurface data from Environmental Studies; Brown and Caldwell Consultants (1993), CTL Environmental Services (1992), Emcon Southwest (1992 and 1993), Thrifty Oil Co., (1993) — Locations shown are for multiple, closely spaced borings
- SEWER TUNNEL ALIGNMENT
- CDDH-1 Core boring by Converse, Davis and Associates (1972)
- CDBH-5 Bucket auger boring by Converse, Davis and Associates (1972)
- 47, 35 Bedding attitude in tunnel by Leroy Crandall and Associates (1979); circled number indicates amount of dip in Pleistocene deposits
- Shear zone in tunnel

Geology Adapted and Modified from Lamar (1970) and Bullard and Lettis (1993).

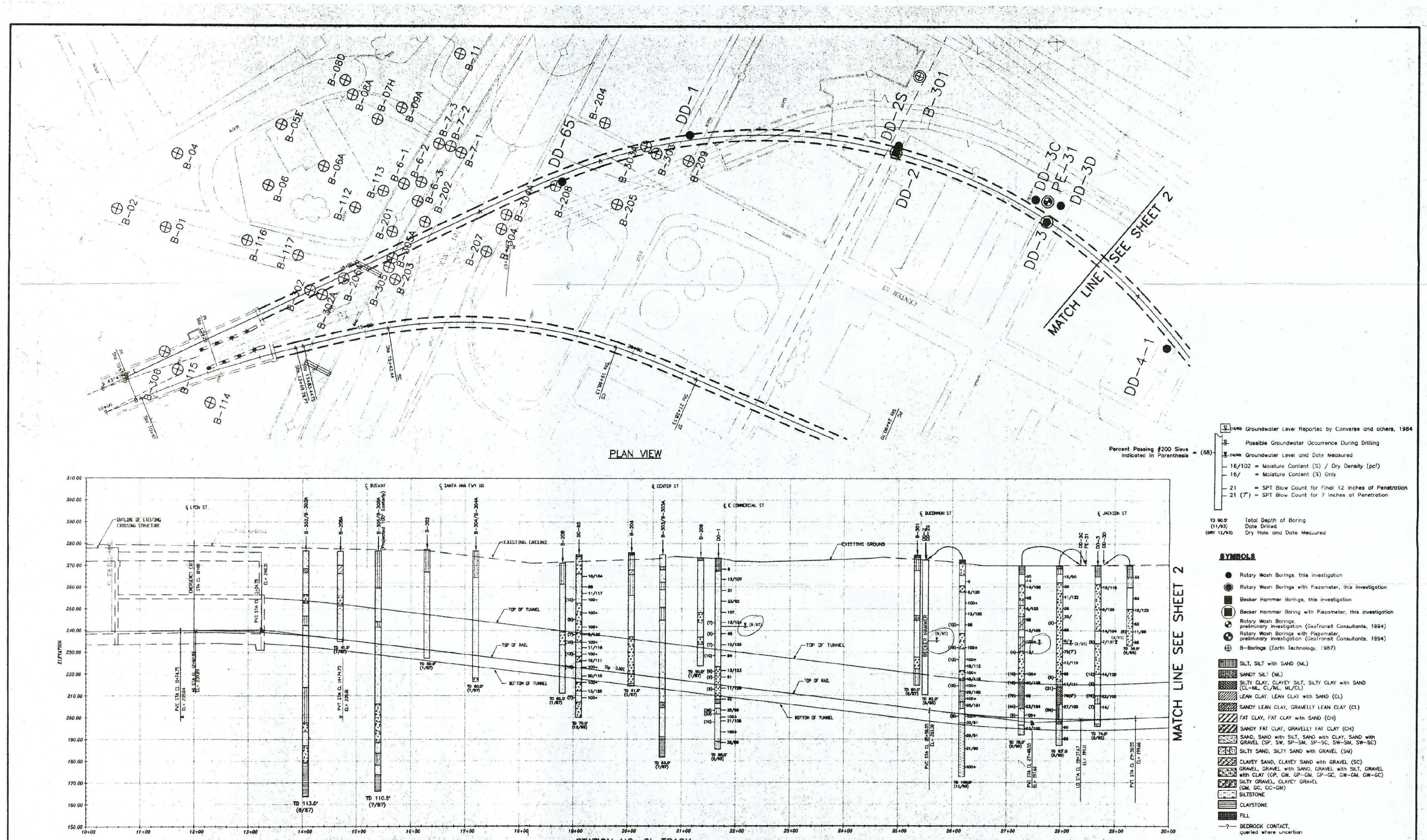
Base Map: U.S. Geological Survey, Los Angeles 7.5 Minute Quadrangle; dated 1966, photorevised 1981.

