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GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION EASTSIDE LRT PROJECT – UNDERGROUND SEGMENT

VOLUME I OF III

ALONG FIRST STREET, FROM CLARENCE STREET TO LORENA STREET
LOS ANGELES, CALIFORNIA

Prepared for:

EASTSIDE LRT PARTNERS

Los Angeles, California

October 22, 2002

Project 70131-1-0138.0005

LAW

RESOURCES CREATING SOLUTIONS

October 22, 2002

Mr. Sam Mayman
Project Manager
Eastside LRT Partners
707 Wilshire Boulevard, Suite 2900
Los Angeles, California 90017

Subject: **Geotechnical and Environmental Investigation**
Eastside LRT Project – Underground Segment
Along First Street, from Clarence Street to Lorena Street
Los Angeles, California
Law/Crandall Project 70131-1-0138.0005

Dear Mr. Mayman:

We are pleased to submit the results of our geotechnical and environmental investigation for the underground segment of the Eastside Light Rail Transit (LRT) project. The underground segment is planned to extend along First Street, roughly from Clarence Street to Lorena Street in Los Angeles, California. This investigation was conducted in general accordance with our proposals addressed to Group Delta Consultants dated June 25, 2001, December 5, 2001, February 21, 2002, May 5, 2002, June 14, 2002, and October 7, 2002.

We have previously submitted a draft report dated July 29, 2001, a preliminary report dated August 9, 2001, a draft final report dated February 11, 2002, and a final report dated April 4, 2002. We have subsequently had numerous meetings with you and with other project design consultants to discuss geotechnical design issues affecting the project. The report enclosed herewith addresses the comments and discussions from those meetings as well as incorporating additions and corrections to our prior report and results of supplementary explorations and laboratory testing. The report enclosed herewith supercedes all previous versions.

The scope of our services was planned with you and Ms. Amanda Elioff of Eastside LRT Partners. Our understanding of the structural features of the underground segment of the proposed light rail transit project is based on design drawings by Mr. Frank Fortunato of Eastside LRT Partners.

The results of our investigation and design recommendations are presented in this report, which is submitted in three volumes. Volume I contains the main text and plates. Volume II contains Appendices A through E. Volume III contains Appendices F through J.

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It has been a pleasure to be of professional service to you. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

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A Division of LAW Engineering and Environmental Services, Inc.

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Attachments

cc: Eastside LRT Partners
Attn: Ms. Amanda Elioff

Group Delta Consultants
Attn: Mr. Shah Ghanbari

GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION
EASTSIDE LRT PROJECT – UNDERGROUND SEGMENT

VOLUME I OF III

FIRST STREET, FROM CLARENCE STREET TO LORENA STREET
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Prepared for:

EASTSIDE LRT PARTNERS

Los Angeles, California

by

Law/Crandall, A Division of Law Engineering and Environmental Services, Inc.

Los Angeles, California

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TABLE OF CONTENTS

Volume I of III

LIST OF TABLES, FIGURES, AND PLATES

Volume II of III

Volume III of III

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical and environmental investigation for the underground segment of the planned Eastside Light Rail Transit (LRT) project (Underground Segment). The Eastside LRT project is a proposed extension of the Pasadena Gold Line (currently under construction). The proposed underground structures include two portal structures, two cut-and-cover tunnel sections, two bored twin-tunnel sections, six cross-passages and two sump structures between the bored tunnels, and two stations.

The primary purposes of this investigation were to evaluate the geotechnical and environmental conditions along the Underground Segment and to obtain geotechnical data for planning and design. The locations, dimensions, and configuration of the proposed portal structures, tunnels, and stations in this report are based on available plans and profiles supplied by Eastside LRT Partners at the time of this investigation.

1.1 PROJECT DESCRIPTION

The Underground Segment is approximately 9,100 feet in length, Station (STA) 179+75 to STA 270+87, and is presented on Plate 1a, Plot Plan – Boring Locations. The twin tracks for the Eastside LRT are referred to as East Bound (EB) and West Bound (WB). In this report, only station numbers and elevations for the EB track are used.

More detailed plans of the Underground Segment with corresponding geologic profiles are shown on Plates 2.1 through 2.12, Project Alignment and Geologic Profile. The legend for Plates 2.1b through 2.12b is shown on Plate 2.13, Legend for Geologic Profile. The proposed project alignment for the Underground Segment is discussed in more detail in Section 2.1.1, Underground Alignment. Descriptions of the proposed underground structures are presented in Section 2.1.2, Underground Structures.

1.2 GEOLOGIC SETTING

The proposed Eastside LRT project is located along the southern flank of the Repetto Hills area of the Los Angeles Basin. The subsurface materials encountered within design depths of the proposed underground structures consist of alluvial deposits of Holocene and Pleistocene age comprised of interlayered fine-grained and coarse-grained deposits. These alluvial deposits are underlain by Tertiary age highly weathered bedrock units of the Fernando Formation consisting of claystone and siltstone.

A linear topographic escarpment impacts the project alignment in the area of and including the First/Soto Station. The results and details of a project-specific-investigation performed by Earth Consultants International, Inc. (ECI) to delineate and characterize the escarpment (Coyote Pass Escarpment) in this location and to assess its seismic capability are presented in a separate report (Earth Consultants International, Inc., 2001). The results indicate that the Coyote Pass Escarpment is an active buried thrust structure consisting of folded alluvial deposits and underlying sedimentary bedrock. Its potential for movement should be considered in the design and construction of project structures immediately adjacent to the escarpment (First/Soto Station) and at the projected escarpment crossings. Estimated strains during characteristic events are presented in the ECI report.

1.3 SUBSURFACE MATERIALS

The planned tunnel and station excavations are expected to be virtually entirely within alluvium. The alluvium consists of fine-grained soils interbedded with coarse-grained soils. The fine-grained soils in the project area consist predominantly of stiff to hard low plasticity clay and silt. The coarse-grained soils consist predominantly of dense to very dense silty sand and clayey sand, some gravel layers, and locally cobbles (up to 12 inches in size). Although cobbles were encountered at only two locations in the current and prior explorations at depths impacting the project, additional cobbly zones may exist between boring locations. Boulders (particles greater than 12 inches in diameter) were not encountered in the borings drilled to date along the Underground Segment. Although unlikely, boulders may be encountered between boring locations.

Highly weathered bedrock units of the Fernando Formation underlie the alluvium. Within the project area, the bedrock materials were generally below the tunnel and station inverts. Where encountered, the bedrock consists predominantly of soft to very soft claystone and siltstone (as defined by the Engineering Geology Field Manual, U.S. Department of the Interior, Bureau of Reclamation).

1.4 GROUNDWATER CONDITIONS

The measured groundwater levels along the tunnel sections and portals are generally from approximately 24 to 85 feet below ground surface (BGS). Assuming the groundwater levels during construction are the same as the most recently measured data, the portion of the tunnels approximately between STA 194+00 and STA 256+50 (Plates 2.4b through 2.9b) will be partly or entirely below the groundwater levels.

The groundwater levels measured at the First/Boyle Station area were 50 to 60 feet BGS, which is at about the station invert. Groundwater level measurements in borings and monitoring wells at the First/Soto Station indicate that the current groundwater level at the station is at about 24 feet BGS, about 32 feet above the bottom of the proposed station structure. Groundwater levels measured at the East Portal area (near First Street and Lorena Street) are deeper than 40 feet below the portal invert and are expected to range from about 80 feet BGS at the west end of the portal to about 60 feet BGS at the east end.

1.5 GEOLOGIC-SEISMIC HAZARDS

The geologic and seismic hazards impacting the project were evaluated. Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath the Underground Segment. In our opinion, the potential for surface fault rupture along the Underground Segment due to fault plane displacement propagating to the ground surface during the design life of the project is low.

Although the project site could be subjected to strong ground shaking in the event of an earthquake, the hazards associated with ground shaking are common in seismically active Southern California and its effects can be mitigated by proper engineering design and construction.

As discussed in Section 1.2, Geologic Setting, there is a potential for ground deformation along portions of the alignment crossing and adjacent to the Coyote Pass Escarpment (Earth Consultants International, Inc., 2001). We understand that the potential strains induced on segments of the project impacted by the escarpment will be addressed in the structural design.

1.6 ENVIRONMENTAL CONSIDERATIONS

1.6.1 Soil Assessment

Based on the soil samples tested, the majority of the soils that are to be excavated as part of the construction of the Underground Segment are not expected to be classified as hazardous. In the samples in which metals were detected, concentrations were within naturally occurring levels. The TPH and VOC concentrations detected in samples were below levels requiring remedial action. However, stockpiles of excavated soils may need to be tested prior to use as compacted fill. Soils impacted by contamination cannot be used as compacted fill without specific regulatory agency approval.

Considering that a portion of the Underground Segment traverses the Boyle Heights Oil Field, the possibility of encountering naturally occurring petroleum deposits remains, especially between boring locations. Although considered non-hazardous, excavated soils containing these naturally occurring petroleum deposits typically require special management and handling. These materials will likely not be reusable as fill.

1.6.2 Groundwater Assessment

Contaminated groundwater is not expected to impact the project beyond potentially special requirements for storage, treatment, or disposal of groundwater extracted during the planned dewatering of the First/Soto Station excavation. The TPH and VOC concentrations detected in the groundwater sampled from Monitoring Wells L-5, L-17, and HE-31 may be within discharge limitations to allow disposal into the sanitary sewer system. However, considering that a significant volume of groundwater will likely be discharged, a National Pollutant Discharge Elimination System (NPDES) permit for disposal into the storm drain system may be necessary. A pre-discharge treatment system will likely be necessary for disposal of extracted groundwater into the storm drain system.

Dewatering at the First/Soto Station excavation may affect the shape and extent of existing contamination plume(s). Further investigation of the plume(s) may help better characterize sources, extent, and direction of migration. The information will be useful in reducing potential claims from property owners in the impacted area.

1.6.3 Soil Gas Condition

Although the investigation did not detect wide-spread gaseous conditions, changes in the groundwater conditions and barometric (atmospheric) pressure, may cause the groundwater to off-gas both hydrogen sulfide and methane gases. Furthermore, considering that a portion of the Underground Segment traverses the Boyle Heights Oil Field, the potential of encountering naturally occurring hydrogen sulfide and methane gas remains. Accordingly, implementing monitoring systems and mitigation measures such as proper ventilation against potentially explosive levels of methane gas for construction and operation seems appropriate.

1.6.4 Environmental Summary

Although our investigation did not encounter widespread contamination due to the historical and current land usage, isolated pockets of soil, groundwater, and gas contamination, in addition to those already identified, may be encountered during construction. Any subsurface explorations or construction activities will require prior planning to address the above mentioned environmental conditions. Ultimately, the State of California Occupational Safety and Health Administration (Cal OSHA) will designate the tunnel classification.

The majority of the soils that are to be excavated as part of the construction of the Underground Segment are not expected to be classified as hazardous. However, excavated soils may need to be tested prior to use as compacted fill. Soils impacted by contamination cannot be used as compacted fill without specific regulatory agency approval.

A pre-discharge treatment system will likely be necessary for disposal of groundwater extracted during dewatering of the First/Soto Station excavation into the storm drain system.

1.7 SEISMIC DESIGN CRITERIA

A ground motion study was performed as part of the current investigation to develop site-specific response spectra and to evaluate ground-motion-induced racking values for the stations. The results indicate that the expected peak ground accelerations (PGAs) along the Underground Segment for the Operating Design Earthquake (ODE) and the Maximum Design Earthquake (MDE) are 0.41g and 0.79g, respectively. The peak ground velocity (PGV) for the ODE and MDE are 1.5 feet per second and 3.2 feet per second, respectively. The vertical components of the PGA and PGV may be assumed to be two-thirds of the horizontal values.

Based on a comparison of the results for the planned stations and the East Portal, it is recommended that underground box structures (stations, portals, and cut-and-cover tunnel sections) be designed using the same racking parameters. Racking is defined as the relative displacement between the top and bottom of a structure. For the ODE, the racking may be taken as ¼ inch in 45 feet of height difference, and for the MDE, the racking may be taken as 1 inch in 45 feet. The racking value may be linearly interpolated and extrapolated for other station heights.

1.8 GEOTECHNICAL DESIGN RECOMMENDATIONS

1.8.1 Bored Tunnel Sections, Cross-Passages, and Sump Structures

As shown in the geologic profile presented on Plates 2.1b through 2.12b, Geologic Profile, the bored tunnels, cross-passages, and sump structures will be in older alluvium (Qoa). These tunnel structures traverse both fine-grained and coarse-grained soils. Although the majority of coarse-grained soils encountered within the planned tunnel excavations have significant fines content, some sand and gravel layers with low fines content were encountered. These layers may be susceptible to raveling, running, or flowing conditions.

The current subsurface explorations did not encounter particles larger than 3 inches (cobbles) within the planned tunnel bore. An obstruction interpreted as either a boulder or large cobble was encountered within the planned tunnel bore in an 8-inch-diameter hollow-stem auger boring (FL-40) performed as part of a previous investigation for the Suspended Project (GeoTransit Consultants, 1996e). The log of Boring FL-40 is shown in Appendix B, Prior Subsurface Explorations. Boring FL-40 is located east of the First/Boyle Station.

1.8.1.1 Excavation Considerations

Several existing conditions along the project alignment may impact the tunnel excavation techniques, face stability, advance rates, and potential ground loss. These conditions include: shallow groundwater conditions in alluvium within or above the tunnel bore; raveling and running/flowing conditions in coarse-grained alluvium; local presence of gravels, cobbles, and possibly boulders; and shallow soil overburden above the tunnel crown in portions of the tunnel sections. These conditions should be taken into consideration in tunnel design and construction.

1.8.1.2 Groundwater Control

Assuming groundwater levels during construction are the same as the most recent measured groundwater levels from this investigation, the portion of the bored tunnel sections approximately between STA 194+00 and STA 256+50 (Plates 2.4b through 2.9b) will likely be fully or partially below the groundwater level. All cross-passages and sump structures will be below the groundwater level except for Cross-Passage No. 6 at STA 254+25.

We understand that because a pressure-face tunnel boring machine and precast, gasketed reinforced concrete liners are planned for excavation of the bored tunnel sections, dewatering will not be required for the tunnel. Dewatering may be required for the construction of cross-passages and sump structures below the groundwater level. These structures will probably be excavated using conventional mining methods.

1.8.1.3 Liner Design

Geotechnical parameters for the design of tunnel, cross-passage, and sump structure liners are presented in Table 8.5-1, Design Geotechnical Parameters.

1.8.2 Portals, Cut-and-Cover Tunnel Sections, and Stations

1.8.2.1 Excavation Methods

The excavations will be in alluvium. It is anticipated that the excavations can be achieved by conventional methods. Some of the coarse-grained layers are expected to run or ravel readily. Thus, timely application of ground support will be essential.

Based on the results of the current and prior investigations, it is unlikely that a significant volume of cobbles will be encountered in the planned excavations. Cobbles were not encountered within design depths of the portals or cut-and-cover tunnel sections. Cobbles were encountered in only one current or prior boring within design depths of underground structures. An approximately 2-

foot-thick layer of gravel with cobbles was encountered in Boring HE-30, which is at the First/Sotó Station. The log of Boring HE-30 is shown in Appendix A, Subsurface Explorations.

1.8.2.2 Dewatering and Groundwater Control

The groundwater level at the First/Boyle Station is expected to be at the station invert. If collection and pumping of groundwater inflow into the excavation is deemed to be inadequate, preconstruction and construction dewatering may be required. Dewatering will be required for the construction of the First/Soto Station. The groundwater level will probably be lowered and maintained about 35 to 40 feet below current levels. Dewatering is not expected to be required for the construction of the portal structures.

1.8.2.3 Foundation Recommendations

The natural soils at and below the planned excavation levels are dense and stiff, and the planned structures may be supported on spread-type footings or mat foundations established in the dense and stiff natural soils exposed at the bottom of the planned excavations. The design geotechnical parameters for station, mezzanine, cut-and-cover tunnel, and portal structures are summarized in Tables 8.6-1a through 8.6-1c, Design Geotechnical Parameters.

1.8.3 Protection of Existing Facilities

The tunnels, portal structures, and stations are located within existing residential and commercial areas with 1- and 2-story buildings and aboveground and underground infrastructure, including two freeway overpasses. The tunnels and stations are generally within the public right-of-way. There will, however, be a number of structures within the zone of influence of the tunnel and station excavations.

Although proper design and careful installation of shoring can typically mitigate potential disturbances to structures adjacent to excavations, the need for additional protection measures will ultimately depend on excavation methods, existing building foundations, and whether the buildings can satisfactorily accommodate the expected settlement due to excavation-related deformation.

Existing structures adjacent to the First/Soto Station may undergo settlement due to dewatering. The settlement is estimated to decrease linearly from about 1 inch adjacent to the excavation perimeter to zero at a distance of about 300 feet away, which corresponds to a deflection ratio of 1/3600 (differential settlement/horizontal distance).

Jacobs Associates is currently performing a survey of existing buildings and infrastructure likely to be impacted by the project to help identify structures that may require the implementation of additional protection measures.

2.1 PROJECT DESCRIPTION

2.1.1 Underground Alignment

The Underground Segment is approximately 9,400 feet in length, from about Station (STA) 179+75 to STA 270+87, and is presented on Plate 1a, Plot Plan – Boring Locations. More detailed plans of the Underground Segment are presented on Plates 2.1a through 2.12a, Alignment and Plot Plan, with corresponding geologic profiles presented on Plates 2.1b through 2.12b, Geologic Profile. The legend for Plates 2.1b through 2.12b is shown on Plate 2.13, Legend for Geologic Profile. The planned underground structures are listed in Table 2.1-1, Proposed Underground Structures.

Table 2.1-1: Proposed Underground Structures

Structure	STA (approx.)	Plate No.
Portals		
West Portal	179+75 to 182+23	2.2
East Portal	267+97 to 270+87	2.11
Cut-and-Cover Tunnels		
West CC Tunnel	182+23 to 186+95	2.2, 2.3
East CC Tunnel	264+34 to 267+97	2.11
Stations		
First/Boyle	186+95 to 191+17	2.3
First/Soto	219+65 to 223+09	2.6
Twin Bored Tunnels		
Western Reach	191+17 to 219+65	2.3 to 2.6
Eastern Reach	223+09 to 264+34	2.6 to 2.11
Cross-Passages		
No. 1	200+00	2.4
No. 2	207+50	2.5
No. 3	231+75	2.7
No. 4	239+25	2.8
No. 5	246+75	2.9
No. 6	254+25	2.9
Sump Structures		
No. 1	208+61	2.5
No. 2	245+54	2.9

The Underground Segment begins at the western portal (West Portal) approximately 35 feet east of the centerline of Gless Street. The portal structure extends east under First Street for approximately 248 feet. A cut-and-cover tunnel section (West CC Tunnel) extends east under First Street from the portal structure and under US 101, which is elevated across First Street, for an approximate distance of 472 feet to a station at Boyle Avenue (First/Boyle Station). The First/Boyle Station is approximately 422 feet in total length.

A bored tunnel section (Western Reach) extends east from the First/Boyle Station beyond the intersection with Bailey Street. As shown on Plate 1a and Plate 2.4a, whereas First Street trends southeast beginning at the intersection with Bailey Street, the tunnel section continues east away from First Street for a few hundred feet before curving southeast to merge with First Street just before Interstate 5. The bored tunnel section continues generally under First Street to a station at Soto Street (First/Soto Station) except between Chicago Street and Breed Street where First Street's offset to the north is more abrupt than the tunnel section's curves. The Western Reach of the Twin Bored Tunnels is approximately 2,848 feet.

The First/Soto Station is approximately 344 feet in total length. Another bored tunnel section (Eastern Reach) begins from the First/Soto Station and extends along First Street to a cut-and-cover tunnel section (East CC Tunnel). The Eastern Reach of the Twin Bored Tunnels is approximately 4,125 feet long. The East CC Tunnel extends east for a distance of about 363 feet to the East Portal, which is approximately 290 feet in total length.

Cross-passages and sump structures are planned at selected locations along the bored tunnel sections listed in Table 2.1-1.

2.1.2 Underground Structures

Most of the information presented in this section is based on information provided to us by Eastside LRT Partners. The Elevations referenced in this report are also based on the aforementioned drawings. Elevations (EL) are presented in feet.

2.1.2.1 Portals

The West Portal is approximately 248 feet in length, increasing in width from 30 feet at the west end to 35 feet at the east end. The portal structure will consist of twin reinforced-concrete retaining walls that are cantilevered at the shallower western portion and braced with struts at the deeper eastern portion. The height of retained earth is expected to be as high as 15 feet at the cantilevered portion and as high as 27 feet at the braced portion.

The proposed East Portal will be approximately 290 feet long and vary in width from about 60 feet at the western end to about 50 feet at the eastern end. Similarly to the western portal, twin cantilevered reinforced-concrete retaining walls are planned for the shallower eastern portion and twin braced reinforced-concrete retaining walls are planned for the deeper western portion. The height of retained earth is expected to range from approximately 26 feet at the western end tapering to essentially zero at the eastern end.

Similar foundation systems are planned for both portals. Within the cantilevered section, the retaining walls and center columns supporting electrical guides are to be supported on independent strip footings. The tracks are to be supported on grade. Within the braced section, the retaining walls, electrical guide columns, and tracks are to be supported on a mat foundation.

2.1.2.2 Cut-and-Cover Tunnel Sections

The cut-and-cover (CC) tunnel sections are planned as a reinforced-concrete box structures. The West CC Tunnel will be approximately 472 feet long and range in width from approximately 35 feet at the west end to approximately 45 feet adjacent to the First/Boyle Station. The tunnel section height will range from 23 feet at the west end to approximately 30 feet toward the east end. The tunnel invert depth will range from approximately 26 feet BGS (EL 252) at the west end to approximately 50 feet BGS (EL 257) at the east end.

The East CC Tunnel will be approximately 363 feet long and range in width from approximately 75 feet at the west end to approximately 60 feet adjacent to the East Portal. The tunnel section height will range from 45 feet at the west end to approximately 22 feet at the east end. The tunnel invert depth will range from approximately 56 feet BGS (EL 265) at the west end to approximately 26 feet BGS (EL 287) at the east end.

2.1.2.3 Stations

The two underground stations proposed at First/Boyle and First/Soto are planned as reinforced-concrete box structures constructed using cut-and-cover methods of construction. These box structures will be supported on mat foundations.

The main station box at First/Boyle will consist of two underground levels, a mezzanine (upper) level above a platform (lower) level. The mezzanine level will have a north wing entrance (without a lower level) that provides access to the station. The bottom of the main station box will range in depth from approximately 50 feet BGS (EL 255) at the west end to approximately 57 feet BGS (EL 258) at the east end. The bottom of the north wing of the mezzanine level will be approximately 35 feet BGS (EL 278). In plan view, the approximately 422-foot-long main station box at First/Boyle will be oblong with widths varying from approximately 45 feet at the west end to approximately 66 feet at the east end. The main station box will be about 45 feet in height.

The main station box at First/Soto will be rectangular in plan view, with two underground levels, a mezzanine level above a platform level, similar to the First/Boyle Station. The mezzanine level will have a south wing entrance (without a lower level) that provides access to the station. The main station box will be approximately 344 feet long, 55 to 60 feet wide, and 45 feet high. The bottom of the station will be approximately 56 feet BGS (EL 238).

The bottom of the main station box at First/Soto will be approximately 56 feet BGS (EL 238). The bottom of the south wing of the mezzanine level will be approximately 34 feet BGS (EL 261). The width of the main station box will range from approximately 60 feet through most of its length to approximately 70 feet at the east and west ends.