GeoTransit Consultants	Project No.: 95-8347-04 Geotechnical Investigation Eastside Extension Metro Red Line

Key Plans and Geology

12-95

Plate 1



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REV.	DATE	BY	SUB	APP	DESCRIPTION
A	12-28-95				

DESIGNED BY:
DRAWN BY:
CHECKED BY:
IN CHARGE:
DATE:

Metropolitan Transportation Authority
METRO RED LINE

GeoTransit Consultants

SUBMITTED:

METRO RED LINE
EASTSIDE EXTENSION
PLAN AND PROFILE – CL TRACK

PLATE 5

CONTRACT NO:
DRAWING NO: REV.
SCALE: HORIZ: 1'=80' VERT: 1'=20'
SHEET NO: 1 OF 2

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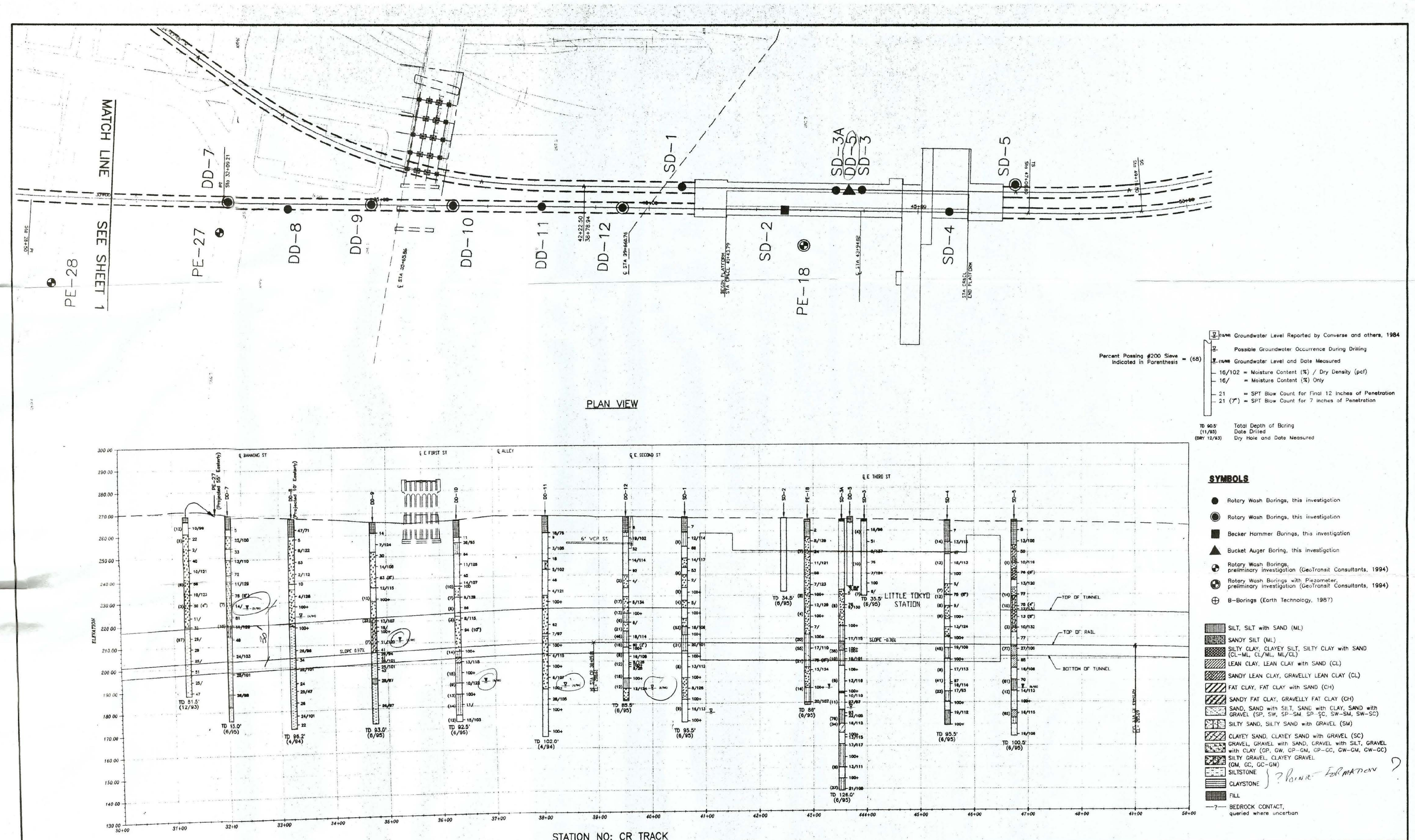

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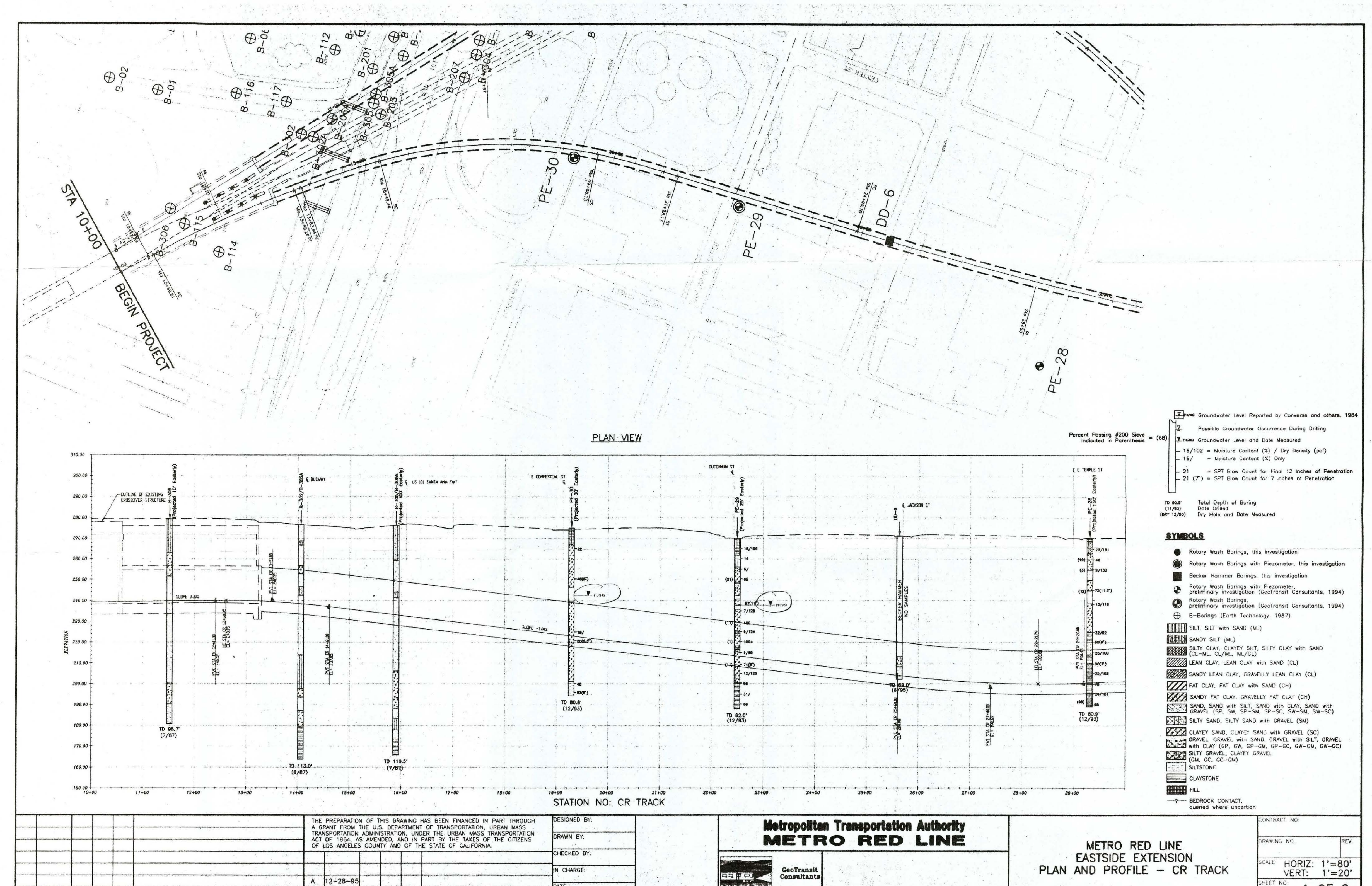
GeoTransit Consultants