2.1.2.4 Bored Tunnel Sections

The proposed tunnel sections consist of two single-track, 19-foot inside diameter openings in a double line configuration. The tunnel liners will consist of precast concrete sections, except possibly for 20- to 25- foot-long sections on each end the of First/Soto Station that may have a steel liner to help mitigate the effects of seismically-induced earth deformations.

The tunnel invert depth will be roughly 55 feet below ground surface (BGS) at EL 259 adjacent to the First/Boyle Station and plunge as deep as 85 feet BGS (EL 216) between Cummings and Saint Louis Street before rising to approximately 55 feet BGS (EL 239) adjacent to the First/Soto Station. East of the First/Soto Station, the tunnel invert will start at approximately 55 feet BGS (EL 239) and descend to approximately EL 235 between Mott Street and Savannah Street where the rising ground surface will be approximately 85 feet above the tunnel invert. The tunnel invert then will rise to approximately 53 feet BGS (EL 269) adjacent to the East Portal.

2.1.2.5 Cross-Passages

As listed in Table 2.1-1, cross-passages and sump structures are planned at selected locations along the tunnel sections. Cross-passages are horseshoe-shaped passageways connecting the parallel EB and WB bored tunnels to provide cross-tunnel access for utilities, maintenance personnel, and in case of emergencies. Each cross-passage will be approximately 13½ feet wide and 14 feet high and consist of a 3-foot-thick concrete mat, 2-foot-thick concrete walls, and a 2-foot-thick crown. The base of the cross-passages are planned to be slightly above the adjacent tunnel invert.

2.1.2.6 Sump Structures

Sump structures are in essence cross-passages with an approximately 14-foot-deep, 8-foot-diameter sump pit in the middle of the passageway. Surface runoff water collected from the bored tunnels will be collected in catch basins located at the centerline of each bored tunnel adjacent to the two sump structures and routed to the sump pits via drain pipes connecting the catch basins and sump pits.

2.2 OBJECTIVE AND SCOPE OF WORK

The objective of the current geotechnical investigation was to evaluate subsurface soil and groundwater conditions and to obtain geotechnical data for planning and design of the Underground Segment. A preliminary geotechnical investigation for the at-grade segments of the Eastside LRT project has been performed by Group Delta Consultants.

Part of the subsurface exploration program for the current phase of the project was performed by Harding ESE. Harding ESE performed the HE-series borings. Harding ESE also performed the environmental testing of the gas, water, and soil samples obtained from the HE-series borings as well as the initial set of readings of groundwater wells and soil gas probes. The field exploration and laboratory testing program for the Underground Segment is presented in Section 3.0, Reconnaissance, Exploration, and Laboratory Testing.

Law/Crandall's scope of work for the current phase of the project consisted of the following:

- review of available literature and reports regarding the geologic, geotechnical, groundwater and seismic conditions for the Underground Segment;
- subsurface explorations and installation of gas and groundwater monitoring wells in selected borings;
- geotechnical and environmental laboratory testing;
- providing engineering and field support for the subsurface exploration and environmental laboratory testing program by Harding ESE;
- supplementary monitoring of groundwater and soil gas wells installed by Law/Crandall, Harding ESE and a few other available wells installed by others;
- evaluation of soil and groundwater contamination based on field and laboratory environmental testing performed by Law/Crandall and others;
- evaluation of geologic-seismic hazards impacting the project;
- performance of a ground motion study;

- performance of aquifer tests to evaluate the hydraulic conductivity characteristics of alluvium encountered below the groundwater level at the First/Boyle Station and the First/Soto Station and to estimate water inflows and dewatering needs;

- evaluation of the environmental and geotechnical data to develop design recommendations;

- performance of a corrosion study; and

- preparation of this report.

The results of the above mentioned aquifer tests are presented in a separate report (Law/Crandall, 2002). Under a separate subcontract with Harding ESE, Law/Crandall performed the following tasks under the subcontract with Harding ESE:

- performance of a geotechnical laboratory testing program on selected representative soil and groundwater samples from the HE-series borings to assess the environmental and engineering properties of subsurface materials; and

- performance of a total of 15 pressuremeter tests, five each at three boring locations.

2.3 OTHER AVAILABLE DATA

A significant number of prior geotechnical and environmental investigations were performed in the project area. The results of prior investigations conducted by Law/Crandall, GeoTransit Consultants, Woodward-Clyde Consultants, and Engineering-Science, Inc., for the suspended Metro Red Line Eastside Extension project were compiled and reviewed as part of this investigation. These reports are listed below.

- Engineering-Science, Inc., 1994, "Subsurface Gas Investigation, Metro Red Line – Eastside Extension, Report Prepared for Engineering Management Consultant," dated December 7, 1994.

- Enviro-Rail, 1998a, "Environmental Summary Report, Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California," dated June 9, 1998.

- Enviro-Rail, 1998b, "Stage 2 Supplemental Gas Study (Draft), Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California, Volumes I and II," dated April, 1998.

- Enviro-Rail, 1998c, "Stage 1 Supplemental Gas Investigation, Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California, Volumes I and II," dated March, 1998.

- Enviro-Rail, 1997a, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0531 – Chavez/Soto Station and Contract Unit C0541 – Line Section First/Boyle to First/Lorena," dated May, 1997.

- Enviro-Rail, 1997b, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0521 – 1st/Boyle Station," dated May, 1997.

- Enviro-Rail. 1997c, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0502 – Line Section Union Station to 1st/Boyle and Little Tokyo Station, Volumes I and II," dated May, 1997.

- Enviro-Rail, 1995, "Baseline Environmental Study, Metro Red Line Segment 3, Eastside Extension," dated October, 1995.

- GeoTransit Consultants, 1996a, "Final Geotechnical Investigation for: First/Boyle Station with Crossover and Little Tokyo – First/Boyle Tunnels, Eastside Extension Metro Red Line Project, Prepared for Engineering Management Consultant," dated June 1996 (Project No. 95-8347-24).

- GeoTransit Consultants, 1996b, "Final Geotechnical Investigation for: Chavez/Soto Station & Adjacent Tunnels, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated June 1996 (Project No. 95-8347-24).

- GeoTransit Consultants, 1996c, "Results of Aquifer Pump Testing, At Mission Road and E. Fourth Street, Eastside Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated May 15, 1996 (Project No. 95-8347).

- GeoTransit Consultants, 1996d, "Final Geotechnical Investigation for: First/Lorena Station & Tail Track Tunnels, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated May 1996 (Project No. 95-8347-15).

- GeoTransit Consultants, 1996e, "Draft Final Fault Investigation for: Engineering Design Program, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated February 8, 1996 (Project No. 95-8347-18).

- GeoTransit Consultants, 1995, "Final Draft Geotechnical Investigation for: Union-Little Tokyo Tunnels & Little Tokyo Station, East Side Extension,

Metro Red Line Project, Volume I of II," dated December 1995 (Project No. 95-8347-04).

- GeoTransit Consultants, 1994a, "Geotechnical Investigation for: Preliminary Engineering Program, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated February 14, 1994 (Project No. 94-1100).

- GeoTransit Consultants, 1994b, "Stage II Environmental Site Assessment, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated January 1994 (Project No. 94-1109-02).

- Law/Crandall, 2000, "Report of Environmental Assessment–Metro Red Line Eastside Transit Corridor Study Area, Prepared for Eastside Corridor Transit Consultants," dated May 2000 (Project No. 70131-9-0387).

- Law/Crandall, 1997a, "Geotechnical Investigation Data Report Pressuremeter Testing and Soil Velocity Measurements, East Side Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated April 9, 1997 (Project No. 70131-9-60552.5000).

- Law/Crandall, 1997b, "Geotechnical Investigation Report–Large Diameter Borings, East Side Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated February 18, 1997 (Project No. 70131-6-0552.1000).

- Woodward-Clyde Consultants, 1997, "East Side Extension Stations, Seismic Hazard Evaluation, Los Angeles Metro Rail System, Los Angeles, California, Prepared for Engineering Management Consultant," dated April 10, 1997 (Project No. 964G152).

The locations of borings from the prior investigations referenced above are presented on Plate 1a and on Plates 2.1a through 2.12a. Logs of prior borings used in the current investigation are presented in Appendix B, Prior Subsurface Explorations.

In addition to the project-specific documents referenced above, we have reviewed applicable geologic and environmental references in the literature as well as prior Law/Crandall projects in the general vicinity of the project alignment. These documents are cited within the text and full references are provided in Section 9.0, Bibliography.

2.4 LIMITATIONS OF INVESTIGATION

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Eastside LRT Partners and their design consultants to be used solely in the design of the proposed Eastside Light Rail Transit Project. The report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

In developing the interpretations and recommendations presented in this report, Law/Crandall relied partly on subsurface information obtained by Harding ESE and GeoTransit Consultants in the area covered by this report. 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, surveys, or other explorations were made by Law/Crandall, Harding ESE, GeoTransit Consultants, Engineering Science, Inc., and Enviro-Rail.

3.0 RECONNAISSANCE, EXPLORATION, AND LABORATORY TESTING

This section provides a description of the subsurface exploration, and field and laboratory testing performed for this project. Applicable results from prior investigations were also used in developing the conclusions and recommendations presented in this report.

3.1 GEOLOGIC RECONNAISSANCE

Previous investigations by GeoTransit Consultants (1996a, 1996b, 1996d, 1996e) for the suspended Metro Red Line Eastside Extension project (Suspended Project) include development of local geologic maps of the area of the current Eastside LRT project alignment. These geologic maps are primarily based on extensive investigations by Bullard and Lettis (1993) that included field mapping and review of historic topographic maps and aerial photography to establish a Quaternary stratigraphy for the region and define the style and pattern of Quaternary deformation in the northeastern Los Angeles Basin. Details from mapping by Lamar (1970) are also included for areas north of the alignment. The degree of urbanization and cultural cover in the alignment corridor makes use of the historic record in determining the geology of the area essential.

A reconnaissance of the Underground Segment was made of observable topographic and geologic features to field check the geologic conditions presented in previous reports. Topographic lows at First/Lorena, First/Rivera, First/Matthews west to Chicago Street, and First/Boyle west to Utah Street identify the approximate locations of younger alluvial deposits in channels that dissect the older alluvial deposits. These channels are culturally modified and concealed as a result of urban development. The topographic relationship between younger and older alluvial deposits is also apparent in the Geologic Profile for the Underground Segment (Plates 2.1b through 2.12b, Geologic Profile) – topographic highs primarily expose uplifted Pleistocene age materials, topographic lows contain younger, Holocene age deposits. Description and age designation of materials encountered in geotechnical borings in the alignment were compared with the geology adapted by GeoTransit Consultants (1996a, 1996b, 1996d, 1996e), and generally agree with units presented in the geologic map.

The lack of surface exposure due to urban cover and the difficulty of differentiating the alluvial deposits based on outcrop appearance alone (without the larger geomorphic perspective of historic topography and aerial photography) necessitate adoption of previous work. Bullard and Lettis (1993) performed a thorough investigation of the geology of the area that includes the Underground Segment. In our opinion, adoption of the work by GeoTransit Consultants for descriptive purposes is appropriate. A more detailed review of previous work by Bullard and Lettis (1993) and Lamar (1970) was not deemed necessary for the current project.

3.2 SUBSURFACE EXPLORATIONS

3.2.1 Borings

The subsurface conditions along the Underground Segment were explored for the current study by drilling 43 borings at the locations shown on Plate 1a, Plot Plan – Boring Locations, and on Plates 2.1a through 2.12a, Alignment and Plot Plan. The borings were drilled to depths of 32½ feet to 121 feet.

In addition to the borings drilled for the current study, logs of prior borings were available from prior investigations for the Suspended Project. Borings for the current study drilled by Law/Crandall have a prefix of L-. Borings for the current study drilled by Harding ESE have a prefix of "HE-." Prior borings for the Suspended Project were designated the following prefixes: "PB-," "B-," "FL-," "DD-," "SD-," and "PE-."

Twenty-eight borings (L-1 through L-13, L-16, L-17, L-18, L-19, L-23, L-24, HE-8, HE-9, HE-10, HE-10A, HE-11, HE-13, HE-14, HE-15, and HE-15A) were drilled along the tunnel and portal sections. As shown on Figures 2.11a and 2.11b, Borings L-23 and L-24 were drilled outside and north of the tunnel and portal envelope to evaluate subsurface conditions for the relocation of an existing retaining wall. Five borings (HE-32, HE-32A, HE-32B, HE-33, and HE-34) were drilled at the First/Boyle Station location. Nine borings (L-14, L-15, L-20, L-21, HE-12, HE-12A, HE-12B, HE-30, and HE-31) were drilled at the First/Soto Station location. Subsurface information from one of the borings (HE-7) to the west of the beginning of the Underground Segment was also used for the current study.

The soils encountered were logged by a representative from Law/Crandall (L-series borings) or Harding ESE (HE-series borings), and undisturbed and Standard Penetration Test (SPT) samples were obtained for laboratory inspection and testing. A split-spoon sampler was used to obtain SPT samples generally every 5 feet. A Crandall sampler (3.187-inch outer diameter and 2.625-inch inner diameter with six 1-inch sample rings) was used to obtain relatively undisturbed samples generally every 10 feet. Continuous core samples were taken in Borings HE-12A and in Boring L-18. After results of laboratory testing became available, the boring field classifications were revised when appropriate.

Undisturbed samples were not collected and SPTs were not performed in Borings L-18, HE10A, HE-12A, HE-12B, and HE-32, which were drilled primarily to collect continuous core samples, install monitoring wells, or to conduct field testing. In addition, SPTs were not performed in Borings HE-15A and HE-32B, which were drilled for field testing (pressuremeter tests and seismic velocity measurements).

The logs of the borings drilled for the current study are presented in Appendix A, Subsurface Explorations; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches are indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were performed in the borings; the results of the tests are indicated on the logs.

In addition to these explorations, we also reviewed copies of geotechnical reports prepared previously by us and by GeoTransit Consultants for the Suspended Project. Nineteen borings from previous investigations drilled to depths of 75 feet to 141 feet were applicable to the current project alignment. Sixteen of the borings (B-2, FL-40, DD-24, DD-25, DD-26, DD-61, PE-13, PE-14, PE-15, SD-6, SD-9, SD-10, SD-15, SD-16, SD-17, SD-18) were drilled along the tunnel and portal sections, and three of the borings (SD-7, SD-8, PE-21) were drilled at the First/Boyle Station location. The logs of borings from previous investigations are presented in Appendix B, Prior Subsurface Explorations.

3.2.2 Cone Penetrometer Tests

In addition to the borings, the subsurface conditions along the Underground Segment were explored for the current study by performing 14 Cone Penetrometer Tests (CPTs) at the locations shown on Plate 1a and on Plates 2.1a through 2.12a. The CPTs were performed to evaluate soil stabilization alternatives at cross-passage and sump pit locations and beneath selected structures that may be potentially impacted during tunneling. CPTs for the current study by Law/Crandall are labeled with a prefix of CPT-.

The CPTs were advanced to depths of 35 to 64½ feet below the existing ground surface. The CPTs were performed by Gregg In Situ, Inc. under observation of a Law/Crandall representative. One of the borings (Boring L-18 was drilled immediately adjacent to CPT-1 to provide correlation data. Details of the CPTs are presented in Appendix A.

3.2.3 Monitoring Well Installation and Groundwater Level Monitoring

Groundwater monitoring wells were installed at 17 boring locations (L-1, L-2, L-3, L-9, L-11, L-20, L-21, HE-10, HE-10A, HE-12, HE-15, HE-30, HE-31, HE-32, HE-32A, HE-33, HE-34) during the current investigation. In addition, 10 monitoring wells from previous investigations (DD-24, DD-25, DD-26, DD-61, EB-12, PE-13, PE-21, SD-6, SD-9, SD-15) were within the current project area. Seven of the 10 wells installed previously were still functioning; DD-26 and DD-61 were not functioning.

Four of the wells from the current investigation (HE-12, HE-12A, HE-30, and HE-31) were installed as nested wells to detect potentially perched water conditions. Well installation diagrams are presented in Appendix C, Monitoring Wells. Groundwater level measurements from these monitoring wells were recorded using an electronic water-level indicator.

A summary of previous and current groundwater records is presented in Table 3.2-1, Monitoring Well Summary. Some interim readings that measured groundwater levels within the range shown in Table 3.2-1 were omitted.

Table 3.2-1: Groundwater Level Summary

Alignment Section	Boring	Installation Date (mm/dd/yy)	Total Well Depth (ft)	Ground Elevation (ft)	Groundwater Depth (ft)	Groundwater Elevation (ft)	Groundwater Date (mm/dd/yy)
Cut-and-Cover Tunnel Section	L-1	11/7/01	60	283.5	ne[1]	-	D.D.[2]
					ne[1]	-	10/10/02
	SD-6	10/27/95	90	295.0	57.8	237.2	11/21/95
					57.8	237.2	09/26/96
					83.6	211.4	10/25/96
					ne[1]	-	05/09/01
					nm[3]	-	06/01/01
					84.1	210.9	02/12/02
					85.8	209.2	10/10/02
First/Boyle Station	HE-32	05/08/01	80	309.8	59.0	250.8	D.D.[2]
					56.0	253.8	05/09/01
					56.1	253.7	06/01/01
					56.8	253.0	01/10/02
					58.0	251.8	10/10/02
	HE-34	05/12/01	80	308.4	60.0	248.4	D.D.[2]
					54.0	254.4	06/01/01
					nm[3]	-	01/10/02
					57.4	251.0	10/10/02
	HE-33	05/10/01	80	309.2	58.0	251.2	D.D.[2]
					55.3	253.9	06/01/01
					nm[3]	-	01/10/02
					56.2	253.0	10/10/02
	PE-21	11/23/93	90	313.0	58.4	254.6	01/11/94
					57.9	255.1	09/25/96
					57.4	255.6	05/09/01
					57.4	255.6	06/01/01
					58.1	254.9	01/10/02
					59.3	253.7	10/10/02

Alignment Section	Boring	Installation Date (mm/dd/yy)	Total Well Depth (ft)	Ground Elevation (ft)	Groundwater		
					Depth (ft)	Elevation (ft)	Date (mm/dd/yy)
Bored Tunnel Section	SD-9	11/10/95	110.4	312.0	56.2	255.8	11/21/95
					55.4	256.6	09/26/96
					55.6	256.4	05/09/01
					nm[3]	-	06/01/01
					57.0	255.0	01/10/02
					59.0	253.0	10/11/02
	L-2	11/9/01	79	303.6	41.4	262.2	D.D.[2]
					40.9	262.7	01/15/02
					42.8	260.8	10/10/02
	HE-10	05/02/01	68	299.2	51.5	247.7	D.D.[2]
					35.6	263.6	05/09/01
					35.7	263.5	06/01/01
					36.4	262.8	1/10/02
					38.0	261.2	10/10/02
	L-5	11/13/01	33	294.1	28.0	266.1	D.D.[2]
					27.2	266.9	12/13/01
					27.2	266.9	01/15/02
					29.2	264.9	10/10/02
	L-17	1/4/02	35	292.1	28.6	263.5	D.D.[2]
					25.0	267.1	01/10/02
					24.9	267.2	01/15/02
					26.9	265.2	10/11/02

Table 3.2-1 (continued): Groundwater Level Summary

Alignment Section	Boring	Installation Date (mm/dd/yy)	Total Well Depth (ft)	Ground Elevation (ft)	Groundwater Depth (ft)	Groundwater Elevation (ft)	Groundwater Date (mm/dd/yy)
First/ Soto Station	L-20	8/19/02	45	294.0	31.0	263.0	D.D.[2]
					28.0	266.0	10/14/02
	HE-31(s)[4]	5/15/01	30	293.3	26.0	267.3	D.D.[2]
					24.1	269.2	06/01/01
					25.4	267.9	01/10/02
					27.2	266.1	10/10/02
	HE-31(d)[5]	5/15/01	55	293.3	48.0	245.3	D.D.[2]
					24.1	269.2	06/01/01
					25.4	267.9	01/10/02
					27.2	266.1	10/10/02
	HE-30(s)	05/18/01	32	293.3	27.0	266.3	D.D.[2]
					24.1	269.2	06/01/01
					25.4	267.9	01/10/02
					27.3	266.0	10/10/02
	HE-30(d)	05/18/01	55	293.3	44.0	249.3	D.D.[2]
					24.0	269.3	06/01/01
					25.3	268.0	01/10/02
					27.3	266.0	10/10/02
	HE-12(s)	04/27/01	31	293.5	25.0	268.5	D.D.[2]
					23.9	269.6	05/09/01
					24.0	269.5	06/01/01
					25.4	268.1	01/10/02
					27.3	266.2	10/11/02
	HE-12(d)	04/27/01	64	293.5	44.0	249.5	D.D.[2]
					24.3	269.2	05/09/01
					24.4	269.1	06/01/01
					25.7	267.8	01/10/02
					27.5	266.0	10/11/02
	HE-12A(s)	04/25/01	34	293.5	20.0	273.5	D.D.[2]
					25.1	268.4	04/26/01
					ne[6]	-	01/10/01
	HE-12A(d)	04/25/01	58	293.5	44.0	249.5	D.D.[2]
					25.1	268.4	04/26/01
					ne[6]	-	01/10/01
	L-21	8/20/02	50	294.5	28.8	265.7	D.D.[2]
					28.4	266.1	10/14/02

Table 3.2-1 (continued): Groundwater Level Summary

Alignment Section	Boring	Installation Date (mm/dd/yy)	Total Well Depth (ft)	Ground Elevation (ft)	Groundwater		
					Depth (ft)	Elevation (ft)	Date (mm/dd/yy)
Bored Tunnel Section	L-9	11/19/01	95	313.8	50.8	263.0	D.D.[2]
					47.4	266.4	12/13/01
					47.5	266.3	01/15/02
					49.8	264.0	10/10/02
	DD-24	11/10/95	90	307.0	47.6	259.4	01/25/96
					48.6	258.4	02/14/96
					48.8	258.2	01/28/98
					47.9	259.1	05/09/01
					47.9	259.1	06/01/01
					48.5	258.5	01/10/02
					50.2	256.8	10/10/02
	HE-15	05/10/01	85	310.8	64.5	246.3	D.D.[2]
					45.4	265.4	05/10/01
					53.4	257.4	06/01/01
					54.0	256.8	01/10/02
					55.9	254.9	10/10/02
	DD-61	11/13/95	89.7	307.0	48.2	258.8	02/14/96
					48.1	258.9	08/07/97
					ne[7]	-	05/09/01
					ne[7]	-	06/01/01
	EB-12A	1/5/94	80	300.0	55.0	245.0	D.D.[2]
					48.0	252.0	01/06/94
					45.1	254.9	01/22/94
					46.3	253.7	01/10/02
	DD-25	11/11/95	88.5	326.0	57.9	268.1	01/25/96
					57.8	268.2	09/23/96
					54.0	272.0	10/25/96
					58.0	268.0	05/09/01
					58.1	267.9	06/01/01
					58.6	267.4	01/10/02
					59.7	266.3	10/10/02
	L-11	11/21/01	85	315.3	73.8	241.5	D.D.[2]
					71.0	244.3	12/13/01
					71.0	244.3	01/10/02
					71.0	244.3	01/15/02
					72.3	243.0	10/10/02
	DD-26	11/17/95	110	323.0	83.3	239.7	01/25/96
					83.1	239.9	02/14/96
					83.7	239.3	08/07/97
					ne[7]	-	05/09/01
					ne[7]	-	06/01/01
					nm[3]	-	01/10/02

Table 3.2-1 (continued): Groundwater Level Summary

Alignment Section	Boring	Installation Date (mm/dd/yy)	Total Well Depth (ft)	Ground Elevation (ft)	Groundwater		
					Depth (ft)	Elevation (ft)	Date (mm/dd/yy)
East Portal	SD-15	08/26/95	110	312.0	79.1	232.9	10/18/95
					78.8	233.2	09/23/96
					79.1	232.9	05/09/01
					79.2	232.8	06/01/01
					80.2	231.8	10/10/02
At-Grade Section	PE-13	11/17/93	65	292.0	ne[1]	-	01/11/94
					ne[1]	-	05/09/01
					ne[1]	-	06/01/01
					ne[1]	-	01/10/02
					ne[1]	-	10/10/02

By: TEA 10/21/02
Chkd: SB 10/21/02

Notes:

1 not encountered
2 measured During Drilling
3 not measured (access to well blocked by parked vehicle)
4 shallow well
5 deep well
6 not encountered (well appears to be malfunctioning)
7 not encountered (full depth of well not accessible due to blockage inside well casing)

3.3 FIELD TESTING

3.3.1 Aquifer Testing

We have performed aquifer tests to evaluate the hydraulic conductivity characteristics of alluvium encountered below the groundwater level at the First/Boyle and First/Soto stations and to estimate water inflows and dewatering needs. The results of the aquifer tests are presented in a separate companion report (Law/Crandall, 2002).