SUBMITTED:

METRO RED LINE
EASTSIDE EXTENSION
PLAN AND PROFILE – CR TRACK

PLATE 4

CONTRACT NO.
DRAWING NO. REV.
SCALE HORIZ: 1'=80' VERT: 1'=20'
SHEET NO. 2 OF 2


STA 10+00
BEGIN PROJECT
PE-30
PE-29
DD-6
PE-28
PLAN VIEW
STATION NO: CR TRACK
Percent Passing #200 Sieve Indicated in Parenthesis = (68)
Groundwater Level Reported by Converse and others, 1984
Possible Groundwater Occurrence During Drilling
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
21 (7") = SPT Blow Count for 7 Inches of Penetration
TD 90.5' Total Depth of Boring
(11/93) Date Drilled
(DRY 12/93) Dry Hole and Date Measured
SYMBOLS
Rotary Wash Borings, this investigation
Rotary Wash Borings with Piezometer, this investigation
Becker Hammer Borings, this investigation
Rotary Wash Borings with Piezometer, preliminary investigation (GeoTransit Consultants, 1994)
Rotary Wash Borings, preliminary investigation (GeoTransit Consultants, 1994)
B-Borings (Earth Technology, 1987)
SILT, SILT with SAND (ML)
SANDY SILT (ML)
SILTY CLAY, CLAYEY SILT, SILTY CLAY with SAND (CL-ML, CL/ML, ML/CL)
LEAN CLAY, LEAN CLAY with SAND (CL)
SANDY LEAN CLAY, GRAVELLY LEAN CLAY (CL)
FAT CLAY, FAT CLAY with SAND (CH)
SANDY FAT CLAY, GRAVELLY FAT CLAY (CH)
SAND, SAND with SILT, SAND with CLAY, 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, GRAVEL with SILT, GRAVEL with CLAY (GP, GW, GP-GM, GP-GC, GW-GM, GW-GC)
SILTY GRAVEL, CLAYEY GRAVEL (GM, GC, GC-GM)
SILTSTONE
CLAYSTONE
FILL
BEDROCK CONTACT, queried where uncertian
THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED IN PART THROUGH A GRANT FROM THE U.S. DEPARTMENT OF TRANSPORTATION, URBAN MASS TRANSPORTATION ADMINISTRATION, UNDER THE URBAN MASS TRANSPORTATION ACT OF 1964, AS AMENDED, AND IN PART BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.
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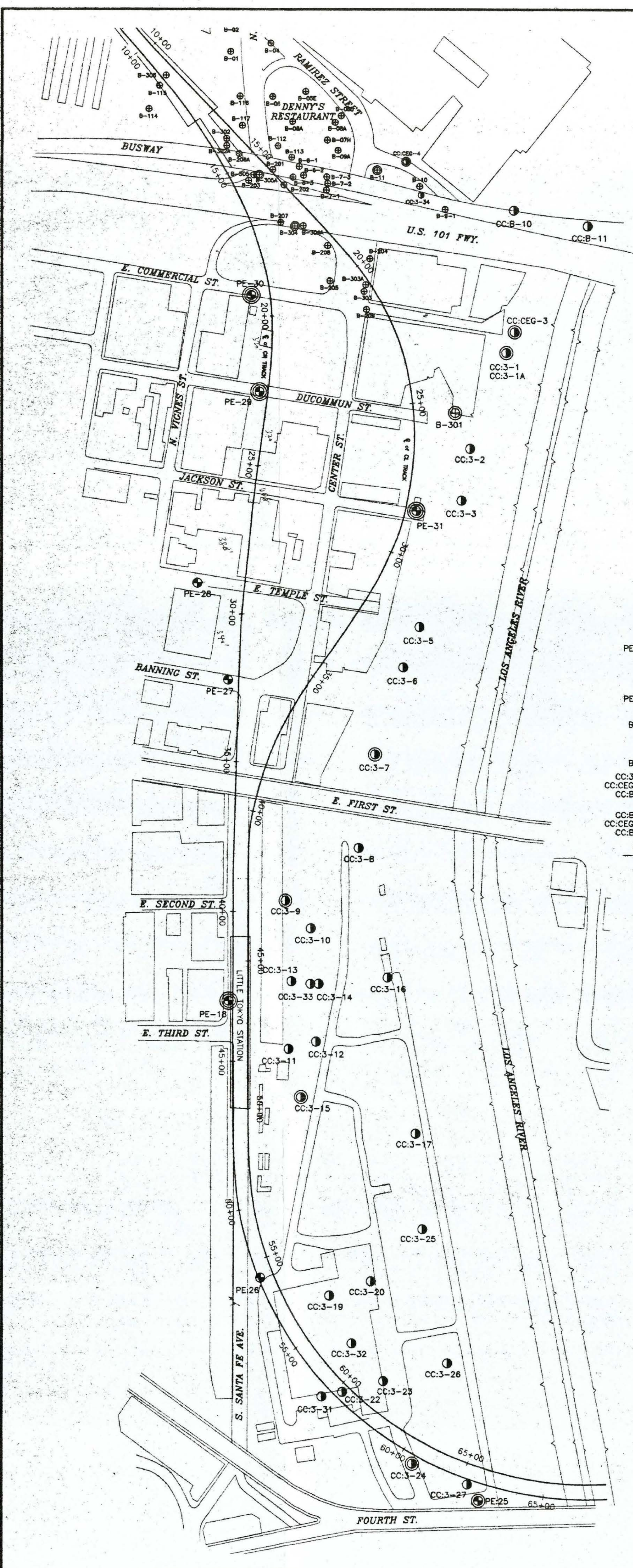
GeoTransit Consultants
SUBMITTED:
METRO RED LINE
EASTSIDE EXTENSION
PLAN AND PROFILE - CR TRACK
PLATE 4
CONTRACT NO:
DRAWING NO.
REV.
SCALE: HORIZ: 1'=80'
VERT: 1'=20'
SHEET NO: 1 OF 2

12-95 Geo Transit



EXPLANATION

PE- ROTARY WASH BORING WITH PIEZOMETER
PRELIMINARY INVESTIGATION. (GEOTRANSIT CONSULTANT, 1994a)
PIEZOMETERS CONVERTED TO MONITORING WELLS SHOWN BY