3.3.2 Downhole Seismic Velocity Survey

Downhole suspension velocity surveys were performed by Geovision, Inc. at L-14, L-15, L-16, HE-32B, HE-12B, and HE-15A. The Geovision, Inc. report is presented in Appendix D, Seismic Velocity Surveys. The dynamic moduli derived from the velocity survey results are summarized in Table 3.3-1a, Dynamic Soil Moduli and Poisson's Ratio (Current Investigation).

Agbabian Associates has previously also performed downhole suspension velocity surveys in prior Borings PB-1, PB-2, and PB-3 and submitted the results in a report dated February 12, 1997. This report is also included in Appendix D.

In addition, other prior downhole seismic velocity surveys were performed within Borings DD-15, DD-25, DD-27S, DD-58, SD-6, SD-11, and SD-15. The results of the seismic velocity surveys are presented in Appendix D. The locations of all the borings referenced in this section are presented on Plate 1a. The dynamic moduli derived from the velocity survey results in borings within the Underground Segment are summarized in Table 3.3-1b, Dynamic Soil Moduli and Poisson's Ratio (Previous Investigations).

Table 3.3-1a: Dynamic Soil Moduli and Poisson's Ratio (Current Investigation)

Boring	Depth Range (feet)	Geologic Stratigraphy	Measured Average Shear (S) Wave Velocity (feet/second)	Measured Average Compressional (P) Wave Velocity (feet/second)	Shear Modulus (10^3 ksf*)	Young's Modulus (10^3 ksf)	Bulk Modulus (10^3 ksf)	Poisson's Ratio
HE-12B	0 – 15	Clay	800	1,600	2.48	6.63	6.63	0.33
	15 – 28	Silt and Sand	1,500	3,000	8.73	23.29	23.29	0.33
	28 – 54	Saturated Sand and Silt	1,300	5,500	6.56	19.29	108.68	0.47
	54 – 75	Saturated Clay	1,300	6,300	6.56	19.39	145.33	0.48
HE-15A	8 – 22	Clay	1,700	3,400	11.22	29.92	29.92	0.33
	22 – 36	Clay and Sand	1,300	2,500	6.56	17.25	15.52	0.31
	36 – 45	Sand and Clay	1,300	4,200	6.56	18.99	59.73	0.45
	45 – 63	Saturated Clay and Sand	1,300	5,800	6.56	19.33	121.84	0.47
	63 – 100	Saturated Sand	1,600	5,200	9.94	28.77	91.72	0.45
HE-32B	0 – 12	Clay and Silt	1,100	2,800	4.70	13.23	24.17	0.41
	12 – 26	Silt	1,500	2,600	8.73	21.85	14.60	0.25
	26 – 40	Silt and Sand	1,300	2,200	6.56	16.16	10.04	0.23
	40 – 58	Sand and Clay	1,800	3,200	12.58	31.91	22.98	0.27
	58 – 84	Saturated Sand and Silt	1,500	5,000	8.73	25.34	85.40	0.45
L-14	0 – 12	Clay	1,200	2,500	5.59	15.10	16.81	0.35
	12 – 22	Clay	1,500	3,000	8.73	23.29	23.29	0.33
	22 – 33	Clay and Sand	1,000	2,400	3.88	10.83	17.18	0.39
	33 – 50	Saturated Clay and Sand	1,400	5,600	7.61	22.32	111.59	0.47
	50 – 57	Saturated Sand	800	5,600	2.48	7.40	118.43	0.49
	57 – 106	Saturated Clay and Sand	1,400	6,000	7.61	22.39	129.61	0.47
L-15	0 – 34	Clay, Silt, and Sand	1,200	2,400	5.59	14.91	14.91	0.33
	34 – 72	Saturated Clay	1,000	5,700	3.88	11.52	120.95	0.48
	72 – 82	Saturated Clay and Sand	1,000	6,200	3.88	11.54	144.05	0.49
	82 – 107	Saturated Clay and Sand	1,300	6,200	6.56	19.38	140.48	0.48
L-16	0 – 14	Clay and Silt	700	1,500	1.90	5.18	6.20	0.36
	14 – 25	Clay	1,200	2,800	5.59	15.51	22.98	0.39
	25 – 47	Clay	1,500	3,800	8.73	24.59	44.41	0.41
	47 – 56	Sand	1,900	3,100	14.01	33.61	18.62	0.20
	56 – 76	Saturated Sand and Siltstone	1,600	5,600	9.94	28.93	108.49	0.46
	76 – 98	Saturated Siltstone	1,500	5,400	8.73	25.47	101.55	0.46

*Kips per Square Foot

Table 3.3-1b: Dynamic Soil Moduli and Poisson's Ratio (Previous Investigations*)

Boring	Depth Range (feet)	Geologic Stratigraphy	Measured Average Shear (S) Wave Velocity (feet/second)	Measured Average Compressional (P) Wave Velocity (feet/second)	Shear Modulus (10^3 ksf)	Young's Modulus (10^3 ksf)	Bulk Modulus (10^3 ksf)	Poisson's Ratio
DD-25	0 – 30	Sand	1,310	2,250	6.9	17.3	11.2	0.24
	30 – 57	Sand	1,510	2,700	9.2	23.4	17.2	0.27
	57 –89	Saturated Sand and Silt	1,750	4,500	12.4	34.9	65.4	0.41
SD-6	0 – 40	Sand, Clay, and Silt	1,120	1,770	5.1	11.8	5.9	0.17
	40 – 58	Clay, Silt, and Sand	1,460	3,000	8.6	23.2	24.9	0.35
	58 – 89	Sand, Clay	1,460	5,200	8.6	25.1	97.8	0.46
SD-15	0 – 17	Sand	940	1,750	3.6	9.3	7.6	0.30
	17 – 40	Clay, Silt, Sand, and Gravel	1,560	3,050	9.8	26.0	24.5	0.32
	40 – 110	Sand and Gravel	1,990	5,150	16.0	45.2	85.8	0.41

*Geotransit Consultants (1996a, 1996b)

3.3.3 Pressuremeter Testing

As part of the current investigation, we performed pressuremeter testing in Borings HE-32B (First/Boyle Station), HE-12B (First/Soto Station), and HE-15A (Eastern Reach of Bored Tunnel Section). The test depths were selected based on the following general criteria:

- Three tests at or above the crown and preferably in different soil strata.
- One test near the center of the tunnel.
- One test below the tunnel invert.

The pressuremeter test depths in each boring are presented in Table 3.3-2a, Current Pressuremeter Tests. All pressuremeter testing was performed in general accordance with ASTM D 4719. The Pressuremeter (Menard) Elastic Modulus (E_m) has been calculated based upon the results of pressuremeter testing. In addition, the lower bound of the limit pressure was estimated, and the in-situ total horizontal stress and total horizontal stress coefficient (assuming an average unit weight of 120 pounds per cubic foot) were estimated. Details of the pressuremeter measurements and computations of properties are given in Appendix E, Pressuremeter Test Results. Table 3.3-2a, Current Pressuremeter Tests, summarizes the results.

Table 3.3-2a: Current Pressuremeter Tests

Boring	Test Depth (feet)	Soil Type	Estimated At-Rest Total Horizontal Earth Pressure (ksf)	At Rest Horizontal Pressure Coefficient [Ko]	E_m (ksf)
HE-32B	22	Sandy Silt	2.5	0.9	782
HE-32B	32	Sandy Silt	3.6	0.9	656
HE-32B	42	Silty Sand	3.9	0.7	1046
HE-32B	52	Well-Graded Sand	4.2	0.7	1999
HE-32B	64	Well-Graded Sand	5.7	0.7	2264
HE-12B	23	Sandy Silt	-		-
HE-12B	32	Sandy Silt	3.9	1.0	490
HE-12B	38	Sandy Silt	4.3	0.9	800
HE-12B	42	Clayey Silt	4.6	0.8	1276
HE-12B	52	Silty Sand	5.3	0.8	832
HE-15A	37	Silty Sand	3.2	0.7	929
HE-15A	52	Silty Clay	4.9	0.7	1214
HE-15A	67	Silty Clay	5.6	0.6	705
HE-15A	78	Medium Sand	6.1	0.5	2043
HE-15A	87	Fine to Medium Sand	7.0	0.5	2590

Prior pressuremeter tests were performed for the suspended Metro Eastside Extension project at Borings DD-27/DD-27S, PB-1, PB-2, and PB-3/PB-3A. The test results are summarized in Table 3.3-2b, Prior Pressuremeter Tests (1996) and Table 3.3-2c, Prior Pressuremeter Tests (1995).

Table 3.3-2b: Prior Pressuremeter Tests (1996)

Boring	Test Depth (feet)	Soil Type	Estimated At-Rest Total Horizontal Earth Pressure (ksf)	At Rest Horizontal Pressure Coefficient [Ko]	E_m (ksf)
PB-1	8	Fill – Sand with Gravel	---	---	100
PB-1	15	Gravel with Sand	3.0	1.6	660
PB-1	19	Gravel with Silt and Sand	4.5	1.2	345
PB-1	39	Gravel with Silt and Sand	5.5	1.1	1043
PB-1	60	Sandy Lean Clay with Gravel;	9.3	1.3	1065
PB-2	11	Sand with Gravel	3.5	2.6	156
PB-2	35	Sand	6.9	1.6	2457
PB-2	56	Silty Claystone	6.8	1.0	1022
PB-2	75	Silty Claystone	7.8	0.8	1867
PB-2	93	Silty Claystone	10.8	0.9	3363
PB-3	23	Sandy Clay	4.1	1.4	720
PB-3	35	Sandy Clay	---	---	642
PB-3	54	Sandy Clay	8.3	1.2	479
PB-3	68	Silty Clay with Gravel	12.8	1.5	1153
PB-3	82	Sandy Silty Clay	14.7	1.5	1619
PB-3a	23	Silty Clay with Sand	4.4	1.5	473
PB-3a	33	Lean Clay with Sand	4.4	1.1	729

Table 3.3-2c: Prior Pressuremeter Tests (1995)

Boring	Test Depth (feet)	Soil Type	Estimated At-Rest Total Horizontal Earth Pressure (ksf)	At Rest Horizontal Pressure Coefficient [Ko]	E_m (ksf)
DD-27	28	Silty Sand	---	---	---
DD-27	40.5	Sandy Clay	---	---	---
DD-27	47	Silty Sand / Clayey Sand	8.6	1.5	3110
DD-27	53.5	Clay / Silty Sand	5.8	0.9	1938
DD-27	60	Silty Sand	14.4	1.9	2151
DD-27S	66.5	Silty Sand	7.2	1.0	1426
DD-27S	73	Silty Sand / Sandy Silt	7.2	0.9	1593
DD-27S	79.5	Silty Sand	8.6	0.9	2390
DD-27S	86	Silty Sand	7.2	0.9	1356
DD-27S	92.5	Fine Silty / Sand	10.1	0.9	818
DD-27S	99.5	Sandy Silt / Silty Sand	10.1	0.8	1328
DD-27S	107.5	Sandy Silt / Silty Sand	---	---	---
DD-27S	112.5	Clayey Sand / Silty Clay	11.5	0.8	1800

The tests summarized in Table 3.3-2b were performed by Law/Crandall in December 1996 (Law/Crandall, 1997a) while the tests summarized in Table 3.3-2c were performed by Hughes InSitu Engineering, Inc. for GeoTransit Consultants in September 1995 (Hughes InSitu Engineering, Inc., 1995). Details are presented in Appendix E.

The locations of all the borings referenced in this section are presented on Plate 1a. The at-rest pressure coefficient (K_o) values presented in the tables above are not measured values but interpreted values based on loading/unloading behavior. Accordingly, differences in the estimated K_o values presented in the tables above may be due to many factors including the differences in location, the applied ultimate pressure, number of load cycles, size and disturbance of the borehole, etc.

3.4 LABORATORY TESTING

3.4.1 Geotechnical Laboratory Testing

As borings were completed, undisturbed samples and split-spoon samples were delivered to Law/Crandall for visual inspection and laboratory testing.

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to evaluate their pertinent engineering properties. The following tests were performed on the selected samples:

- direct shear,
- sieve analysis,
- wash analysis (percent passing the #200 sieve),
- hydrometer analysis,
- Atterberg limits,
- moisture content and dry density determinations,
- consolidation,

- permeability and,
- unconfined compression.

Testing was performed in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in Appendix F, Geotechnical Laboratory Test Results. A summary of the geotechnical laboratory test results from the current investigation is presented in Table F-1 of Appendix F, Summary of Geotechnical Laboratory Test Results – Current Borings.

In addition, laboratory test results were available from prior investigations. The results applicable to the current investigation are presented in Appendix G, Prior Geotechnical Laboratory Test Results. A summary of the applicable laboratory test results from previous investigations is presented in Table G-1 of Appendix G, Summary of Geotechnical Laboratory Test Results – Prior Borings.

3.4.2 Analytical (Chemical) Testing

Fourteen soil samples from the proposed location of the tunnel and stations were sent to M.J. Schiff and Associates for additional corrosivity testing to evaluate the corrosion and sulfate attack potential of the site soils. The corrosion report is presented in Appendix H, Corrosion Study.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The Underground Segment is located in the north-central portion of the Los Angeles Coastal Plain. The coastal plain is an alluviated lowland area bounded on the north by the Santa Monica Mountains and the Elysian, Repetto, and Puente Hills, on the east and southeast by the Santa Ana Mountains and the San Joaquin Hills, and on the south and southwest by the Pacific Ocean. A deep structural basin underlies the coastal plain. Parts of the basin have undergone deposition of sediments since late Cretaceous time and continuous marine deposition and subsidence of the basin have been ongoing since middle Miocene time. Numerous oil fields are located in the basin, such as the Boyle Heights Oil Field that underlies a portion of the Underground Segment.

The Underground Segment is located along the southern flank of the Elysian and Repetto Hills and generally traverses a dissected Pleistocene age terrace in an east-southeast to west-northwest direction. The Los Angeles River is located about 0.35 mile west of the western tunnel portal.

Regionally, the site is in the Peninsular Ranges geomorphic province that is characterized by northwest-trending mountain ranges separated by straight-sided, sediment-floored valleys (Yerkes et al., 1965). The northwest trend is further reflected by the dominant geologic structural features of the province, that include northwest to west-northwest trending faults and fault zones such as the Newport-Inglewood fault zone and Whittier fault zone.

The Underground Segment is underlain at depth by the Elysian Park Thrust (Petersen et al., 1996), and the shallower Elysian Park fault (Oskin et al., 2000) is located approximately 1.3 miles to the northeast. Both of these structures are deep buried thrust faults that are expressed at the surface as the Elysian Park Anticline (EPA). The EPA is a 20-kilometer-long and 10-kilometer-wide active fold on the northern margin of the Los Angeles Basin (Oskin et al., 2000). Studies of the contraction rate of secondary folds on the southern limb of the EPA have been used to estimate the rate of contraction of the entire EPA. These contraction rates, in turn, are used to estimate the slip rates on the underlying thrust faults (Oskin et al., 2000). The Coyote Pass

Escarpment is a gentle south-facing and east-west trending lineament northeast of downtown Los Angeles, in the immediate vicinity of the Underground Segment. The escarpment is related to secondary folding on the southern limb of the EPA. The results of previous fault studies performed for the project indicate that the Elysian Park fault and the Elysian Park Thrust are active. Future movement at depth along these faults could result in monoclinal folding and deformation of near surface alluvial deposits and the underlying Fernando Formation bedrock along the escarpment.

The relationship of the Underground Segment to local geologic features is presented on Plate 3, Local Geology, and the faults in the vicinity of the alignment are shown on Plate 4, Regional Faults. Plate 5, Regional Seismicity, shows the locations of major faults and earthquake epicenters in southern California.

4.2 SUBSURFACE STRATIGRAPHY

The geologic materials encountered in explorations along the Underground Segment include artificial fill, Quaternary age younger alluvial deposits, Quaternary age older alluvial and terrace deposits, and sedimentary bedrock units of the Pliocene age Fernando Formation. The graphic representation of the subsurface materials encountered in the borings along the Underground Segment, and the subsurface distribution of the materials with respect to the design tunnel depth, are shown on Plates 2.1b through 2.12b, Geologic Profile.

4.2.1 Artificial Fill

Artificial fill locally mantles younger alluvium and older terrace and alluvial deposits within the Underground Segment. The artificial fills are generally associated with commercial and residential development. The artificial fill encountered in borings along the Underground Segment consists primarily of mixed clay, silt, and sand and extend locally to depths up to 20 feet. However, the typical depth of fill is generally less than 5 feet.

4.2.2 Younger Alluvium (Qal)

Quaternary age younger alluvium (postulated to be Holocene age) locally fills channels that dissect uplifted older alluvial and terrace deposits along the Underground Segment. The younger alluvium encountered in borings along the Underground Segment consists primarily of stiff to hard and medium dense to dense interbedded clay, silt, and sand with localized gravelly zones.

The base of the younger alluvium was identified in borings in the area of the First/Soto Station at depths between 11 and 16 feet below ground surface (EL 284 and EL 277). These deposits grade with depth into lithologically similar older alluvial deposits. Where not identified during field logging, the contact between younger and older alluvium is inferred on Plates 2.1b through 2.12b using dashed lines broken by question marks based on the relationships depicted on the geologic map of surficial exposures and review of materials encountered in borings.

4.2.3 Older Alluvium (Qoa)

Quaternary age older alluvial and terrace deposits (postulated to be Pleistocene age) encountered in borings along the Underground Segment are weakly cemented, predominantly very stiff to hard and dense to very dense, and vary in texture from fine-grained silt and clay to sand with gravel and cobbles. These sediments (herein after referred to as older alluvium) were deposited in alluvial fans and along stream channels forming a cover on Pliocene age bedrock. As shown on Plates 2.1b through 2.12b, the thickness of the older alluvium varies from approximately 50 feet to greater than 120 feet along the Underground Segment. In general, the older alluvium deepens from the western end of the Underground Segment toward the Los Angeles River. However, the thickness of the older alluvium varies, and is thickest where the underlying bedrock surface has been dissected by past stream erosion.

4.2.4 Fernando Formation (Tfr)

The unconsolidated older alluvium is underlain by bedrock of the Pliocene age Fernando Formation (also called the Repetto Formation) that is predominantly massive claystone (Yerkes et al., 1977; Dibblee, 1989). Thin well-cemented calcareous beds and nodules occur locally within the Fernando Formation but typically at depths deeper than the planned invert of the proposed underground structures. Bedrock of the Fernando Formation is approximately 4,000 feet thick in the vicinity of the Underground Segment. Bedrock was encountered in the borings listed in Table 4.2-1, Depth to Bedrock (listed from west to east).

Table 4.2-1: Depth to Bedrock

Boring No.	Station	Depth to Bedrock (feet)
L-16	192+25	70½
SD-9	192+60	54
SD-10	193+80	64½
FL-40	196+25	102½
L-10	240+10	102
PE-15	250+35	86½
L-11	256+60	81
PE-14	258+70	101
SD-16	271+35	125
PE-13	273+65	88½
SD-17	274+45	99

As shown on Plates 2.1b through 2.12b, Geologic Profile, the bedrock surface undulates as a result of past folding and erosion. Considering the highly weathered nature of the Fernando Formation encountered in the borings, within the depth of interest for this project, the engineering properties are expected to be similar to those of the overlying alluvium.

Local zones of hard, well-cemented concretionary nodules of up to 2½ inches in size were reported in prior borings SD-9 and SD-10 located in the eastern part of the First/Boyle Station. These zones are below the current planned design depths of underground structures and are not expected to impact the project.

4.3 GROUNDWATER CONDITIONS

The Underground Segment is located in the northern portion of the Los Angeles Coastal Plain. The coastal plain is divided into the Hollywood, the Santa Monica, the Central, and the West Coast ground-water basins (California Department of Water Resources, 1961). The Central ground-water basin is further divided in the Los Angeles Forebay, the Montebello Forebay, the Whittier, and the Central Basin Pressure Areas. The project is located within the Los Angeles Forebay Area that is characterized as an area of unrestricted infiltration of surface water. Groundwater in the Los Angeles Forebay predominantly occurs in the Quaternary age sediments. This is due to the relatively low permeability of the underlying bedrock of the Fernando Formation. However, limited groundwater may be present in the bedrock materials, generally occurring along fractures or joints, or locally in the more porous sandstone beds.

Aquifers in the Los Angeles Forebay Area include the Semiperched, the Gaspur, the Exposition, the Gardena, and the Gage. Because bedrock is relatively shallow (and the water-bearing sediments are relatively thin) along the majority of the Underground Segment, only the Semiperched and the Gaspur Aquifers are present in the project area (California Department of Water Resources, 1961). The Semiperched Aquifer generally consists of the older (Pleistocene age) sediments and locally the younger (Holocene age) sediments overlying the bedrock, and the Gaspur Aquifer consists of the coarser-grained younger (Holocene age) sediments in channel areas.

Current and historic groundwater levels in the area of the Underground Segment have been evaluated based on data from the following sources:

- groundwater information from monitoring wells installed for the current study,
- groundwater information from monitoring wells installed for the tunnel portion of the suspended Eastside Extension project,
- groundwater information from previous geotechnical investigations for other projects in the immediate area,
- available groundwater data from the County of Los Angeles Department of Public Works, and

- groundwater data developed by the California Division of Mines and Geology (1998) for "Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California," Open-File Report 98-20.

The groundwater levels measured during the current and prior investigations are presented graphically on Plates 2.1b through 2.12b, Geologic Profile, and discussed in the sections below.

4.3.1 First/Boyle Station Area

The bottom of the First/Boyle Station is proposed to be at approximately EL 255. Except for the groundwater level measured at Monitoring Well SD-6, the current levels are the highest groundwater levels reported in the general area. A downward fluctuation of about 26 feet was measured in Monitoring Well SD-6 between September 1996 and October 1996.

Recently measured groundwater levels at the First/Boyle Station were 50 to 60 feet BGS, which corresponds approximately to EL 255. Previous historic high groundwater levels in this area reported by the California Division of Mines and Geology (1998) were about 70 feet BGS (EL 244); about 12 feet below the bottom of the proposed structure.