PE- ROTARY WASH BORING; PRELIMINARY INVESTIGATION (GEOTRANSIT CONSULTANTS, 1994a)

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

PROPOSED BORINGS

- BUCKET AUGER
- PIEZOMETER/WELL
- BECKER HAMMER
- COMPLETED BORING
- HSA/RW BORING

NORTH

SCALE
0 200 400 FEET

NOTE: BORING SYMBOLS AND DESIGNATIONS HAVE BEEN REDUCED IN SIZE IN SOME AREAS FOR CLARITY

GeoTransit Consultants

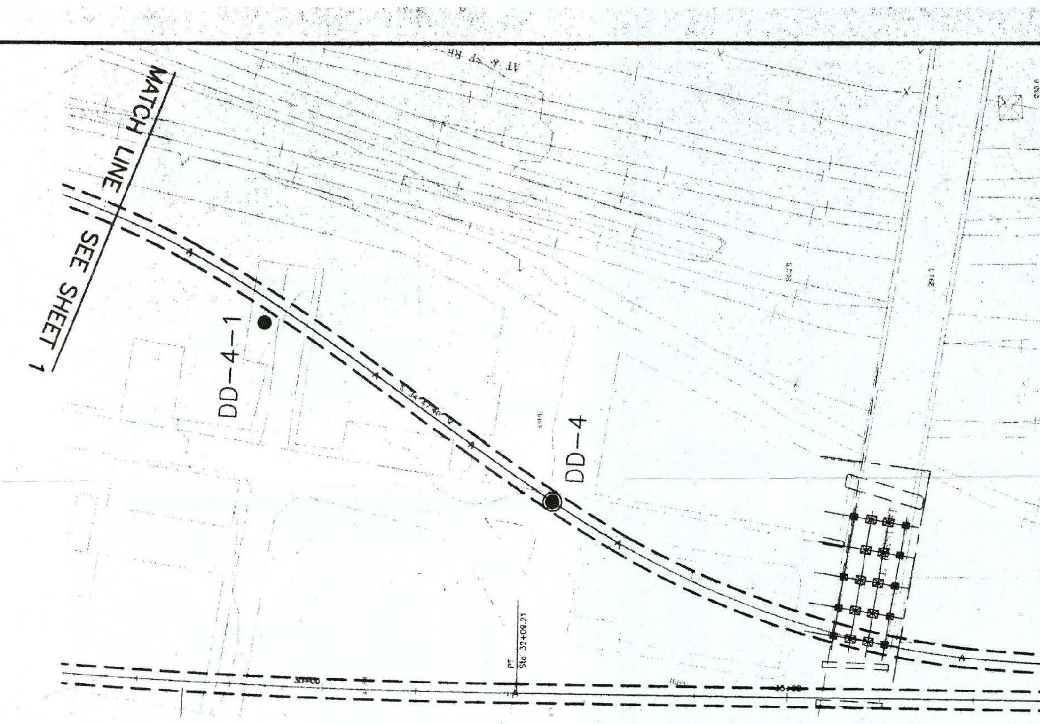
Project No.: 95- 8347

Geotechnical Investigation
Eastside Extension
Metro Red Line

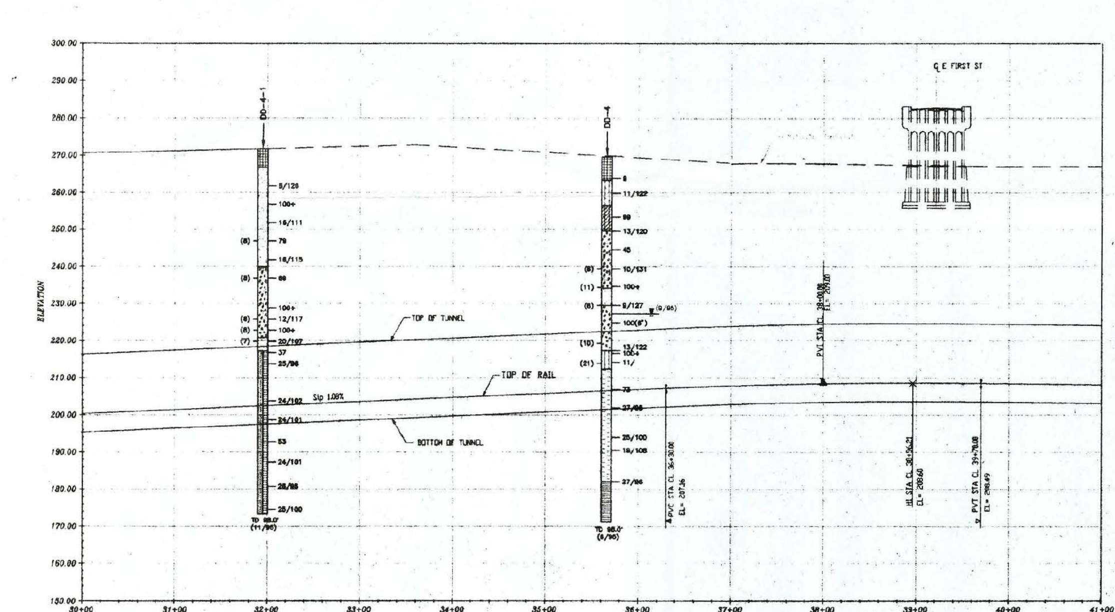
AVAILABLE GEOTECHNICAL BORINGS IN THE PROJECT AREA

5-95

Plate 2



PLAN VIEW



STATION NO: CL TRACK

Groundwater Level Reported by Converse and others, 1984

Possible Groundwater Occurrence During Drilling

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

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

21 (7") = SPT Blow Count for 7 Inches of Penetration

TD 80.5' Total Depth of Boring

(11/93) Date Drilled

(DRY 12/93) Dry Hole and Date Measured

SYMBOLS

- Rotary Wash Borings, this investigation
- Rotary Wash Borings with Piezometer, this investigation
- Becker Hammer Borings, this investigation
- Rotary Wash Borings with Piezometer, preliminary investigation (GeoTransit Consultants, 1994)
- Rotary Wash Borings, preliminary investigation (GeoTransit Consultants, 1994)
- B-Borings (Earth Technology, 1987)

- SILT, SILT with SAND (ML)
- SANDY SILT (ML)
- SILTY CLAY, CLAYEY SILT, SILTY CLAY with SAND (CL-ML, CL/ML, ML/CL)
- LEAN CLAY, LEAN CLAY with SAND (CL)
- SANDY LEAN CLAY, GRAVELLY LEAN CLAY (CL)
- FAT CLAY, FAT CLAY with SAND (CH)
- SANDY FAT CLAY, GRAVELLY FAT CLAY (CH)
- SAND, SAND with SILT, SAND with CLAY, 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, GRAVEL with SILT, GRAVEL with CLAY (GP, GW, GP-GM, GP-GC, GW-GM, GW-GC)
- SILTY GRAVEL, CLAYEY GRAVEL (GM, GC, GC-GM)
- SILTSTONE
- CLAYSTONE
- FILL
- —?— BEDROCK CONTACT, queried where uncertian

REV.	DATE	BY	SUB	APP	DESCRIPTION
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DESIGNED BY:

DRAWN BY:

CHECKED BY:

IN CHARGE:

DATE:

Metropolitan Transportation Authority

METRO RED LINE

GeoTransit Consultants

SUBMITTED:

METRO RED LINE
EASTSIDE EXTENSION
PLAN AND PROFILE – CL TRACK

PLATE 5

CONTRACT NO.

DRAWING NO:

REV.

SCALE: HORIZ: 1"=80' VERT: 1"=20'

SHEET NO 2 OF 2

c:\metro\MET-1-L.dwg view=SHT-2L