4.3.2 First/Soto Station Area

The bottom of the First/Soto Station is proposed to be at about EL 238. Groundwater level measurements in borings and monitoring wells during the current and prior investigations indicate that the current groundwater level at the station is at about 24 feet BGS (EL 269), about 32 feet above the bottom of the proposed station structure. The current groundwater level data from borings and monitoring wells indicate that the current groundwater levels appear to be near the historic highs.

The previous historic high groundwater level in the general area of the station were reported as being about 84 feet BGS (EL 215) by the California Division of Mines and Geology (1998). This historic groundwater level is based on a groundwater level contour map developed from a limited number of deep wells at a considerable distance (generally about a mile) away from the station.

4.3.3 East Portal Area

The planned bottom of the East CC Tunnel and the adjoining East Portal ranges from approximately 56 feet BGS (EL 265) at the west end to at-grade at the east end. Based on review of groundwater levels measured for prior investigations, current groundwater levels are expected to range from 84 feet BGS (EL 237) at the west end of the East CC Tunnel to 71 feet BGS (EL 229) at the east end of the portal. These levels are the historical high groundwater levels recorded in the immediate area.

Previous historic high groundwater levels in this area were reported by the California Division of Mines and Geology (1998) as being about 200 feet BGS (EL 91), about 160 feet below the bottom of the proposed station structure. These historic groundwater levels are based on a groundwater level contour map developed from a limited number of deep wells at a considerable distance (generally about a mile) away from the station.

4.3.4 Bored Tunnel Portions

The measured groundwater levels along the bored tunnel sections are generally from approximately 24 to 85 feet BGS (EL 268 to EL 238). Assuming the groundwater levels during construction are the same as the most recently measured data, the portion of the tunnels approximately between STA 194+00 and STA 256+50 will be partly or entirely below the groundwater levels.

4.3.5 Perched Groundwater Conditions

What appeared to be perched groundwater conditions were observed at Borings HE-12, HE-12A, HE-12B, HE-14, HE-30, and HE-31 during drilling and were reported as perched in several previous borings for the suspended Eastside Extension project. To evaluate potential perched groundwater conditions, nested wells were installed in the following borings: HE-12, HE-30, and HE-31. Well development logs are presented in Appendix A.

The stabilized water levels observed in these nested wells indicate that the higher groundwater level observed during drilling is the actual groundwater level. Nonetheless, due to the highly stratified nature of the alluvium, actual localized perched groundwater zones at higher elevations can be expected to exist.

4.4 FAULTS AND SEISMICITY

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Division of Mines and Geology (CDMG) for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1997).

By definition, an active fault is one that has had fault rupture within Holocene time (about the last 11,000 years). A potentially active fault is a fault that has demonstrated fault rupture of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years.

A list of nearby active faults and the distance in miles between the closest portion of the Underground Segment and the nearest point on the fault, the maximum magnitude earthquake, and the slip rate for the fault is given in Table 4.4-1, Major Named Faults Considered to be Active in Southern California. A similar list for potentially active faults is presented in Table 4.4–2, Major Named Faults Considered to be Potentially Active in Southern California. The faults in the vicinity of the Underground Segment are shown on Plate 4, Regional Faults.

Table 4.4-1: Major Named Faults Considered to be Active in Southern California

Fault (in increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site* (miles)	Direction From Site
Elysian Park Thrust (Petersen et al., 1996)	6.7	(a)	RO	1.5	—	—
Elysian Park Fault (Oskin et al., 2000)	6.2-6.7	(b)	RO	0.8-1.2	1.3	NE
Compton-Los Alamitos Thrust	6.8	(a)	RO	1.5	4	SW
Raymond	6.5	(a)	RO	0.5	5	N
Hollywood	6.4	(a)	RO	1.0	5.3	NNW
Newport-Inglewood Zone	6.9	(a)	SS	1.0	6.8	SW
Verdugo Zone	6.7	(a)	RO	0.5	6.8	NNE
Whittier	6.8	(a)	SS	2.5	10	SE
Santa Monica	6.6	(a)	RO	1.0	10½	W
Sierra Madre	7.0	(a)	RO	3.0	11	NNE
San Fernando	6.7	(a)	RO	2.0	16	NNW
Northridge Thrust	6.9	(a)	RO	1.5	16½	NW
San Gabriel	7.0	(a)	SS	1.0	16½	NNE
Palos Verdes	7.1	(a)	SS	3.0	17½	SW
Malibu Coast	6.7	(a)	RO	0.3	18	W
Anacapa-Dume	7.3	(a)	RO	3.0	27	WSW
Cucamonga	7.0	(a)	RO	5.0	29	ENE
Simi-Santa Rosa	6.7	(a)	RO	1.0	32	NW
Elsinore (Glen Ivy Segment)	6.8	(a)	SS	5.0	33	SE
San Andreas (Southern Segment)	7.4	(a)	SS	24.0	33	NE
Oak Ridge	6.9	(a)	RO	4.0	34	NW
San Jacinto (San Bernardino Segment)	6.7	(a)	SS	12.0	40	ENE
San Cayetano	6.8	(a)	RO	6.0	50	NW

* Measured from the site to a projection of the fault to the ground surface
(a) Petersen et al., 1996
(b) Oskin et al., 2000
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique

4.4.1 Active Faults

The known active faults with the potential for ground rupture within 10 miles of the Underground Segment are discussed below. Additionally, the San Andreas fault zone is discussed because it is considered a potential source of significant ground shaking along the Underground Segment. Table 4.4-1, Major Named Faults Considered to be Active in Southern California, lists these and other active faults in the Southern California area. A separate discussion of active blind thrust fault zones is presented in Section 4.4.2, Blind Thrust Fault Zones.

4.4.1.1 Raymond Fault

The closest active fault with the potential for ground rupture is the Raymond fault located approximately 5 miles north of the Underground Segment. The fault is a high-angle reverse fault, thrusting basement rocks north of the fault over alluvial sediments south of the fault. The Raymond fault has long been recognized as a groundwater barrier in the Pasadena/San Marino area and numerous geomorphic features along its entire length (such as fault scarps, sag ponds, springs, and pressure ridges) attest to the fault's activity during the Holocene epoch (last 11,000 years). Within the last 36,000 years, eight separate earthquake events have been recognized along the Raymond fault (Crook et al., 1987). The most recent fault movement, based on radiocarbon ages from materials collected in an excavation exposing the fault, occurred sometime between 2,160 ± 105 and 1,630 ± 100 years before present (LeRoy Crandall and Associates, 1978; Crook et al., 1987).

4.4.1.2 Hollywood Fault

The active Hollywood fault is located approximately 5.3 miles north-northwest of the Underground Segment. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood-Beverly Hills area (Dolan and Sieh, 1992) to the Los Feliz area of Los Angeles. The fault is a groundwater barrier within Holocene sediments (Converse et al., 1981). Scarps 6 to 9 feet high in Holocene flood plain deposits have been suggested along the fault trace in the Atwater area (Weber et al. 1980). Studies by several investigators (Dolan et al., 2000b; Dolan et al., 1997; Dolan and Sieh, 1992; and Crook and Proctor, 1992) have indicated that the fault is active, based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies. Additionally, recent investigations performed in the Hollywood area by Law/Crandall (2000) have demonstrated that Holocene age alluvial sediments have been offset by several strands of the Hollywood fault. An Alquist-Priolo Earthquake Fault Zone has not been established for the Hollywood fault. However, the Hollywood fault is considered active by the State Geologist. Also, the City of Los Angeles considers the Hollywood fault active for planning purposes.

4.4.1.3 Newport-Inglewood Fault Zone

The active Newport-Inglewood fault zone is located approximately 6.8 miles southwest of the Underground Segment. This fault zone is composed of a series of discontinuous northwest-trending en echelon faults extending from Ballona Gap southeastward to the area offshore of Newport Beach. This zone is reflected at the surface by a line of geomorphically young anticlinal hills and mesas formed by the folding and faulting of a thick sequence of Pleistocene age sediments and Tertiary age sedimentary rocks (Barrows, 1974). Fault-plane solutions for 39 small earthquakes (between 1977 and 1985) show mostly strike-slip faulting with some reverse faulting along the north segment (north of Dominguez Hills) and some normal faulting along the south segment (south of Dominguez Hills to Newport Beach) (Hauksson, 1987). Investigations by Law/Crandall (1993) in the Huntington Beach area indicate that the North Branch segment of the Newport-Inglewood fault zone offsets Holocene age alluvial deposits in the vicinity of the Santa Ana River. The recent magnitude 4.2 earthquake on September 9, 2001 reportedly originated at depth along the northern end of the Newport-Inglewood fault zone.

4.4.1.4 Verdugo Fault Zone

The Eagle Rock fault of the active Verdugo fault zone is located approximately 6.8 miles north-northeast of the Underground Segment. The active Verdugo fault zone is composed of several faults including the Verdugo fault, the San Rafael fault, and the Eagle Rock fault. The most recent documented activity along this fault occurs in the Holocene age alluvial deposits along the western flank of the Verdugo Mountains in the Burbank area (County of Los Angeles Seismic Safety Element, 1990). Additionally, this portion of the fault is considered active by the State Geologist (Jennings, 1994). An Alquist-Priolo Earthquake Fault Zone has not been established for the Verdugo fault zone by the State Geologist. However, a fault rupture hazard zone has been designated by the City of Burbank for the Verdugo fault zone. It is our opinion that the Verdugo fault zone should be considered active for planning purposes.

4.4.1.5 Whittier Fault

The active Whittier fault is located about 10 miles southeast of the Underground Segment. This northwest-trending fault extends along the south flank of the Puente Hills from the Santa Ana River on the southeast to the Merced Hills, and possibly beyond, on the northwest. The main fault trace is a high-angle reverse fault, with the north side uplifted over the south side at an angle of approximately 70 degrees. In the Brea-Olinda Oil Field, the Whittier fault displaces Pleistocene age alluvium, and Carbon Canyon Creek is offset in a right lateral sense by the Whittier fault. Yerkes (1972) estimates vertical separation along the fault zone to be approximately 5,900 to 11,800 feet, with a right slip component of about 15,000 feet.

4.4.1.6 San Andreas Fault Zone

The active San Andreas fault zone is located about 33 miles northeast of the Underground Segment. This fault zone, California's most prominent geological feature, trends generally northwest for almost the entire length of the state. The southern fault segment, closest to the Underground Segment, is approximately 280 miles long and extends from the Mexican Border to the Transverse Ranges west of Tejon Pass. Wallace (1968) estimated the recurrence interval for a magnitude 8.0 earthquake along the entire fault zone to be between 50 and 200 years. Sieh (1984) estimated a recurrence interval of 140 to 200 years. The 1857 Fort Tejon earthquake (estimated magnitude 8.0) was the last major earthquake along the San Andreas fault zone in Southern California.

4.4.2 Blind Thrust Fault Zones

4.4.2.1 Elysian Park Thrust (Petersen et al., 1996)

The Elysian Park Thrust, previously defined by Hauksson (1990) as the Elysian Park Fold and Thrust Belt, was postulated to extend northwesterly from the Santa Ana Mountains to the Santa Monica Mountains, extending westerly and paralleling the Santa Monica-Hollywood and Malibu Coast faults. The Elysian Park Thrust is now believed to be smaller in size, only underlying the central Los Angeles Basin (Petersen et al., 1996). The Elysian Park Thrust underlies the Underground Segment at depth (10 to 15 km BGS). Like other blind thrust faults in the Los Angeles area, the Elysian Park Thrust is not exposed at the surface and does not present a potential surface rupture hazard; however, the Elysian Park Thrust should be considered an active feature capable of generating future earthquakes with associated significant ground shaking and possible deformation of the near surface materials. An average slip rate of 1.5 mm/yr and a maximum magnitude of 6.7 are estimated by Petersen et al. (1996) for the Elysian Park Thrust.

4.4.2.2 Elysian Park Fault (Oskin et al., 2000)

The Elysian Park fault, located approximately 1.3 miles northeast of the Underground Segment, is described by Oskin et al. (2000) as a blind thrust fault located northeast of and at a shallower depth than the Elysian Park Thrust of Petersen et al. (1996). Both of these faults are a source of compressional folding of the Tertiary age bedrock underlying the northern Los Angeles Basin. The resulting geologic structure formed by the compressional folding, is known as the the Elysian Park Anticline (EPA). The EPA is expressed at the surface as the Elysian, Repetto, and Montebello Hills.

Oskin et al. (2000) postulate several geometries and activity rates for the recently described Elysian Park fault. The estimated average recurrence interval for events of the Elysian Park fault range from 500 to 1,300 years with an estimated moment magnitude of up to 6.7. Evidence to define the activity of the Elysian Park fault is lacking, however, given the history of seismic events on blind thrust faults in the greater Los Angeles area (i.e. Whittier Narrows and Northridge earthquakes) and proximity to the project area of this newly defined fault, we consider the Elysian Park fault active for planning and design of the proposed project.

4.4.2.3 Coyote Pass Escarpment

The Coyote Pass Escarpment is a gentle south-facing, east-west trending topographic lineament that forms the southern flank of the Repetto Hills, from the Los Angeles River channel to the Monterey Park area (Woodward-Clyde Consultants, 1997). The escarpment is an area of young, near-surface monoclinal folding, believed to be a result of fault rupture on the Elysian Park Thrust (Petersen et al., 1996) and/or the shallower Elysian Park fault (Oskin et al., 2000).

The results of recent investigations of the Coyote Pass Escarpment indicate that the Elysian Park fault is active. Future fault rupture at depth along the Elysian Park fault and/or the Elysian Park Thrust could result in near-surface folding of the alluvial sediments and underlying bedrock in the area of the escarpment. Thus, no ground rupture is anticipated along the Coyote Pass Escarpment, but there is a potential for ground deformation (active folding) of the bedrock and the overlying alluvial sediments along the mapped location of the escarpment.

A project-specific evaluation of the Coyote Pass Escarpment was performed by Earth Consultants International, Inc. (ECI). The results are presented in a separate report (Earth Consultants International, Inc., 2001).

4.4.3 Potentially Active Faults

The known potentially active faults within 10 miles of the Underground Segment are discussed below. Table 4.4-2, Major Named Faults Considered to be Potentially Active in Southern California, lists these and other potentially active faults in the Southern California area.

Table 4.4-2: Major Named Faults Considered to be Potentially Active in Southern California

Fault (in increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site* (miles)	Direction From Site
Coyote Pass	6.7	(b)	RO	0.1	0.4	NE
MacArthur Park	5.7	(c)	RO	3.0	1.5	W
Overland	6.0	(d)	SS	0.1	10	SW
Norwalk	6.7	(d)	RO	0.1	10½	SE
Charnock	6.5	(d)	SS	0.1	11½	SW
Duarte	6.7	(d)	RO	0.1	14	NE
Los Alamitos	6.2	(b)	SS	0.1	14	SSE
Clamshell-Sawpit	6.5	(a)	RO	0.5	14½	NE
Northridge Hills	6.6	(e)	SS	1.2	18½	NW
Indian Hill	6.6	(b)	RO	0.1	20	ENE
El Modeno	6.5	(b)	NO	0.1	21	SE
San Jose	6.5	(a)	RO	0.5	22	E
Peralta Hills	6.5	(b)	RO	0.1	25	SE
Santa Susana	6.6	(a)	RO	5.0	25	NW
Chino – Central Avenue	6.7	(a)	NO	1.0	27	E
Holser	6.5	(a)	RO	0.4	36	NW

* Measured from the site to a projection of the fault to the ground surface
(a) Petersen et al., 1996
(b) Mark, 1977
(c) Hummon et al., 1994
(d) Slemmons, 1979
(e) Wesnousky, 1986
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique

4.4.3.1 Coyote Pass Fault

The closest potentially active fault to the Underground Segment is the Coyote Pass fault (not associated with the Coyote Pass Escarpment, which is related to the Elysian Park fault and/or the Elysian Park Thrust) located approximately 0.4 mile to the northeast. This fault trends east-west across the southerly flank of the Repetto Hills for a distance of about 3 miles (California Department of Water Resources, 1961). Based on available information, the fault is a northerly-dipping reverse fault with rocks of the Pliocene age Fernando Formation, north of the fault, thrust over early Pleistocene age sediments, south of the fault. There is no evidence that this fault has offset late Pleistocene or Holocene age alluvial deposits (County of Los Angeles Seismic Safety Element, 1990). Ziony and Jones (1989) indicate that the fault is potentially active (no displacement of Holocene age alluvium). Additionally, Jennings (1994) indicates the fault is potentially active.

4.4.3.2 MacArthur Park Fault

The potentially active MacArthur Park fault is located approximately 1.5 miles west of the Underground Segment. The fault, inferred west of downtown Los Angeles, has been located based on south-facing scarps, truncated drainages, and other geomorphic features (Dolan and Sieh, 1993). The fault is approximately 5 miles long, extending northwest from the Pershing Square area in downtown Los Angeles through MacArthur Park to the Paramount Studios area in Hollywood. Current information suggests the fault is potentially active.

4.4.3.3 Overland Fault

The potentially active Overland fault is located approximately 10 miles southwest of the Underground Segment. The Overland fault trends northwest between the Charnock fault and the Newport-Inglewood fault zone. The fault extends from the northwest flank of the Baldwin Hills to Santa Monica Boulevard in the vicinity of Overland Avenue. Based on groundwater level measurements, displacement along the fault is believed to be vertical, with an offset of about 30 feet (Poland, 1959). The west side of the fault has apparently moved downward, relative to the east side, forming a graben between the Charnock and Overland faults. However, there is no evidence that this fault has offset late Pleistocene or Holocene age alluvial deposits (County of Los Angeles Seismic Safety Element, 1990). Ziony and Jones (1989) indicate that the fault is potentially active (no displacement of Holocene age alluvium). Additionally, the State Geologist considers this fault to be potentially active (Jennings, 1994).

4.5 GEOLOGIC-SEISMIC HAZARDS

4.5.1 Fault Rupture

The Underground Segment is not within, and does not traverse, a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. The closest Alquist-Priolo Earthquake Fault Zone, established for a portion of the Raymond fault, is approximately 10.1 miles north of the Underground Segment at its closest point. Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located directly beneath or projecting toward the Underground Segment. Therefore, the potential for surface rupture due to fault plane displacement propagating to the surface at the site during the design life of the project is considered low.

4.5.2 Coyote Pass Escarpment Deformation

The Underground Segment is south of and parallels the Coyote Pass Escarpment. The Coyote Pass Escarpment is an area of surface deformation believed to be a result of fault movement along the Elysian Park fault and/or the Elysian Park Thrust. These buried thrust faults are considered active and there is a potential for ground deformation (active folding) of the bedrock and the overlying alluvial sediments in the vicinity of the escarpment during the design life of the proposed project.

The results and details of a project-specific-investigation performed by ECI to delineate and characterize the escarpment (Coyote Pass Escarpment) and to assess its seismic capability are presented in a separate report (Earth Consultants International, Inc., 2001). The results reportedly indicate that the Coyote Pass Escarpment is an active structure consisting of folded alluvial deposits and underlying sedimentary bedrock. Its potential for movement should be considered in the design and construction of project structures immediately adjacent to the escarpment (First/Soto Station) and at projected escarpment crossings, if any. Estimated strains during characteristic events are presented in the ECI report.

4.5.3 Seismicity

A number of earthquakes of moderate to major magnitude have occurred in the Southern California area within the last 69 years that have produced significant ground shaking in the area of the Underground Segment. The earliest of these was the March 10, 1933 magnitude 6.4 Long Beach earthquake. The epicenter of this earthquake was located about 32 miles to the south-southeast of the Underground Segment.

The epicenter of the February 9, 1971, magnitude 6.6 San Fernando earthquake was about 27 miles northwest of the Underground Segment. Surface rupture occurred on various strands of the San Fernando fault zone as a result of this earthquake, including the Tujunga and Sylmar faults.

The magnitude 5.9 Whittier Narrows earthquake occurred on October 1, 1987, on a previously unrecognized fault, now believed to be a separate blind thrust fault that is situated below the Elysian Park fault and above the Elysian Park Thrust. The earthquake epicenter was located approximately 6.8 miles east of the Underground Segment.

The Sierra Madre earthquake occurred on June 28, 1991, along the Sierra Madre fault zone. The epicenter of the magnitude 5.8 earthquake was located in the San Gabriel Mountains about 20 miles northeast of the Underground Segment.

On June 28, 1992, two major earthquakes occurred east of Los Angeles. At 4:58 a.m., a magnitude 7.5 earthquake occurred in the High Desert region and is known as the Landers earthquake. The epicenter was located about 98 miles east-northeast of the Underground Segment. The second event occurred at 8:04 a.m. near Big Bear Lake and had a magnitude of 6.6; the epicenter was about 77 miles east-northeast of the Underground Segment.

On January 17, 1994, the magnitude 6.7 Northridge earthquake occurred on a previously unknown blind thrust fault that is now known as the Northridge Thrust. The Northridge Thrust is located beneath the majority of the San Fernando Valley and is considered to be the eastern extension of the active Oak Ridge fault. The epicenter of the Northridge earthquake was located about 21 miles northwest of the Underground Segment.

Most recently, the magnitude 7.0 Hector Mine earthquake occurred on October 16, 1999. The earthquake is believed to have occurred on the Lavic Lake fault, previously thought to have been inactive. The epicenter of the Hector Mine earthquake is located approximately 118 miles east-northeast of the Underground Segment.

4.5.4 Slope Stability

According to the City of Los Angeles Safety Element (1996) and the County of Los Angeles Seismic Safety Element (1990), the Underground Segment is not located within an area identified as having a potential for slope instability. There are no known landslides near the Underground Segment, nor is the Underground Segment in the path of any known or potential landslides.

Additionally, the Underground Segment is not located within an area identified as having a potential for seismic slope instability (California Division of Mines and Geology, 1999).

4.5.5 Liquefaction

Liquefaction potential is greatest where the groundwater level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as density, grain size, and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the County of Los Angeles Seismic Safety Element (1990) and the City of Los Angeles Safety Element (1996), portions of the Underground Segment are within areas identified as having a potential for liquefaction. These areas consist of the segments between State Street and Soto Street (STA 196+60 and STA 222+20), between Matthews Street and Savannah Street (STA 226+20 and STA 243+30), and between Dacotah Street and Lorena Street (STA 254+65 and STA 272+80). More recent geologic mapping and engineering analysis by the California Division of Mines and Geology (1999) indicate that no portion of the Underground Segment is located within an area identified as having a potential for liquefaction.

Based on recent groundwater level measurements in monitoring wells and geotechnical borings for the proposed Eastside LRT project, groundwater occurs at depths less than 50 feet in portions of the alignment. According to the California Division of Mines and Geology (1999), the older alluvial deposits within the Underground Segment are not expected to liquefy. However, younger saturated alluvial deposits, designated as Qal in the borings and in our geologic profiles, may be susceptible to liquefaction. These deposits are postulated to occur west of STA 184+00, between STA 213+80 and STA227+20, between STA 248+35 and STA 253+00, and between STA 270+00 and STA 276+40. However, the younger alluvial deposits encountered along the Underground Segment below the design groundwater level consist predominantly of stiff clay, which would preclude the potential for liquefaction. Accordingly, the potential for liquefaction along the Underground Segment due to the ODE and MDE events is deemed unlikely and need not be considered for design.

4.5.6 Seismic Settlement

Seismically induced settlement is often caused by loose to medium-dense granular soils densified during ground shaking. Dry and partially saturated soils as well as saturated granular soils are subject to seismically induced settlement. Generally, differential settlements induced by ground failures such as liquefaction, flow slides, and surface ruptures would be much more severe than those caused by densification alone.

The generally dense granular soils encountered in the current and previous exploratory borings below the planned underground structures are not in the loose to medium-dense category. Based on the soil conditions along the Underground Segment, the potential for seismically induced settlement below the underground structures due to the ODE and MDE events is low and need not be considered for design. Seismically induced settlement of loose and medium dense granular soils encountered above the tunnel sections at a few boring locations may occur but is not expected to exceed ½ inch and 1 inch for the ODE and MDE events, respectively.

4.5.7 Tsunamis, Inundation, Seiches, and Flooding

The alignment is not in a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard.

According to the City of Los Angeles Safety Element (1996) and County of Los Angeles Seismic Safety Element (1990), the Underground Segment is not located downslope of any large bodies of water that could adversely affect the site in the event of earthquake-induced dam failures or seiches (wave oscillations in an enclosed or semi-enclosed body of water).

The Underground Segment is in an area of minimal flooding potential (Zone C) as defined by the Federal Insurance Administration.

4.5.8 Subsidence

The portion of the Underground Segment east of Soto Street and west of Dacotah Street is within the boundaries of the abandoned Boyle Heights Oil Field. Regional subsidence associated with petroleum production has been identified in some of the oil fields in the Los Angeles Basin; however, regional subsidence has not been identified in the Boyle Heights Oil Field. In addition, the Underground Segment is not within an area of known regional subsidence associated with groundwater withdrawal, peat oxidation, or hydrocompaction. Consequently, the potential for regional subsidence within the Underground Segment is considered low.

4.5.9 Conclusions

Based on the available geologic data, active or potentially active faults with the potential for ground rupture are not known to be located beneath the Underground Segment. In our opinion, the potential for ground rupture along the Underground Segment due to fault plane displacement propagating to the ground surface during the design life of the project is low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The Coyote Pass Escarpment has the potential to induce ground deformation at the First/Soto Station and underground structures to the west of Soto Street. We understand that design provisions will be implemented as necessary to address potential deformations and racking at the station and along tunnel portions.

Slope stability hazards are not expected to affect the project. The potential for other geologic hazards such as liquefaction, seismically induced settlement, tsunamis, inundation, seiches, flooding, and subsidence affecting the project is considered low.

5.0 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

Engineering properties of subsurface materials were evaluated using the results of current and prior field and laboratory tests. The results of geotechnical laboratory tests performed during the current investigation and in pertinent borings from prior investigations are summarized in Tables F-1 and G-1, Summary of Geotechnical Laboratory Test Results, in Appendix F, Geotechnical Laboratory Test Results, and Appendix G, Prior Geotechnical Laboratory Test Results, respectively. Blowcounts obtained during sampling and Standard Penetration Tests (SPTs) are shown in the boring logs presented in Appendix A, Subsurface Explorations, and in Tables F-1 and G-1.

Table 5.1-1, Summary of Estimated Engineering Properties, presents a summary of the pertinent geotechnical parameters for the natural soils and weathered bedrock encountered within the project area. The values in the "Range" columns in Table 5.1-1 are actual measurements or observations. The values in the "Best Estimate" columns are based partly on statistical analyses, published values and correlations in the literature, and engineering judgment.

Because artificial fill was not encountered within depths that could potentially impact station or tunnel design, its engineering properties were not evaluated. Station and tunnel excavation and construction is expected to be entirely within alluvium deposits. However, because weathered bedrock is expected immediately below a portion of the tunnel section east of the First/Boyle station, engineering properties for the weathered bedrock were also evaluated.

Engineering parameters for dynamic analysis are summarized in Table 5.1-2, Engineering Properties for Dynamic Analysis. The methodologies used to develop these parameters are discussed in Appendix D, Seismic Velocity Surveys, and in Appendix J, Ground Motion Study.

5.1 ALLUVIUM

In terms of engineering properties, the younger alluvium (Qal) and the older alluvium (Qoa) are similar. Accordingly, both are collectively classified simply as alluvium. However, a distinction is made between fine-grained and coarse-grained alluvium.

5.1.1 Particle Size Distribution

As shown on Plates 2.1b through 2.12b, Geologic Profile, the alluvium along the Underground Segment consists of fine-grained soils interbedded with coarse-grained soils. In general, the alluvium is predominantly fine-grained in the middle section of the Underground Segment (in vicinity of the First/Soto Station) with increasing coarse-grained layers near the First/Boyle Station and the East Portal.

The fine-grained soils in the project area consist predominantly of stiff to hard low plasticity clay and silt. The coarse-grained soils consist predominantly of dense to very dense silty sand and clayey sand, some poorly graded sand and gravel layers, and locally cobbles (up to 12 inches in size). Although cobbles were encountered at two locations in the current and prior explorations at depths impacting the project, additional cobbly zones may exist between boring locations. Boulders (particles greater than 12 inches in diameter) were not encountered in the borings drilled to date along the Underground Segment. Although unlikely, boulders may be encountered between boring locations.

Although the majority of coarse-grained soils encountered within the planned tunnel and station excavations have significant fines content, some sand and gravel layers with low fines content were encountered. These layers may be susceptible to raveling, running, or flowing conditions. Results of particle size distribution and fines content tests are presented in tabular and graphic format in Appendices F and G. The range of fines content is presented in Table 5.1-1.

5.1.2 Moisture Content, Density, and Index Properties

The moisture content and dry density of fine-grained alluvium ranges from 2 to 41 percent and 79 to 123 pounds per cubic foot, respectively. For the coarse-grained alluvium, the range of moisture content and dry density is 2 to 28 percent and 88 to 133 pounds per cubic foot, respectively. Atterberg Limits tests indicate that the fine-grained alluvium is predominantly low-plasticity clay with Liquid Limits ranging from 22 to 63 percent and Plasticity Indices ranging from 6 to 27 percent.

5.1.3 Shear Strength

The results from direct shear and triaxial tests performed as part of the current investigation as well as from prior investigations were used to develop shear strength parameters. The results of these tests are presented in Appendices F and G.

For the fine-grained alluvium, the cohesion intercept and friction angle representing the effective peak shear strength range from 400 to 2,000 pounds per square foot and 24 to 35 degrees, respectively. The corresponding values of the cohesion intercept and friction angle for the coarse-grained alluvium range from 0 to 700 pounds per square foot and 29 to 45 degrees.

5.1.4 Uniaxial Compressive Strength

Unconfined compression tests were performed on selected fine-grained alluvium samples to determine the undrained shear strength and the uniaxial compressive strength. The undrained shear strength is assumed to be half of the compressive strength. Based on the test results, the uniaxial compressive strength of the fine-grained alluvium ranges from 1,100 to 13,200 pounds per square foot.

5.1.5 Static and Dynamic Moduli

The range of static elastic (Young's) modulus values presented in Table 5.1-1 is based on the results of pressuremeter tests from the current and prior investigations. The static elastic modulus ranges from 490 to 1,280 kips per square foot for the fine-grained alluvium and from 830 to 2,590 kips per square foot for the coarse-grained alluvium.

The dynamic modulus values presented in Table 5.1-2 are based on published correlations with shear wave velocity and Poisson's ratio (shown in Table 5.1-1). The results of seismic velocity surveys were used to derive the low-strain shear modulus and to estimate Poisson's ratio. The modulus degradation factors for the Operating Design Earthquake (ODE) and the Maximum Design Earthquake (MDE) were developed based on published degradation curves and strain values from site-specific ground motion study presented in Section 7.0, Ground Motion Study.

5.1.6 Consolidation, Collapse, and Swell Potential

The consolidation, collapse, and swell potential of fine-grained alluvium as well as coarse-grained alluvium with sufficient fines content were evaluated using consolidation tests. The low compression and swelling indices shown in Table 5.1-1 are representative of the predominantly very stiff to hard fine-grained alluvium and the generally dense to very dense coarse-grained alluvium. Both the collapse and swell potential upon saturation is estimated to be less than 1% for fine-grained and coarse-grained alluvium.

5.1.7 Corrosion Potential

Based on chemical test results, the alluvium is classified as severely corrosive to ferrous metals, not particularly aggressive to copper, and sulfate attack on concrete as negligible. The results of soil corrosivity studies are presented in Appendix H, Corrosion Study.

Table 5.1-1: Summary of Estimated Engineering Properties

Engineering Characteristics	Fine-Grained Alluvium		Coarse-Grained Alluvium		Bedrock	
USCS Classification	CL, CH, ML, MH ML-CL		SP, SW, GP, GW, SM, SC, GM, GC GP-GM, GW-GP SP-SM, SW-SM, SC-SM		CL, CH, ML, MH	
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
SPT Blowcounts	8 to >100	-	5 to >100	-	27 to >100	-
Moisture Content (%)	2 – 41	22	2 – 28	15	17 – 28	25
Dry Density (pcf)	78 – 123	104	88 – 133	112	85 – 113	100
Void Ratio	0.14 – 1.09	0.63	0.22 – 1.09	0.48	0.62 – 0.8	0.72
% Fines (by weight)	50 – 99	-	2 – 50	-	96 – 98**	-
Specific Gravity	2.71 – 2.76	2.71	2.64 – 2.69	2.65	2.76	2.76
Liquid Limit (%)	22 – 63	34	0 – 31	-	34-50	45
Plasticity Index (%)	6 – 27	15	0 – 12	-	19-26	19
Peak Shear Strength: Cohesion (psf)	400 – 2,000	500	0 – 700	0	0 – 3,000	1,400
Friction Angle (degrees)	24 – 35	28	29 – 45	35	25 – 35	28
Uniaxial Compressive Strength (psf)	1,100 – 13,200	5,000	*	*	11,000 – 18,400	14,000
Static Elastic Modulus*** (ksf)	490 – 1,280**	1,100	830 – 2,590**	1,500	1,020 – 3,360**	2,000
Poisson's Ratio: Unsaturated	0.27 – 0.45	0.4	0.2 – 0.45	0.35		0.4
Saturated	0.45 – 0.49	0.47	0.45 – 0.49	0.47	0.46**	0.45
pH	6.4 – 8.8	-	5.75 – 8.7	-	8.15**	-
Chloride Content (ppm)	25 – 907	-	11 – 618	-	654**	-
Sulfate Content (ppm)	15 – 210	-	36 – 353	-	79**	-
Electrical Resistivity: Field Moisture	958 – 440,000	-	862-19,000	-	2,665**	-
(ohms-cm) Saturated	860-2941	-	2083-3704			
Compression Index-Cc'	0.05 – 0.16	0.09	0.04 – 0.06**	0.05	0.05 – 0.113	0.07
Swelling Index-Cs'	0.006 – 0.04	0.01	0.005 – 0.008**	0.005	0.01 – 0.018	0.01
Rate of Secondary Compression-Cx'	0.0005 – 0.0084	0.001	0.0004 – 0.0011**	0.0004	0.0003 – 0.0006	0.001
Collapse (-)/Swelling (+), %	-0.02 to +0.82	*	-0.65 to -0.02**	*	+0.15 to +0.88	*

Notes: - not appropriate due to broad range
* not applicable
** limited data
***large-strain Young's Modulus (E) from linear portion of pressuremeter pressure-expansion curve (please see Appendix E, Pressuremeter Testing)

Table 5.1-2: Summary of Engineering Properties for Dynamic Analysis

Material Property[1]	Fine-Grained Alluvium		Coarse-Grained Alluvium		Bedrock	
	Range	Best Estimate	Range	Best Estimate	Range	Best Estimate
Shear Wave Velocity (feet/second)	700 – 1,800	1,300	800 – 1,990	1,400	1,500 – 1,600	1,500
Shear Modulus (10^3 ksf)	1.9 – 12.6	7	3.6 – 16.0	7.5	8.7 – 10.0	9
Elastic Modulus (10^3 ksf)	5.2 – 31.9	20	7.4 – 45.2	20	25.5 – 28.9	27
Damping	5% for the Operating Design Earthquake (ODE) 10% for Maximum Design Earthquake (MDE)					

Notes: [1]Values correspond to small strains (shear strains less than 0.001%).
Apply the following reduction factors to obtain values for the ODE the MDE:

<u>Shear Wave Velocity</u>
ODE = 0.79
MDE = 0.59

<u>Shear and Elastic Moduli</u>
ODE = 0.625
MDE = 0.35

5.2 WEATHERED BEDROCK

As discussed previously, bedrock is expected immediately below a portion of the tunnel section east of the First/Boyle station. However, within the zone of influence of the tunnel, the bedrock is highly weathered. Its engineering properties are expected to be similar to those for fine-grained alluvium. Accordingly, we recommend that the engineering parameters for fine-grained alluvium in Tables 5.1-1 and 5.1-2 be used in the design in lieu of the values for bedrock presented in those tables.

6.0 ENVIRONMENTAL CONSIDERATIONS

This section summarizes the results of the environmental investigation for the Underground Segment. As shown on Plate 1a, Plot Plan – Boring Locations, a portion of the Underground Segment traverses the Boyle Heights Oil Field. We have identified four abandoned oil wells in the vicinity of the project alignment. Of these four identified wells, the well near the southeast corner of First Street and Rivera Street (between Borings PE-15 and DD-25) in the eastern portion of the Underground Segment appears to be most pertinent to the project. This well, designated Evergreen No. 1, was drilled in 1957 and abandoned in the same year. The other abandoned wells are either a significant distance away or near at-grade portions of the alignment.

In addition to abandoned oil wells, we have identified four sites along the Underground Segment with existing or previously removed underground storage tanks (USTs) for the storage of petroleum hydrocarbons. These USTs may have released contaminants into the subsurface soils or groundwater, or both. The potential contamination from these sites, if present, likely consists of petroleum hydrocarbons and volatile organic compounds. The sites are listed as follows:

- *Evergreen Cemetery/Crematory* (3301 E. First Street) – diesel or stove oil UST removed in September 1987, remedial action taken, file closed by the Regional Water Quality Control Board (RWQCB) in October 1998.
- *M & Y Service Station* (2701 E. First Street) at the northeast corner of First and Mott Streets – three gasoline USTs removed in 1995, remedial action taken, file closed by the RWQCB in January 1997.
- *Vega Auto Service* (1869 E. First Street) at the northwest corner of First and State Streets – four gasoline and waste oil USTs removed in November 1991, remediation in progress.
- *Murray Lefkowitz* (2239 E. First Street) at the northwest corner of First and Breed Streets – four gasoline and waste oil USTs removed in December 1991, remedial action planned but not implemented.

A former gas station facility located at the southwest corner of First Street and Soto Street, currently operating as a used car dealership (*Guadalajara Auto Sales* at 111 S. Soto Street), is documented as having three empty USTs. We did not locate records indicating that these USTs have released contaminants into the subsurface soils or groundwater.

In addition to the sites listed above, three other former gasoline station sites with USTs were identified in a prior environmental report (Enviro-Rail, 1998) at and in the immediate vicinity of the First/Boyle Station. These three sites were referenced as:

- *Site 39 from a Vista Report* at the southwest corner of the intersection of First Street and Boyle Avenue;
- *undocumented former Chevron gasoline station* on the south side of First Street, approximately 150 feet east of the centerline of Boyle Avenue; and
- *undocumented former Shell gasoline station* at the southeast corner of Boyle Avenue and Pleasant Avenue, approximately 100 feet north of First Street.

Considering that identified and potentially unidentified oil wells and sites with USTs may exist along the Underground Segment, the possibility of encountering contamination and dangerous soil gases during construction is deemed significant. The environmental investigation for the current study consisted of field and laboratory analyses of soil, groundwater, and soil vapor samples to assist in the identification of contaminated areas and their potential impact on the project. A more detailed discussion of the results of the current soil, groundwater, and soil vapor analyses is presented in Appendix I, Environmental Data.

6.1 PREVIOUS INVESTIGATIONS

Portions of the Underground Segment were previously investigated by several consultants as part of environmental investigations for the suspended MTA Metro Red Line Eastside Extension project (Suspended Project). These prior investigations were completed by Engineering-Science, Inc., Law/Crandall, Enviro Rail, GeoTransit Consultants, Earth Technology Corporation and Environmental Science and Engineering. A discussion of these applicable prior investigations is included in Appendix I, Environmental Data. In general, the results of the current investigation, which are presented in Section 6.2, Current Soil, Groundwater, and Soil Vapor Assessment, are consistent with the results of previous environmental investigations for the Suspended Project at areas coincident with the Underground Segment.

6.2 CURRENT SOIL, GROUNDWATER, AND SOIL VAPOR ASSESSMENT

Our scope of work for the current study included:

- installation of additional groundwater monitoring wells,
- collection and analysis of soil and groundwater samples, and
- collection and analysis of vapor samples.

6.2.1 Soil Assessment

As part of the current investigation, we collected environmental soil samples from 20 L-series borings (Borings L-1 through L-13 and L-17 through L-23). The environmental samples from Borings L-18 and L-19 were taken from the soil cuttings generated during the drilling activities. Harding ESE collected environmental soil samples from 11 HE-series borings within the Underground Alignment (Borings HE-9, HE-10, HE-11, HE-12, HE-13, HE-14, HE-15, HE-30, HE-31, HE-32A, and HE-33. The boring locations are shown on Plate 1a, Plot Plan – Boring Locations.

Laboratory analyses were performed on 26 environmental soil samples from the L-series borings that, in some cases, were deemed to be potentially contaminated based on visual observations, odors, and/or headspace vapor measurements using a photo ionization or flame ionization detector (PID/FID). Environmental soil samples collected from the L-series borings were analyzed in the laboratory to quantify the following:

- Total recoverable petroleum hydrocarbons (TRPH) by EPA Method 418.1,
- Total petroleum hydrocarbons (TPH) gasoline and diesel ranges by EPA Method 8015 Modified (M), and
- Volatile organic compounds (VOCs) including methyl tertiary butyl ether (MTBE) by EPA Method 8260B.

In addition, selected soil samples were also analyzed for California Code of Regulations (CCR) Title 22 metals by EPA Methods 6010B and 7471A. The soil analytical results above the laboratory reporting limits (RL) for metals, TRPH, TPH, and VOCs are summarized in Table I-1 of Appendix I, Soil Analytical Results.

Environmental soil samples collected by Harding ESE from the HE-series borings were similarly analyzed in the laboratory to quantify the TRPH by EPA Method 418.1, and the VOCs including MTBE by EPA Method 8260B. In addition, selected soil samples were also analyzed for CCR Title 22 metals by EPA Methods 6010B and 7471A. The soil analytical results above the laboratory RL for metals, TRPH, TPH, and VOCs are summarized in Table I-1 of Appendix I, Soil Analytical Results.

6.2.1.1 Metals

The analytical laboratory results indicate that detected concentrations of metals are generally at concentrations within naturally occurring ranges. Detected concentrations did not exceed the Total Threshold Limit Concentrations (TTLCs) established by Title 22 to define a material as being a hazardous waste. Metals concentrations also did not exceed 10 times the Soluble Threshold Limit Concentrations (STLCs). Excavated soils are not expected to be classified as hazardous due to metals impact.

6.2.1.2 Petroleum Hydrocarbons

Petroleum hydrocarbons were detected at concentrations above the laboratory reporting limits in 12 samples from nine borings (Borings L-5, L-17, L-18, L-19, HE-9, HE-13, HE-15, HE-31, and HE-32A). The detected concentrations of TRPH, and TPH compounds in the gasoline, diesel and heavy oil ranges were below 1,000 mg/kg. Based on the site location and the depth to groundwater, TRPH and TPH concentrations of 1,000 mg/kg are generally considered an action level by regulatory agencies. Accordingly, excavated soils are not expected to be classified as hazardous due to petroleum hydrocarbons.

6.2.1.3 Volatile Organic Compounds

Volatile organic compounds (generally gasoline additives and aromatic volatile organics) were detected in samples from Borings L-5, L-17, HE-9, HE-30, HE-31, and HE-32A. The detected VOC concentrations are below the EPA Region 9 Residential Preliminary Remediation Goals (PRGs). Accordingly, excavated soils are not expected to be classified as hazardous due to VOCs.

6.2.1.4 Findings

Based on the soil samples tested, the majority of the soils that are to be excavated as part of the construction of the Underground Segment are not expected to be classified as hazardous. In the samples in which metals were detected, concentrations were indicative of naturally occurring levels. The TPH and VOC concentrations detected in samples were below levels requiring remedial action. However, stockpiles of excavated soils may need to be tested prior to use as compacted fill. Soils impacted by contamination cannot be used as compacted fill without specific regulatory agency approval.

The TPH and VOC concentrations detected in soil samples obtained near the groundwater level at Borings L-5, L-17, HE-9, and HE-31 may have been caused by "smearing" from groundwater contamination plumes. Documented contamination plumes exist in the immediate vicinity of the aforementioned borings.

In the immediate vicinity of L-5, L-17, and HE-31, a groundwater contamination plume from the Murray Lefkowitz site is documented as having migrated offsite. The Guadalajara Auto Sales site at First Street and Soto Street, which has three USTs, is also in the immediate vicinity of L-17 and HE-31. However, we did not observe records indicating that the USTs at the Guadalajara Auto Sales site had released contaminants into the subsurface soils or groundwater. As shown on Plate 2.6b, the groundwater surface appears to slope downward to the west between HE-31 and L-17 but appears to be essentially level between L-17 and L-5.

A groundwater contamination plume from the Vega Auto Service site, which is currently under remediation, is documented as having migrated offsite. The Vega Auto Service site is in the immediate vicinity of Boring HE-9.

Considering that a portion of the Underground Segment traverses the Boyle Heights Oil Field, the possibility of encountering naturally occurring petroleum deposits remains, especially between boring locations. Although considered non-hazardous, excavated soils containing these naturally occurring petroleum deposits typically require special management and handling. These impacted materials will likely not be reusable as fill.

6.2.2 Groundwater Assessment

No floating free product was observed in the developed wells. We collected groundwater samples from five of the six groundwater monitoring wells installed in the L-series borings (wells at Borings L-2, L-5, L-9, L-11 and L-17). We did not encounter groundwater in the monitoring well at Boring L-1. In addition, Harding ESE collected groundwater samples from the wells installed in the HE-series borings. The samples from the following HE-series wells were analyzed: HE-12 (shallow and deep nested wells), HE-30 (shallow and deep nested wells), HE-31 (shallow and deep nested wells), HE-32, HE-33, and HE-34. Groundwater samples were analyzed for:

- Dissolved methane (Robert S. Kerr Standard Operation Procedure, RSK, 175M),
- Dissolved oxygen (EPA Method SM 4500-O G),
- Sulfate (EPA Method 300.0),
- Sulfide (EPA Method 376.2),
- Title 22 metals (EPA Methods 6010B and 7471A),
- TRPH (EPA Method 418.1)
- TPH (EPA Method 8015M), and
- VOCs plus MTBE (EPA Method 8260B).

The groundwater analytical results are summarized in Table I-2 of Appendix I, Groundwater Analytical Results.

6.2.2.1 Dissolved Methane and Dissolved Oxygen

Dissolved methane was detected in 10 wells ranging in concentration from 0.001 to 4.88 milligrams per liter (mg/L). Relatively high concentrations of dissolved methane (1.36 to 4.88 mg/L) in Monitoring Wells L-5, L-17, and the shallow nested Monitoring Well HE-31 are most likely due to biological degradation of the TPH and VOC contaminants in the wells.

Dissolved oxygen was detected in the wells with concentrations ranging from 2.29 to 7.94 mg/L.

6.2.2.3 Sulfate and Sulfide

Sulfate was detected above the laboratory RL (10 mg/L) in 13 of the 14 samples submitted for analysis. The concentrations ranged from 19.6 to 386 mg/L. Sulfide was not detected above the laboratory RL in the wells that were sampled.

6.2.2.4 Metals

The metals concentrations detected in the groundwater samples did not exceed published Federal and State of California maximum contaminant levels (MCLs).

6.2.2.5 Petroleum Hydrocarbons

Minor concentrations of TRPH (3.0 mg/L and 13.0 mg/L) were detected in Monitoring Wells L-5 and L-17, respectively. TPH concentrations in the gasoline range of 2.9 mg/L and 15.0 mg/L were detected in Monitoring Wells L-5 and L-17.

Samples from HE-series wells were not evaluated for TRPH. TPH concentrations in the gasoline range were detected in HE-30 (shallow nested well – 427 mg/L), and HE-31 (shallow nested well – 235 mg/L and deep nested well – 770 mg/L).

6.2.2.6 Volatile Organic Compounds

Volatile organic compounds were detected above the laboratory RL in Monitoring Wells L-2, L-5, L-17, HE-12 (deep nested well), HE-30 (shallow and deep nested wells), HE-31 (shallow and deep wells), and HE-32. The detected concentrations were below the MCLs and PRGs for drinking water (Table I-2 in Appendix I).

6.2.2.7 Findings

TPH (gasoline range), VOCs, and dissolved methane were encountered in groundwater samples from some of the monitoring wells along an approximately 500-foot long section of the Underground Segment. The most likely sources of the contamination are plumes from nearby leaking USTs.

Contaminated groundwater is not expected to impact the project beyond potentially special requirements for storage, treatment, or disposal of groundwater extracted during the planned dewatering of the First/Soto Station excavation. The TPH and VOC concentrations detected in the groundwater sampled from Monitoring Wells L-5, L-17, and HE-31 may be within discharge limitations to allow disposal into the sanitary sewer system. For the aquifer pumping test performed at the First/Soto Station site (Law/Crandall, 2002), we obtained a permit for discharge into the sanitary sewer from the City of Los Angeles Department of Public Works Bureau of Sanitation. However, considering that a significant volume of groundwater will likely be discharged, a National Pollutant Discharge Elimination System (NPDES) permit for disposal into the storm drain system may be necessary. Pre-discharge treatment will likely be necessary for disposal of extracted groundwater into the storm drain system.

Dewatering at the First/Soto Station excavation may affect the shape and extent of existing contamination plume(s). Further investigation of the plume(s) may help better characterize sources, extent, and direction of migration. The information will be useful in reducing potential claims from property owners in the impacted area.

The potential impact of high levels of dissolved methane is off-gassing potentially explosive levels of methane gas, which is discussed in Section 6.2.3, Soil Gas Condition.

6.2.3 Soil Gas Condition

, We installed vadose zone monitoring probes in Borings L-1, L-2, L-5, L-9, L-11 and L-17 to evaluate the concentrations of methane and hydrogen sulfide gases along the Underground Segment. In addition, Harding ESE installed vadose zone monitoring probes in Borings HE-10A, HE-12A, HE-15, and HE-32A along the Underground Segment. Headspace gas samples were also collected from groundwater wells installed into the saturated zone to measure off-gassing of hydrogen sulfide and methane gases from groundwater.

As part of this investigation, accessible vadose zone probes and groundwater wells previously installed by other consultants along the Underground Segment were monitored using field instruments. Headspace gas samples from previously installed Monitoring Wells DD-25, EB-12, SD-6, and SD-9 were analyzed in the laboratory to evaluate methane gas and hydrogen sulfide gas concentrations. Field measurements and laboratory results are summarized on Table I-3 of Appendix I, Laboratory and Field Soil Vapor Results.

6.2.3.1 Methane Gas

In general, field-monitoring results indicated low levels of methane gas except at EB-12A, in which a concentration of 264,000 parts per million (ppm) was detected on December 12, 2001 and a concentration of 185,000 ppm was detected on January 31, 2002. Elevated concentrations of methane gas were measured in laboratory samples from the following monitoring wells:

- L-5 (probe sample = 9,300 ppm and well-head sample = 3,200 ppm),
- L-17 (probe sample = 22,000 ppm and well-head sample = 2,600 ppm), and
- EB-12A (well-head sample = 110,000 ppm).

A methane gas concentration of 5,000 ppm is considered an action level by regulatory agencies in the Los Angeles area. Considering that facilities related to oil-field activities and abandoned oil wells were not identified in the vicinity, the methane detected in wells L-5 and L-17 appears to have been generated by biological degradation of impacted soil and groundwater with gasoline compounds. The methane detected in well EB-12A may be related to former oil-field activities and abandoned oil wells in the vicinity.

Significant positive soil vapor pressure was not detected in any of the vapor probes tested. The results were generally consistent with the previous investigations in this area for the Suspended Project. Although the methane measurements are only representative of small confined volumes of the monitoring well casings, they indicate the potential for accumulation of methane if proper ventilation is not provided.

6.2.3.2 Hydrogen Sulfide Gas

Hydrogen sulfide was not present above the detection limits in the gas samples tested.

6.2.3.3 Summary of Soil Gas Condition

Although the investigation did not detect widespread gaseous conditions, changes in the groundwater conditions and barometric (atmospheric) pressure may cause the groundwater to off-gas both hydrogen sulfide and methane gases. Furthermore, considering that a portion of the Underground Segment traverses the Boyle Heights Oil Field, the potential of encountering naturally occurring hydrogen sulfide and methane gas remains. Accordingly, implementing monitoring systems and mitigation measures, such as proper ventilation, against potentially explosive levels of methane gas for construction and operation seems appropriate.

6.3 ENVIRONMENTAL SUMMARY

Although our investigation did not encounter widespread contamination due to the historical and current land usage, there remains a possibility that isolated pockets of soil, groundwater, and gas contamination, in addition to those already identified, may be encountered during construction. Any subsurface explorations or construction activities will require prior planning to address the above mentioned environmental conditions. Ultimately, the State of California Occupational Safety and Health Administration (Cal OSHA) will designate the tunnel classification.

The majority of the soils that are to be excavated as part of the construction of the Underground Segment are not expected to be classified as hazardous. However, stockpiles of excavated soils may need to be tested prior to use as compacted fill. Soils impacted by contamination cannot be used as compacted fill without specific regulatory agency approval.

A pre-discharge treatment system will likely be necessary for disposal of groundwater extracted during dewatering from the First/Soto Station excavation into the storm drain system.

7.0 GROUND MOTION STUDY

Ground motions have been postulated using design earthquakes specified in accordance with the seismic design criteria obtained from Eastside LRT Partners. As part of this ground motion study, a Probabilistic Seismic Hazard Analysis (PSHA) was performed to help characterize the ground motions that may be experienced along the Underground Segment. The ground motion study is discussed in more detail in Appendix J, Ground Motion Study.

Specifically, the PSHA was performed to develop site-specific design response spectra for postulated earthquake shaking corresponding to the Operating Design Earthquake (ODE) and the Maximum Design Earthquake (MDE). The ODE has an average return period (recurrence) of 200 years while the MDE has an average return period of 2,000 years.

For both the ODE and MDE, following the development of response spectra, synthetic response spectrum–compatible acceleration time-histories of the postulated ground motions were developed. The ground motions were then numerically propagated from denser rock to the ground surface at the locations of the First/Boyle Station, the First/Soto Station, and the East Portal using a one-dimensional equivalent linear solution (SHAKE91).

The site response analyses (performed using SHAKE91) provided shear strain time-histories from which the time-histories of the relative displacement between the top and bottom of the aforementioned stations and portal (racking) were computed.

7.1 RESPONSE SPECTRA

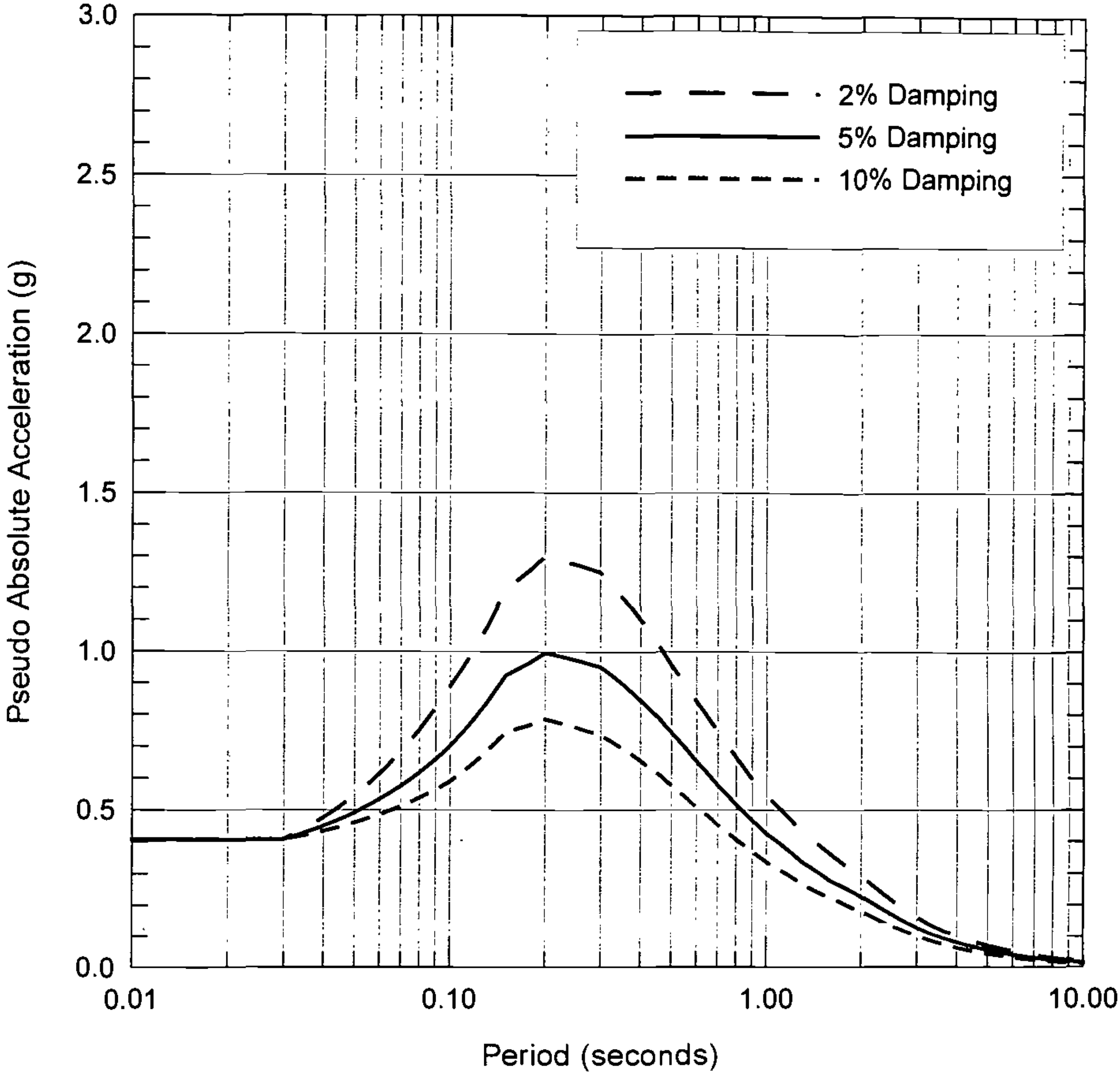
Site-specific response spectra were determined by a PSHA using the computer program FRISKSP, version 3.01b (Blake, 1995). Ground motions were found to be nearly identical at the three sites and from one end of the Underground Segment to the other. Accordingly, one set of response spectra based on the approximate midpoint of the Underground Segment (at the intersection of First Street and Ficket Street) has been developed for the entire Underground Segment.

The Boore et al. (1993) ground motion attenuation relationships were used in the PSHA to provide estimations of the response spectra for periods from 0 to 2 seconds. Based on the shear wave velocity surveys performed along the Underground Segment, which are presented in Appendix D, Seismic Velocity Surveys, the average of the attenuation relations for a Site Class B and Site Class C classifications discussed in Boore et al. (1993) was selected. For periods greater than 2 seconds, the response spectrum was extended based on the shape of the response spectra from PSHA for the site using the attenuation relationships of Abrahamson, 1997 and Sadigh, 1997.

The response spectra for the outcropping ground motions corresponding to the ODE and MDE are presented on Figures 7.1-1 and 7.1-2, Horizontal Response Spectra, for structural damping values of 2%, 5%, and 10%. The response spectra in digitized form are shown in Tables 7.1-1a and 7.1-1b, Pseudo Spectral Velocity, and Table 7.1-2, Pseudo Spectral Acceleration. The peak ground accelerations (PGA) for the ODE and MDE are 0.41g and 0.79g, respectively. The peak ground velocities (PGV) for the ODE and MDE are 0.15g and 0.32g, respectively. The vertical components of the PGA and PGV may be assumed to be two-thirds of the respective horizontal components.

7.2 RESPONSE SPECTRUM COMPATIBLE TIME-HISTORIES

The 5 percent damped site-specific response spectrum for the ODE and MDE ground motions were used as "target spectra" for generation of matched response spectrum-compatible ground motion time histories for the outcropping motion. An empirically recorded "seed" record was modified using Abrahamson's RSPMATCH program (Abrahamson, 1998) such that the resulting response spectra matched the site specific design spectra ("target spectra" for matching) for the ODE and MDE events. Plots of the original, target, and matched response spectra, and plots of the original and matched acceleration, velocity and displacement time histories are provided in Appendix J.

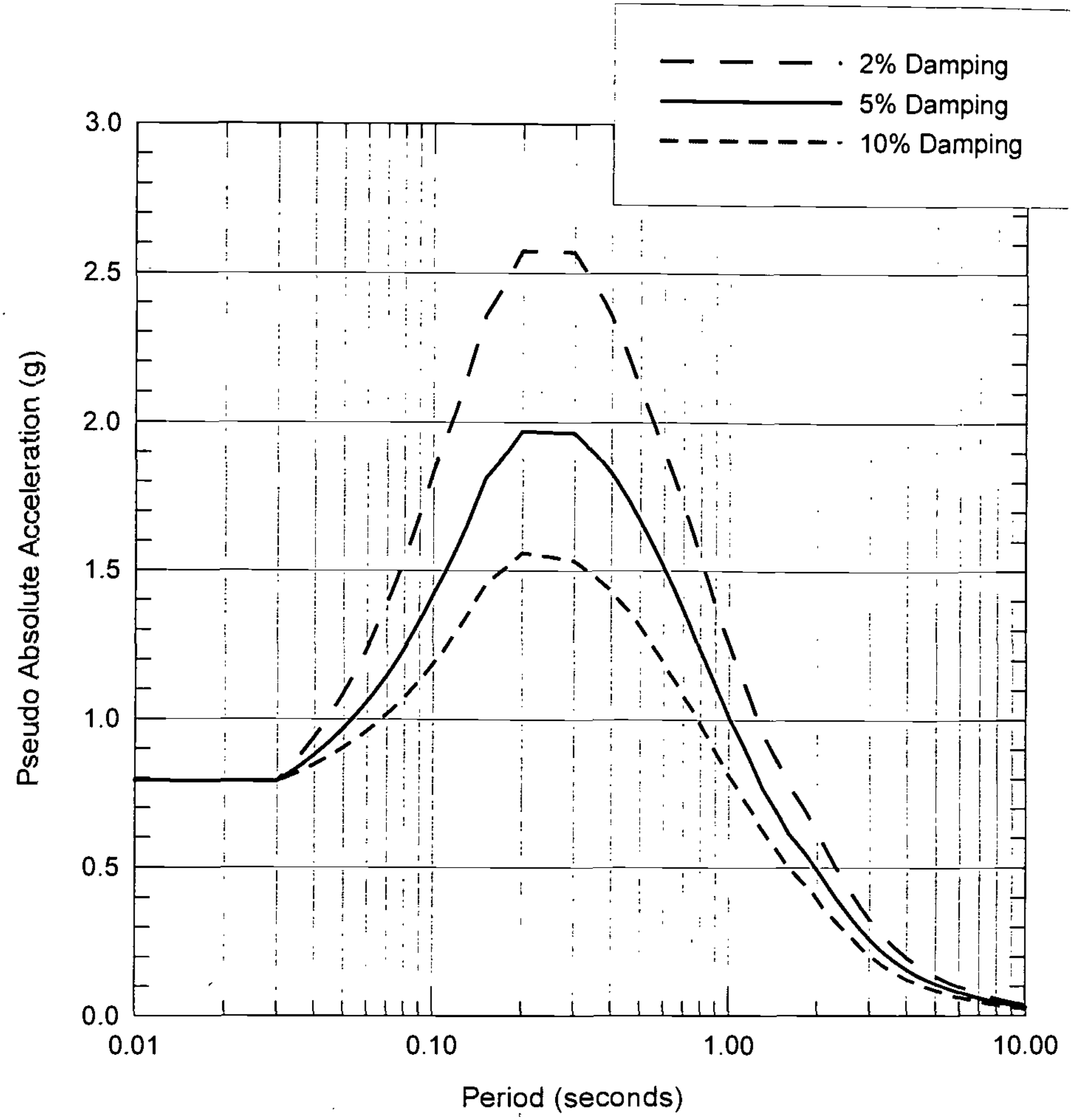


HORIZONTAL RESPONSE SPECTRA

Operating Design Earthquake (ODE)
200-Year Average Return Period

Eastside LRT Project
Underground Segment

LAW/CRANDALL



HORIZONTAL RESPONSE SPECTRA

Maximum Design Earthquake (MDE)
2000-Year Average Return Period

Eastside LRT Project
Underground Segment

LAW/CRANDALL

FIGURE 7.1-2

10138.0002\calculations and analysis\ground motion study\final EPA model\figure j-2.grf

JOB: 70131-1-0138 DATE: 30 August 2001 F.T.: n/a DR.: ER O.E.: cc CHKD:

Table 7.1-1a: Horizontal Ground Motion Pseudo Spectral Velocity (Feet/Second)						
Period	**2% Damping**		**5% Damping**		**10% Damping**	
(Seconds)	**ODE***	**MDE****	**ODE**	**MDE**	**ODE**	**MDE**
0.01	0.02	0.04	0.02	0.04	0.02	0.04
0.03	0.06	0.12	0.06	0.12	0.06	0.12
0.10	0.46	0.94	0.36	0.73	0.30	0.61
0.15	0.92	1.81	0.71	1.39	0.57	1.12
0.20	1.33	2.64	1.02	2.02	0.80	1.60
0.30	1.92	3.95	1.46	3.02	1.13	2.36
0.40	2.26	4.82	1.73	3.75	1.34	2.93
0.50	2.45	5.41	1.90	4.28	1.48	3.36
0.60	2.59	5.86	2.01	4.65	1.56	3.65
0.70	2.68	6.17	2.08	4.91	1.62	3.88
0.80	2.73	6.34	2.11	5.04	1.67	4.04
0.90	2.75	6.37	2.16	5.15	1.70	4.12
1.00	2.79	6.43	2.18	5.18	1.72	4.16
1.30	2.86	6.38	2.24	5.14	1.78	4.16
1.60	2.94	6.39	2.27	5.04	1.81	4.09
2.00	2.99	6.36	2.34	5.04	1.85	4.05
3.00	2.50	5.01	1.98	4.00	1.55	3.17
4.00	2.06	4.06	1.64	3.25	1.28	2.57
5.00	1.79	3.49	1.43	2.79	1.12	2.20
6.00	1.58	3.04	1.28	2.44	0.99	1.91
7.00	1.39	2.67	1.13	2.14	0.87	1.68
8.00	1.24	2.36	1.00	1.90	0.77	1.49
9.00	1.11	2.12	0.90	1.70	0.69	1.33
10.00	1.00	1.91	0.81	1.54	0.63	1.20

*ODE: Operating Design Earthquake, 200-year average return period
**MDE: Maximum Design Earthquake, 2,000-year average return period

By: CK 1/30/02
Chk: JSM 1/31/02

Table 7.1-1b: Horizontal Ground Motion Pseudo Spectral Velocity (Inches/Second)						
Period	2% Damping		5% Damping		10% Damping	
(Seconds)	ODE	MDE	ODE	MDE	ODE	MDE
0.01	0.25	0.49	0.25	0.49	0.25	0.49
0.03	0.75	1.46	0.75	1.46	0.75	1.46
0.10	5.47	11.29	4.31	8.71	3.63	7.30
0.15	11.01	21.74	8.53	16.71	6.86	13.41
0.20	15.98	31.65	12.26	24.22	9.66	19.19
0.30	23.04	47.40	17.57	36.24	13.55	28.29
0.40	27.10	57.89	20.81	45.01	16.13	35.21
0.50	29.44	64.86	22.84	51.33	17.73	40.37
0.60	31.06	70.32	24.11	55.76	18.73	43.86
0.70	32.15	74.03	24.91	58.88	19.49	46.59
0.80	32.76	76.02	25.37	60.52	20.01	48.43
0.90	33.00	76.42	25.88	61.75	20.39	49.46
1.00	33.48	77.11	26.15	62.14	20.65	49.92
1.30	34.34	76.50	26.85	61.66	21.31	49.92
1.60	35.32	76.73	27.26	60.44	21.71	49.11
2.00	35.89	76.27	28.07	60.48	22.19	48.55
3.00	29.96	60.15	23.75	47.98	18.63	38.08
4.00	24.71	48.74	19.68	38.97	15.40	30.79
5.00	21.53	41.83	17.21	33.52	13.44	26.38
6.00	18.92	36.46	15.32	29.25	11.83	22.97
7.00	16.69	31.98	13.53	25.67	10.43	20.14
8.00	14.85	28.37	12.04	22.78	9.29	17.86
9.00	13.32	25.41	10.81	20.40	8.34	15.99
10.00	12.05	22.94	9.77	18.42	7.54	14.43

*ODE: Operating Design Earthquake, 200-year average return period
**MDE: Maximum Design Earthquake, 2,000-year average return period

By: CK 1/30/02
Chk: JSM 1/31/02

Table 7.1-2: Horizontal Ground Motion Pseudo Spectral Acceleration (g's)

Period	2% Damping		5% Damping		10% Damping	
(Seconds)	ODE	MDE	ODE	MDE	ODE	MDE
0.01	0.41	0.79	0.41	0.79	0.41	0.79
0.03	0.41	0.79	0.41	0.79	0.41	0.79
0.10	0.89	1.84	0.70	1.42	0.59	1.19
0.15	1.19	2.36	0.92	1.81	0.74	1.45
0.20	1.30	2.57	1.00	1.97	0.79	1.56
0.30	1.25	2.57	0.95	1.96	0.73	1.53
0.40	1.10	2.35	0.85	1.83	0.66	1.43
0.50	0.96	2.11	0.74	1.67	0.58	1.31
0.60	0.84	1.91	0.65	1.51	0.51	1.19
0.70	0.75	1.72	0.58	1.37	0.45	1.08
0.80	0.67	1.55	0.52	1.23	0.41	0.98
0.90	0.60	1.38	0.47	1.12	0.37	0.89
1.00	0.54	1.25	0.43	1.01	0.34	0.81
1.30	0.43	0.96	0.34	0.77	0.27	0.62
1.60	0.36	0.78	0.28	0.61	0.22	0.50
2.00	0.29	0.62	0.23	0.49	0.18	0.39
3.00	0.16	0.33	0.13	0.26	0.10	0.21
4.00	0.10	0.20	0.08	0.16	0.06	0.13
5.00	0.07	0.14	0.06	0.11	0.04	0.09
6.00	0.05	0.10	0.04	0.08	0.03	0.06
7.00	0.04	0.07	0.03	0.06	0.02	0.05
8.00	0.03	0.06	0.02	0.05	0.02	0.04
9.00	0.02	0.05	0.02	0.04	0.02	0.03
10.00	0.02	0.04	0.02	0.03	0.01	0.02

*ODE: Operating Design Earthquake, 200-year average return period
**MDE: Maximum Design Earthquake, 2,000-year average return period

By: CK 1/30/02
Chk: JSM 1/31/02

7.3 SITE RESPONSE ANALYSES

The site response analyses were performed using the computer program SHAKE91 (Idriss and Sun, 1992). The site response analyses results provided strain time-histories from which the relative displacement between the top and bottom of each station (racking) time-histories were estimated. The calculated relative displacement time-histories between one point at 10 feet below the ground surface (BGS) and another at 55 feet BGS are presented in Appendix J.

7.4 ESTIMATION OF RACKING FOR DESIGN

The recommended racking values for design of the underground structures are based on the computation discussed in the preceding section. Based on the sensitivity analyses, noticeable differences in estimated racking are observed based on the different assumptions for shear wave velocity profiles (best estimate, upper and lower bound).

Based on a comparison of the results from the three locations analyzed, it is recommended that the same racking parameters be used for structures along the Underground Segment. For the ODE, the racking may be taken as ¼ inch in 45 feet of height difference. For the MDE, the racking may be taken as 1 inch in 45 feet. The racking values may be linearly extrapolated for other structure heights.

8.0 DESIGN AND CONSTRUCTION

This section provides a description of geotechnical evaluations and recommendations, and key geotechnical issues for the design of the proposed underground structures.

8.1 GEOTECHNICAL CONSIDERATIONS

Based on current plans and profiles, the underground structures for the project will be constructed within alluvium. The alluvium consists of fine-grained soils interbedded with coarse-grained soils. The fine-grained soils in the project area consist predominantly of stiff to hard low plasticity clay and silt. The coarse-grained soils consist predominantly of dense to very dense silty sand and clayey sand, some gravel layers, and locally cobbles (up to 12 inches in size). Although cobbles were encountered at only two locations in the current and prior explorations at depths impacting the project, additional cobbly zones may exist between boring locations. Boulders (particles greater than 12 inches in minimum diameter) were not encountered in the borings drilled to date along the Underground Segment. Although unlikely, boulders may be encountered between boring locations.

8.2 DESIGN GROUNDWATER LEVELS

The groundwater levels observed and measured during the current investigation are generally consistent with previous project-specific investigations (GeoTransit Consultants, 1996a, 1996b, 1996c, 1994a and 1994b). As described in Section 4.3, Groundwater Conditions, the current groundwater levels in the project area appear to be at or near historic high levels. Groundwater levels in the project area before and during tunnel and station construction are expected be similar to the most recent measured groundwater levels obtained from this investigation.

Although the groundwater levels appear to have remained relatively stable at least since 1994 (except for Monitoring Well SD-6), considering the long design life of the project (100 years), the potential for a rise of the groundwater from current levels should be addressed for long-term performance of underground structures.

As shown in Table 3.2-1, Groundwater Level Summary, the measured fluctuation does not exceed 5 feet except at Monitoring Well SD-6, in which a downward fluctuation of about 26 feet was observed between September 1996 and October 1996. The groundwater level at Monitoring Well SD-6 appears to have stabilized at the lower elevation based on our readings on May and June 2001 and February 2002.

Based on the findings above, the following design groundwater levels are recommended for design:

- for construction, use the most recent groundwater levels obtained from this investigation, and
- for long-term design, use a groundwater level that corresponds to 10 feet below existing grade or 25 feet above the current level, whichever is lower.

We have assumed that construction will commence within 5 years of the date of this report. It is recommended that groundwater levels be monitored prior to and during the construction.

8.3 SEISMIC DESIGN CONSIDERATIONS

As discussed in Section 7.1, Response Spectra, a site-specific Probabilistic Seismic Hazard Analysis (PSHA) was performed for the project site to determine the design response spectra for Operating Design Earthquake (ODE), 200-year average return period, and Maximum Design Earthquake (MDE), 2,000-year average return period. The results are presented on Figure 7.1-1, Horizontal Response Spectra (ODE), and Figure 7.1-2, Horizontal Response Spectra (MDE), for structural damping values of 2%, 5%, and 10%. The response spectra in digitized form are shown in Table 7.1-1, Pseudo Spectral Velocity, and Table 7.1-2, Pseudo Spectral Acceleration. The peak accelerations at the ground surface for the ODE and MDE are 0.41g and 0.79g, respectively. The peak velocities at the ground surface for the ODE and MDE are 1.5 feet per second and 3.2 feet per second, respectively.

Based on the site response analysis discussed in Section 7.3, Estimation of Earthquake Ground Motions at the Station Depths – Site Response Analyses, the racking at the station locations have been estimated for design. Based on a comparison of the results for the three planned stations, it is recommended that each of the stations be designed using the same racking parameters. For the

ODE, the racking may be taken as ¼ inch in 45 feet, and for the MDE, the racking may be taken as 1 inch in 45 feet. The racking value may be linearly interpolated and extrapolated for other station heights.

8.4 ENVIRONMENTAL CONSIDERATIONS

As discussed in Section 6.3, Environmental Summary, although our investigation did not encounter wide-spread contamination; due to the historical and current land usage, isolated pockets of soil, groundwater, and gas contamination in addition to those already identified may be encountered during construction.

The majority of the soils that are to be excavated as part of the construction of the Underground Segment are not expected to be classified as hazardous. However, stockpiles of excavated soils may need to be tested prior to use as compacted fill. Soils impacted by contamination cannot be used as compacted fill without specific regulatory agency approval.

A pre-discharge treatment system will likely be necessary for disposal of groundwater extracted during dewatering of the First/Soto Station excavation into the storm drain system.

8.5 BORED TUNNEL SECTIONS, CROSS-PASSAGES, AND SUMP STRUCTURES

As shown in the geologic profile presented on Plates 2.1b through 2.12b, Geologic Profile, the bored tunnels, cross-passages, and sump structures will be in older alluvium (Qoa). These tunnel structures traverse both fine-grained and coarse-grained soils. Although the majority of coarse-grained soils encountered within the planned tunnel excavations have significant fines content, some sand and gravel layers with low fines content were encountered. These layers may be susceptible to raveling, running, or flowing conditions.

The current subsurface explorations did not encounter particles larger than 3 inches (cobbles) within the planned tunnel bore. An obstruction interpreted as either a boulder or large cobble was encountered within the planned tunnel bore in an 8-inch-diameter hollow-stem auger boring (FL-40) performed as part of a previous investigation for the Suspended Project (GeoTransit

Consultants, 1996e). The log of Boring FL-40 is shown in Appendix B, Prior Subsurface Explorations. Boring FL-40 is located east of the First/Boyle Station.

8.5.1 Excavation Considerations

There are several conditions that may impact the tunnel excavation techniques, face stability, advance rates, and potential ground loss. These conditions include shallow groundwater conditions in alluvium within or above the tunnel envelope; raveling and running/flowing conditions in granular alluvium, local presence of gravels, cobbles, and possibly boulders; and shallow soil overburden above the tunnel crown in portions of the tunnel sections. These conditions should be taken into consideration in tunnel design and construction.

8.5.2 Groundwater Control

Assuming groundwater levels during construction are the same as the most recent measured groundwater levels from this investigation, the portion of the bored tunnel sections approximately between STA 194+00 and STA 256+50 (Plates 2.4b through 2.9b) will likely be fully or partially below the groundwater level. All cross-passages and sump structures will be below the groundwater level except for Cross-Passage No. 6 at STA 255+85.

We understand that because a pressure-face tunnel boring machine and precast, gasketed reinforced concrete liners are planned for excavation of the bored tunnel sections, dewatering will not be required for the tunnel. Dewatering may be required for the construction of cross-passages and sump structures below the groundwater level. These structures will probably be excavated using conventional mining methods.

Due to the potential for fluctuation of the groundwater levels during the service life of the structures, the design groundwater level recommended in Section 8.2, Design Groundwater Levels, should be adopted.

8.5.3 Liner Design

Geotechnical parameters for the design of tunnel, cross-passage, and sump structure liners are presented in Table 8.5-1, Design Geotechnical Parameters. Although the bored tunnel sections, cross-passages, and sump structures traverse both fine-grained and coarse-grained, the engineering properties for purposes of liner design are deemed sufficiently similar that one set of design parameters was developed to simplify design.

8.6 CUT-AND-COVER TUNNEL SECTIONS, PORTALS, AND STATIONS

The proposed cut-and-cover tunnel sections, portals, and stations will be supported on alluvium. We understand that cut-and-cover tunnel sections and stations will be designed and constructed as relatively rigid and water-tight boxes supported on mat foundations. Twin cantilevered reinforced-concrete retaining walls are planned for the shallower portions of portals and twin braced reinforced-concrete retaining walls are planned for the deeper portions. Within the cantilevered section of the portals, retaining walls and center columns supporting electrical guides are to be supported on independent strip footings. The tracks are to be supported on grade. Within the braced section, the retaining walls, electrical guide columns, and tracks are to be supported on a mat foundation.

Vertical loads from soil cover and anticipated traffic and other live loads should be determined and added to the roof loading. A unit weight of 125 pounds per cubic foot may be assumed for the soil cover. The design geotechnical parameters for station, mezzanine, cut-and-cover tunnel, and portal structures are summarized in Tables 8.6-1a through 8.6-1c, Design Geotechnical Parameters.

8.6.1 Excavation Methods

The excavations will be primarily in heterogeneous alluvium. It is anticipated that the excavations can be achieved by conventional excavation methods. However, suitable excavation equipment must be considered to handle the local presence of gravelly interbeds in the alluvium. Most of the coarse-grained alluvium is expected to run or ravel readily. Thus, timely application of ground support will be essential.

Table 8.5-1: Design Geotechnical Parameters
Bored Tunnel Sections, Cross-Passages, and Sump Structures

Parameter	Design Value
Geologic Unit	Alluvium
Unit Weight of Soil (pounds/foot^3)	125
Void Ratio	0.6
Cohesion (pounds/foot^2)	500
Angle of Internal Friction of Soil (degrees)	28
Soil Pressure Coefficient	
At Rest (K_o)(1)	0.8
Active (K_a)	0.3
Groundwater level(2)	10 feet BGS or 25 feet above existing level, whichever is lower
Static Elastic Modulus (kips/foot^2)	1,000
Dynamic Elastic Modulus (kips/foot^2)	
Small Strain (Initial)	20,000
ODE(3)	12,500
MDE(3)	7,000
Shear Wave Velocity(4) (feet/second)	
Small Strain (Initial)	1,600
ODE	1,260
MDE	950
Peak Ground Acceleration (g)	
ODE Horizontal	0.41
ODE Vertical	0.27
MDE Horizontal	0.79
MDE Vertical	0.53
Peak Ground Velocity (feet/second)	
ODE Horizontal	1.5
ODE Vertical	1.0
MDE Horizontal	3.2
MDE Vertical	2.1

Notes:
(1) Use for design of bored tunnel sections, cross-passages, and sump structures.
(2) Assumed maximum groundwater level during the design life of the station.
(3) Operating Design Earthquake (ODE) and Maximum Design Earthquake (MDE) are discussed in Section 7.0, Ground Motion Study.
(4) Value selected from high side of range to reduce compound conservatism in liner design using MTA's Supplementary Seismic Design Criteria dated June 1984.

Table 8.6-1a: Design Geotechnical Parameters
First/Boyle Station and Appurtenant Structures

Parameter	**Design Value**		
Geologic Unit	Main Station Box	Mezzanine	West CC Tunnel & West Portal
Unit Weight of Soil (pounds/foot^3)	125		
Void Ratio	0.5		
Cohesion (pounds/foot^2)	0		
Angle of Internal Friction of Soil (degrees)	35		
Soil Pressure Coefficient			
At Rest (K_o)$^{(1)}$	0.45		
Active (K_a)	0.30		
Groundwater level$^{(2)}$	10 feet BGS or 25 feet above existing level, whichever is lower		
Coefficient of Subgrade Reaction$^{(3)}$			
Upper Bound K_b (kips/foot^3)	80	70	60
Lower Bound K_b (kips/foot^3)	40	35	30
Peak Ground Acceleration (g)			
ODE$^{(4)}$ Horizontal	0.41		
Vertical	0.27		
MDE$^{(4)}$ Horizontal	0.79		
Vertical	0.53		
Peak Ground Velocity (feet/second)			
ODE Horizontal	1.5		
Vertical	1.0		
MDE Horizontal	3.2		
Vertical	2.1		

Notes:
(1) Use for design of below grade walls.
(2) Assumed maximum groundwater level during the design life of Underground Segment.
(3) For an allowable bearing pressure of 5 kips per square foot or lower. Lower Bound values for other bearing pressures can be obtained from settlement chart (Figure 8.6-3a). The Upper Bound value may be assumed to be twice the Lower Bound value. The Lower Bound values 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 Maximum Design Earthquake (MDE) are discussed in Section 7.0, Ground Motion Study.

Table 8.6-1b: Design Geotechnical Parameters
First/Soto Station and Mezzanine

Parameter	**Design Value**	
Geologic Unit	Main Station Box	Mezzanine
Unit Weight of Soil (pounds/foot^3)	125	
Void Ratio	0.6	
Cohesion (pounds/foot^2)	500	
Angle of Internal Friction of Soil (degrees)	28	
Soil Pressure Coefficient		
At Rest (K_o)[1]	0.50	
Active (K_a)	0.30	
Groundwater level [2]	10 feet BGS	
Coefficient of Subgrade Reaction[3]		
Upper Bound K_b (kips/foot^3)	80	60
Lower Bound K_b (kips/foot^3)	40	30
Peak Ground Acceleration (g)		
ODE[4] Horizontal	0.41	
Vertical	0.27	
MDE[4] Horizontal	0.79	
Vertical	0.53	
Peak Ground Velocity (feet/second)		
ODE Horizontal	1.5	
Vertical	1.0	
MDE Horizontal	3.2	
Vertical	2.1	

Notes:
(1) Use for design of below grade walls.
(2) Assumed maximum groundwater level during the design life of Underground Segment.
(3) For an allowable bearing pressure of 5 kips per square foot or lower. Lower Bound values for other bearing pressures can be obtained from settlement chart (Figure 8.6-3b). The Upper Bound value may be assumed to be twice the Lower Bound value. The Lower Bound values 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 Maximum Design Earthquake (MDE) are discussed in Section 7.0, Ground Motion Study.

Table 8.6-1c: Design Geotechnical Parameters
East CC Tunnel and East Portal

Parameter		Design Value	
Geologic Unit		East CC Tunnel	East Portal
Unit Weight of Soil (pounds/foot^3)		125	
Void Ratio		0.5	
Cohesion (pounds/foot^2)		0	
Angle of Internal Friction of Soil (degrees)		35	
Soil Pressure Coefficient			
At Rest (K_o)(1)		0.45	
Active (K_a)		0.30	
Groundwater level(2)		10 feet BGS or 25 feet above existing level, whichever is lower	
Coefficient of Subgrade Reaction(3)			
Upper Bound K_b (kips/foot^3)		80	60
Lower Bound K_b (kips/foot^3)		40	30
Peak Ground Acceleration (g)			
ODE(4)	Horizontal	0.41	
	Vertical	0.27	
MDE(4)	Horizontal	0.79	
	Vertical	0.53	
Peak Ground Velocity (feet/second)			
ODE	Horizontal	1.5	
	Vertical	1.0	
MDE	Horizontal	3.2	
	Vertical	2.1	

Notes:
(1) Use for design of below grade walls.
(2) Assumed maximum groundwater level during the design life of Underground Segment.
(3) For an allowable bearing pressure of 5 kips per square foot or lower. The Upper Bound value may be assumed to be twice the Lower Bound value. The Lower Bound values 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 Maximum Design Earthquake (MDE) are discussed in Section 7.0, Ground Motion Study.

Based on the results of the current and prior investigations, it is unlikely that a significant volume of cobbles will be encountered in the planned excavations. Cobbles were not encountered within design depths of the portals or cut-and-cover tunnel section. Cobbles were encountered in only one boring within design station depths. An approximately 2-foot-thick layer of gravel with cobbles was encountered above the groundwater level in Boring HE-30, which is at the First/Soto Station.

8.6.2 Dewatering and Groundwater Control

If collection and pumping of groundwater inflow into the excavation is deemed to be inadequate, preconstruction or construction dewatering may be required for the West CC Tunnel and the First/Boyle Station. The groundwater level in this area is expected to be at the tunnel and station inverts. Lowering and maintenance of the groundwater level at least 5 feet below the bottom of excavation is typically required.

Dewatering will be required for the construction of the First/Soto Station. The groundwater level will probably be lowered and maintained about 35 to 40 feet below current levels. Settlement of existing structures adjacent to the excavation at the First/Soto Station due to dewatering is expected to be on the order of 1 inch or less. The settlement is estimated to decrease linearly from about 1 inch adjacent to the excavation perimeter to zero at a distance of about 300 feet away, which corresponds to a deflection ratio of 1/3600 (differential settlement/horizontal distance). Recommendations for dewatering are presented in a separate report (Law/Crandall, 2002).

Dewatering is not expected to be required for the construction of the portal structures. However, due to the stratified nature of the alluvium, localized perched groundwater inflows may be encountered due to the accumulation of infiltration water on less permeable fine-grained layers. If the inflows into the excavation are significant, a system of collector ditches and sump pumps may need to be installed to dewater the bottom of the excavation.

Groundwater levels should be regularly monitored up to and through the construction period to verify those levels. Rising groundwater levels may require the installation of supplementary dewatering systems.

8.6.3 Sloped Excavations

Considering the proximity of existing buildings and infrastructure, sloped excavations will probably not be feasible for a large portion of the planned underground structures. Nonetheless, recommendations for sloped excavations are provided below in case shallow excavations for any ancillary structures are feasible.

Where the necessary space is available, temporary unsurcharged embankments may be sloped back at 1:1 without shoring. Sloped excavations may be feasible for portal structures depending on the available right-of-way. Adjacent to any existing structure, the bottom of any unshored excavation should be restricted so as not to extend below a plane drawn at 1½:1 (horizontal to vertical) downward from the foundations of existing structure. Large, critical structures, such as shallow foundations for bridge structures, will require a flatter setback slope. These cases should be analyzed on a case-by-case basis. Where space is not available, shoring will be required. Data for design of shoring are presented in Section 8.6.4, Shored Excavations.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the tops of the slopes. A greater setback may be necessary when considering embankments greater than 15 feet in height or heavy vehicles and equipment, such as concrete trucks and cranes. We should be advised of such heavy loadings so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces.

The recommended slopes above are for reference only. The design, construction, and proper maintenance of safe, stable slopes will be the responsibility of the Contractor. The actual excavation slopes should be based on geotechnical evaluation of construction conditions and subsurface conditions encountered during excavation.

8.6.4 Shored Excavations

Excavations as deep as 65 feet below the existing grade may be required for the planned underground structures. The planned excavations will likely require vertical cuts and shoring due to the proximity of the planned structures to existing buildings and limited construction space within the public right-of-way. Various temporary shoring systems such as sheet piles, cantilevered and braced soldier piles and lagging, tangent piles, and slurry walls supported by tieback anchors and/or internal bracing struts may be installed as shoring.

Based on local practice under similar subsurface conditions, soldier piles and lagging walls with tiebacks and/or internal bracing (struts and wales) are the most likely shoring systems. The use of slurry wall construction for support of excavations in lieu of soldier piles and lagging (and dewatering) may be another alternative. Driving of sheet piles may be difficult due to the density and stiffness of subsurface materials and may not be permitted due to the noise generated by driving equipment.

The recommended lateral earth pressures for shoring design as well as various other design and installation considerations are provided in the section below. The following information on the design and installation of the shoring is as complete as possible at this time.

8.6.4.1 Lateral Pressures

For excavation heights of 15 feet or less, the use of cantilevered shoring may be an economical option. For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot.

For heights of shoring greater than 15 feet, the use of braced or tied-back shoring is typically more cost-effective. For the design of tied-back or braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated on Figure 8.6-1, Earth Pressure for Tied-back or Braced Shoring, with the maximum pressure equal to 30H in pounds per square foot, where H is the height of the shoring in feet.

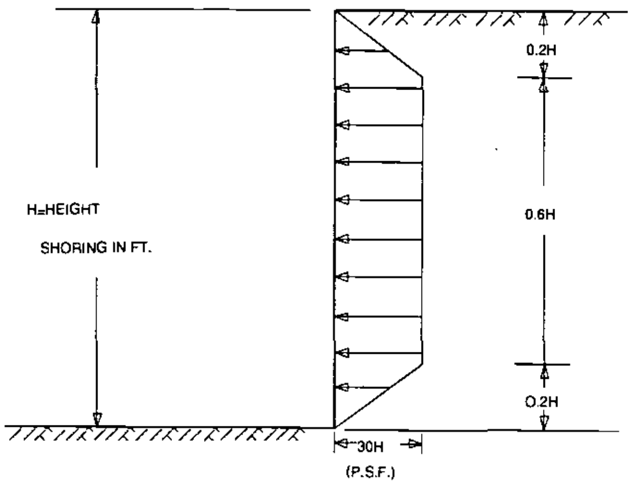


Figure 8.6-1: Earth Pressure for Tied-back or Braced Shoring

The above recommended lateral earth pressures assume a level backfill and a dewatered condition. If the ground surface retained by shoring is sloped at 1:1, 1½:1, or 2:1 (horizontal to vertical), the pressures presented above should be multiplied by 2.0, 1.65, and 1.5, respectively. We can review specific backfill cases if desired.

For secant piles and other relatively impermeable shored excavation walls, if the groundwater level extends above the bottom of the excavation, hydrostatic pressure should be accounted for by applying an equivalent fluid pressure of 45 pounds per cubic foot to the height of wall below the groundwater level. (Excavations extending below the groundwater table will probably be dewatered in which case hydrostatic pressures will not impact shoring design.)

In addition to the recommended earth pressure, shoring adjacent to vehicular traffic, construction equipment, and existing structures should be designed to resist a lateral surcharge pressure. The surcharge pressure can be estimated using Figure 8.6-2, Lateral Surcharge Pressures from Adjacent Vertical Loading. For normal automobile traffic, a vertical pressure of 300 pounds per square foot may be used to calculate the estimated surcharge. For heavier loads, the actual vertical loads should be used to calculate the estimated surcharge.

<u>For Strip Loads:</u>

$$p = \frac{2q}{\pi}(\beta - \sin\beta\cos 2\alpha)$$

<u>For Line Loads:</u>

for $m \leq 0.4$

$$p = \frac{0.2q_e}{H}\,\frac{n}{(0.16+n^2)^2}$$

for $m > 0.4$

$$p = \frac{1.28q_e}{H}\,\frac{m^2 n}{(m^2+n^2)^2}$$

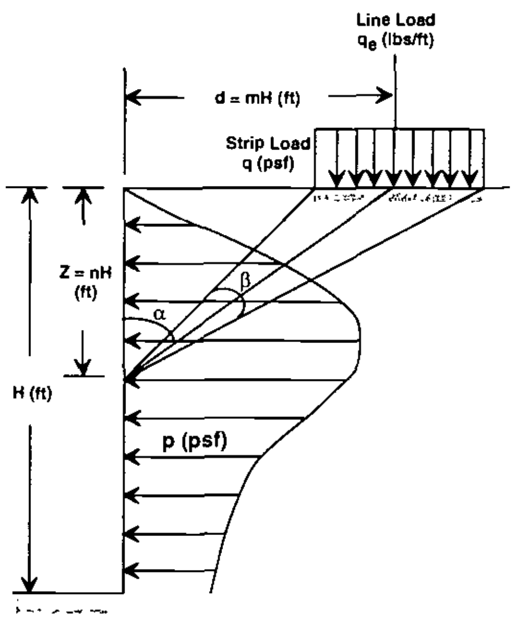


Figure 8.6-2: Lateral Surcharge Pressures from Adjacent Vertical Loading

Considering that shoring is expected to remain in place for several years during construction, we recommend that shoring be designed to support a seismic active pressure due to the ODE event. To represent the seismic active pressure induced on shoring by the ODE event, a uniform lateral surcharge pressure of 8H in pounds per square foot may be used, where H is the height of shoring.

8.6.4.2 Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 6,000 pounds per square foot. For soils below the groundwater level, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 400 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 4,000 pounds per square foot. A one-third increase in the lateral bearing value may be used when considering seismic and other transient loads.

A pile spacing of 6 to 8 feet is typical for local practice. A pile spacing greater than 8 feet is not recommended.

To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load (if applicable). The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.) In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads.

For preliminary design, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be assumed to increase with embedment depth. A frictional resistance of 300 pounds per square foot may be used for the first 10 feet of embedment. A frictional resistance of 500 pounds per square foot may be used for the next 10 feet of embedment (depths of 10 feet to 20 feet) and a maximum frictional resistance of 700 pounds per square foot may be used for deeper sections. The end bearing developed at the pile tip should be neglected. In developing the frictional resistance presented above, we have assumed that the groundwater level will be at or near the excavation level.

The design parameters presented above for the preliminary design should be verified for the shoring system selected.

8.6.4.3 Lagging

Continuous lagging will probably be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure discussed in the preceding sections. However, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for the earth pressure discussed in the preceding sections but limited to a maximum value of 400 pounds per square foot for center-to-center pile spacing of 8 feet or less.

8.6.4.4 Anchor Design

Tie-back friction anchors may be used to resist lateral loads. Installing tie-back anchors in the project area will likely require permission from local agencies and owners of adjacent buildings and avoidance of underground obstructions such as basements, foundations, and utility lines. Tie-back anchors in the public right-of-way will probably require removal to depths of at least 20 feet BGS.

For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. For anchors within the lower 30 feet of the excavation, anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities. Above the lower 30 feet of the excavation, anchors should extend at least 35 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in the section below on Anchor Testing. For design purposes, we estimate that drilled friction anchors will develop an average friction value of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

8.6.4.5 Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

8.6.4.6 Anchor Testing

The details regarding anchor testing should be developed after the shoring requirements are finalized. Generally, an appropriate percentage of the total anchors in each excavation should be selected for 24-hour 200% tests, and for quick 200% tests. The purpose of the 200% tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The rest of the anchors should be proof-tested to 150% of the design capacity.

8.6.4.7 Internal Bracing

Internal struts or raker bracing may be used to internally brace the soldier piles. The strut loads should be determined based on the lateral pressures for restrained conditions presented above. The vertical spacing between struts should be designed to reduce ground movements. All struts should be preloaded to eliminate any slack and to reduce ground movement.

If necessary to reduce shoring deflection, a preload of 25 percent of the design load may be used. However, it must be noted that a preload of 25 percent of the 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.

If used, raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of such temporary footings, poured with the bearing surface normal to the rakers, which are assumed to be inclined at 45 to 60 degrees with the vertical, a bearing value of 3,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

8.6.4.8 Combined Shoring System

If a shoring system which combines 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 relative stiffness and deflection characteristics of the support elements which may induce substantially different load distributions. In some cases, tangent piles may be used to reduce ground loss during excavation and to provide stiffer excavation walls.

8.6.4.9 Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment, which is dependent on the flexibility of the shored wall, excavation methods, and spacing of support members such as soldier piles, struts, etc. It should be realized, however, that some deflection will occur. For properly installed tied-back shoring, we estimate that the deflection at the top of the shored embankment could be on the order of 0.002 times the height of the excavation, which is about 1½ inches for a 60-foot-high excavation. Braced shoring will typically deflect less than tied-back shoring. Deflection of braced shoring is highly dependent on strut spacing, strut stiffness, and whether preloading is performed.

If deflection greater than anticipated occurs during construction, additional bracing may be necessary to minimize settlement of adjacent structures. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design. Also, shoring braced by internal struts and/or rakers should significantly reduce the shoring deflection if properly designed and installed.

8.6.4.10 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops and underlying intermediate points of all the soldier piles as well as installation and regular readings of inclinometers. An excavation-specific instrumentation and monitoring program should be implemented.

8.6.5 Excavation Heave

The maximum excavation depth for the underground structures is expected to be up to approximately 65 feet, which would relieve about 8,000 pounds per square foot of stress from the bottom of the excavation. We estimate that the maximum excavation heave due to elastic rebound will be about 1 inch. Elastic rebound will take place essentially instantaneously during excavation. The consolidation rebounds are estimated to be about ½ inch. The bulk of the consolidation rebound is anticipated to take place within 6 to 8 months following the excavation. The potential for rupture of the excavation bottom due to excessive heave is low.

If construction of the underground structures begins shortly after excavation, a portion of the consolidation rebound will be negated by the settlement of the structure. The estimated settlement of underground structures is discussed in Section 8.6.7, Foundation Recommendations.

8.6.6 Tunnel Box Structures and Retaining Walls

We understand that the cut-and-cover tunnel box structures as well as the portal walls will be designed and constructed as a relatively rigid and water-tight elements. Vertical loads from soil cover and anticipated traffic and other live loads should be determined and added to the roof loading of the cut-and-cover tunnel section. A unit weight of 125 pounds per cubic foot may be assumed for the soil cover over the tunnel section.

8.6.6.1 Lateral Pressures

For design of permanent below grade walls less than 15 feet below grade where the ground surface behind the wall is level, it may be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. For deeper permanent walls up to 60 feet below grade where the surface of the backfill is level, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the wall, is as illustrated on Figure 8.6-1, Earth Pressure for Tied-back or Braced Shoring, with a maximum pressure equal to 30H in pounds per square foot, where H is the height of the shoring in feet.

We understand that higher design lateral earth pressures are desired for added conservatism and to conform to the MTA design criteria for "long-term" loads. Accordingly, if desired, a triangular lateral earth pressure derived from the effective vertical soil pressure times the at-rest soil pressure coefficients presented in Tables 8.6-1a through 8.6-1c may be used for long-term design of permanent below grade walls.

Hydrostatic pressure based on the appropriate design groundwater level discussed in Section 8.2, Design Groundwater Levels, should be included. Hydrostatic pressure should be accounted for by applying an equivalent fluid pressure of 45 pounds per cubic foot per foot of height above the bottom of the wall.

In addition to the recommended earth pressure, below grade walls adjacent to vehicular traffic and existing or new structures should be designed to resist a lateral surcharge pressure. Lateral surcharge pressures on the wall from the foundations of adjacent structures may be reduced with proper underpinning (or alternative protection/isolation methods) of those foundations.

The surcharge pressure can be estimated using Figure 8.6-2, Lateral Surcharge Pressures from Adjacent Vertical Loading. For normal automobile traffic, a vertical pressure of 300 pounds per square foot may be used to calculate the estimated surcharge. For heavier loads, the actual vertical loads should be used to calculate the estimated surcharge. The surcharge pressures imposed by the mat foundation of the planned mezzanine level extensions at the First/Boyle and First/Soto stations should be accounted for in the design of the adjacent walls of the lower platform level.

In addition to the above-mentioned lateral earth pressures, retaining walls more than 6 feet high with an unbalanced earth condition should be designed to support a seismic active pressure. Tunnel walls and rigidly braced walls need not be designed for seismic lateral earth pressures unless an unbalanced earth condition exists. An unbalanced earth condition is defined as a difference in backfill height or elevation between opposite tunnel walls. To represent earthquake-induced surcharge pressures, uniform lateral surcharge pressures of 10H and 20H in pounds per square foot, where H is difference in backfill height, may be used for the ODE and MDE, respectively.

As discussed in Section 7.4, Estimation of Racking for Design, we recommend that the design of tunnel walls and rigidly braced walls account for the expected racking induced by ground motions. For the ODE, the racking may be taken as ¼ inch in 45 feet of height difference. For the MDE, the racking may be taken as 1 inch in 45 feet. The racking values may be linearly extrapolated for other structure heights.

8.6.6.2 Backfill

Any required soil backfill should be mechanically compacted, in layers not more than 8 inches thick, to at least 95% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. The backfill should be sufficiently impermeable when compacted to restrict the inflow of surface water. Some settlement of the deep backfill should be allowed for in planning utility connections and overlying concrete hardscape.

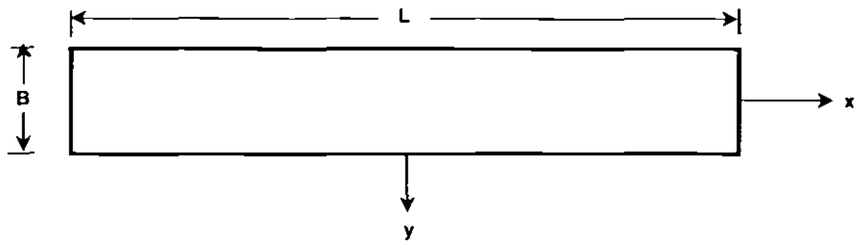
8.6.7 Foundation Recommendations

The natural soils at and below the planned excavation levels are dense and stiff, and the planned structures may be supported on spread-type footings or mat foundations established in the dense and stiff natural soils exposed at the bottom of the planned excavations. We understand that the rail lines and walls of the portal structures, the cut-and-cover tunnel section, and the stations are planned to be supported on thick concrete slabs that will function as relatively rigid mat foundations. The design of mat foundations is typically governed by settlement. Available information indicates that the average bearing pressure on the mat foundations would be less than the overburden removed by the excavation, which mitigates settlement.

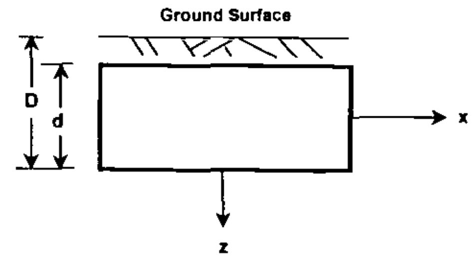
Upon unloading (excavation) and loading (structure construction), the underlying alluvium and bedrock will respond in terms of heave and settlement. The granular alluvium below the planned underground structures will undergo little or no time-dependent, long-term heave and consolidation responses. The fine-grained alluvium, however, will exhibit a certain amount of time dependent heave and consolidation response.

For the mat foundation design and soil-structure interaction analysis, separate soil spring parameters for coarse-grained alluvium and fine-grained alluvium are provided for static and dynamic conditions. Coefficients of subgrade reaction presented in Tables 8.6-1a through 8.6-1c may be used as soil spring constants for static conditions. Dynamic soil spring constants for the ODE and MDE are presented in Tables 8.6-2 through 8.6-4, Elasto-Plastic Soil Spring Constants.

Table 8.6-2a: Elasto-Plastic Soil Spring Constants
First/Boyle Station - Main Box



PLAN VIEW



SECTION

Length of Main Station Box (L) = 420 feet
Width of Main Station Box (B) = 45 feet

Depth of Station Invert (D) = 50 feet
Height of Station Wall (d) = 45 feet

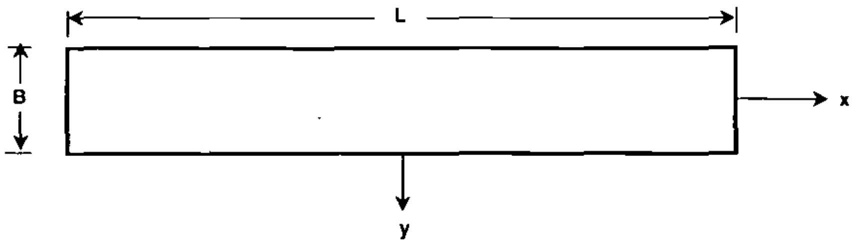
k_x, k_y, k_z = Elasto-Plastic Soil Spring Constants in x, y, and z directions, respectively

Loading Case	Best Estimate of Soil Spring Constant (Due to Bearing and Friction on Bottom Slab)									Best Estimate of Soil Spring Constant (Due to Embedment Effects)											
	k_{xb} (slab friction)			k_{yb} (slab friction)			k_{zb} (bearing resistance)			k_{xe} (wall friction)			k_{xe} (passive resistance)			k_{ye} (passive resistance)			k_{ze} (wall friction)		
	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[a]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[b]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[c]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[d]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[e]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[f]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[g]	Maximum Elastic Deflection (inch)
ODE[h]	2.66	59200	0.5N[i]	3.23	7700	0.5N	3.90	200	2.7	1.21	480(z)	0.3	1.21	21.6(z)	0.25	2.93	5.6(z)	0.95	1.97	2100	0.2
MDE[j]	1.49	33100	0.5N	1.81	4300	0.5N	2.18	110	4.9	0.68	270(z)	0.5	0.68	12.1(z)	0.4	1.64	3.2(z)	1.7	1.10	1200	0.35

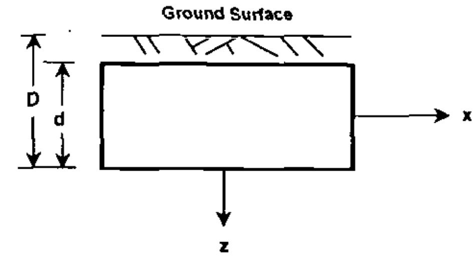
[a]Distribute along short side (B) of bottom slab
[b]Distribute along long side (L) of bottom slab
[c]Distribute over area (LxB) of bottom slab
[d]Distribute along each side (d) of end wall, unit stiffness increases with depth (z, in feet)
[e]Distribute over area of end wall (Bxd), unit stiffness increases with depth (z, in feet)
[f]Distribute over area of side wall (Lxd), unit stiffness increases with depth (z, in feet)
[g]Distribute along perimeter of bottom slab (2L+2B)
[h]Degraded stiffness corresponding to cyclic shear strains expected from the Operating Design Earthquake
[i]Total Normal Force on Bottom Slab
[j]Degraded stiffness corresponding to cyclic shear strains expected from the Maximum Design Earthquake

MTA Eastside LRT Project

Table 8.6-2b: Elasto-Plastic Soil Spring Constants
First/Boyle Station - Mezzanine, and West CC Tunnel



PLAN VIEW



SECTION

Length of Main Station Box (L) = 160 feet
Width of Main Station Box (B) = 50 feet

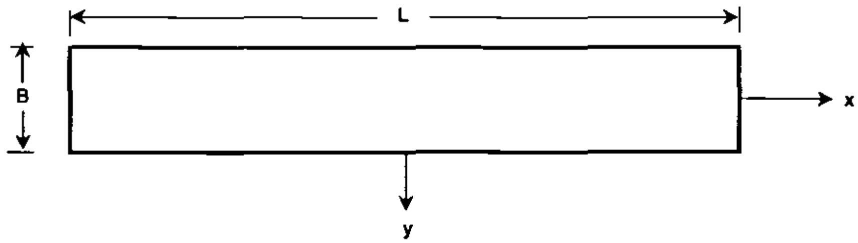
Depth of Station Invert (D) = 30 feet
Height of Station Wall (d) = 25 feet

k_x, k_y, k_z = Elasto-Plastic Soil Spring Constants in x, y, and z directions, respectively

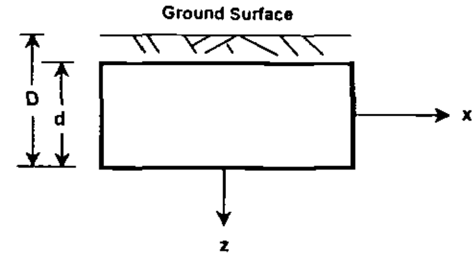
Loading Case	Best Estimate of Soil Spring Constant (Due to Bearing and Friction on Bottom Slab)									Best Estimate of Soil Spring Constant (Due to Embedment Effects)											
	k_{xb} (slab friction)			k_{yb} (slab friction)			k_{zb} (bearing resistance)			k_{xe} (wall friction)			k_{xe} (passive resistance)			k_{ye} (passive resistance)			k_{ze} (wall friction)		
	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[a]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[b]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[c]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[d]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[e]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[f]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[g]	Maximum Elastic Deflection (inch)
ODE[h]	0.75	15000	0.5N[i]	0.83	5200	0.5N	1.02	120	3	0.31	340(z)	0.15	0.31	13.9(z)	0.4	0.68	9.6(z)	0.55	0.35	800	0.15
MDE[j]	0.42	8400	0.5N	0.46	2900	0.5N	0.57	70	5.1	0.17	190(z)	0.25	0.17	7.8(z)	0.6	0.38	5.4(z)	0.95	0.20	450	0.3

[a]Distribute along short side (B) of bottom slab
[b]Distribute along long side (L) of bottom slab
[c]Distribute over area (LxB) of bottom slab
[d]Distribute along each side (d) of end wall, unit stiffness increases with depth (z, in feet)
[e]Distribute over area of end wall (Bxd), unit stiffness increases with depth (z, in feet)
[f]Distribute over area of side wall (Lxd), unit stiffness increases with depth (z, in feet)
[g]Distribute along perimeter of bottom slab (2L+2B)
[h]Degraded stiffness corresponding to cyclic shear strains expected from the Operating Design Earthquake
[i]Total Normal Force on Bottom Slab
[j]Degraded stiffness corresponding to cyclic shear strains expected from the Maximum Design Earthquake

Table 8.6-3a: Elasto-Plastic Soil Spring Constants
First/Soto Station - Main Box



PLAN VIEW



SECTION

Length of Main Station Box (L) = 340 feet
Width of Main Station Box (B) = 60 feet

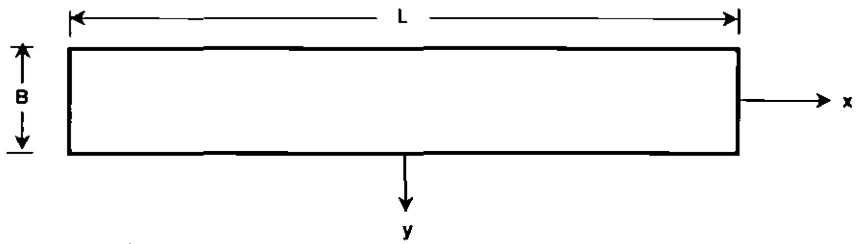
Depth of Station Invert (D) = 55 feet
Height of Station Wall (d) = 45 feet

k_x, k_y, k_z = Elasto-Plastic Soil Spring Constants in x, y, and z directions, respectively

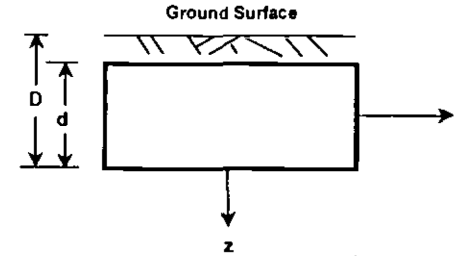
	Best Estimate of Soil Spring Constant (Due to Bearing and Friction on Bottom Slab)									Best Estimate of Soil Spring Constant (Due to Embedment Effects)											
	k_{xb} (slab friction)			k_{yb} (slab friction)			k_{zb} (bearing resistance)			k_{xe} (wall friction)			k_{xe} (passive resistance)			k_{ye} (passive resistance)			k_{ze} (wall friction)		
Loading Case	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[a]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[b]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[c]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[d]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[e]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[f]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[g]	Maximum Elastic Deflection (inch)
ODE[h]	2.03	33700	0.5N[i]	2.34	6900	0.5N	2.85	140	3.9	0.93	310(z)	0.2	0.93	10.6(z)	0.4	2.15	4.3(z)	0.75	1.24	1530	0.15
MDE[j]	1.13	18900	0.5N	1.31	3800	0.5N	1.60	80	6.8	0.52	175(z)	0.35	0.52	5.9(z)	0.7	1.20	2.4(z)	1.3	0.69	850	0.3

[a] Distribute along short side (B) of bottom slab
[b] Distribute along long side (L) of bottom slab
[c] Distribute over area (LxB) of bottom slab
[d] Distribute along each side (d) of end wall, unit stiffness increases with depth (z, in feet)
[e] Distribute over area of end wall (Bxd), unit stiffness increases with depth (z, in feet)
[f] Distribute over area of side wall (Lxd), unit stiffness increases with depth (z, in feet)
[g] Distribute along perimeter of bottom slab (2L+2B)
[h] Degraded stiffness corresponding to cyclic shear strains expected from the Operating Design Earthquake
[i] Total Normal Force on Bottom Slab
[j] Degraded stiffness corresponding to cyclic shear strains expected from the Maximum Design Earthquake

Table 8.6-3b: Elasto-Plastic Soil Spring Constants
First/Soto Station - Mezzanine



PLAN VIEW



SECTION

Length of Main Station Box (L) = 135 feet
Width of Main Station Box (B) = 55 feet

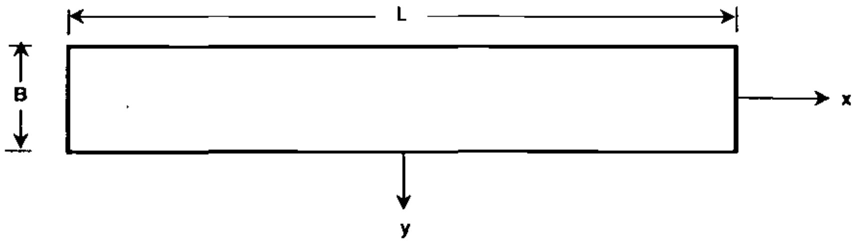
Depth of Station Invert (D) = 35 feet
Height of Station Wall (d) = 25 feet

k_x, k_y, k_z = *Elasto-Plastic Soil Spring Constants in x, y, and z directions, respectively*

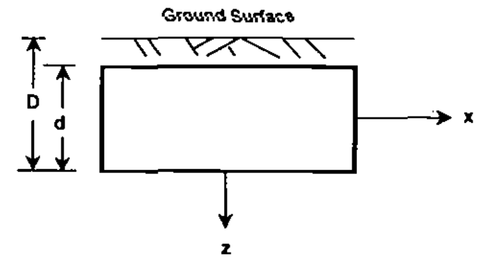
Loading Case	Best Estimate of Soil Spring Constant (Due to Bearing and Friction on Bottom Slab)									Best Estimate of Soil Spring Constant (Due to Embedment Effects)											
	k_{xb} (slab friction)			k_{yb} (slab friction)			k_{zb} (bearing resistance)			k_{xe} (wall friction)			k_{xe} (passive resistance)			k_{ye} (passive resistance)			k_{ze} (wall friction)		
	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[a]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[b]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[c]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[d]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[e]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft²/ft)[f]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[g]	Maximum Elastic Deflection (inch)
ODE[h]	0.59	10700	0.5N[i]	0.64	4700	0.5N	0.79	100	3.6	0.28	240(z)	0.1	0.28	8.9(z)	0.2	0.59	7.8(z)	0.25	0.28	720	0.2
MDE[j]	0.33	6000	0.5N	0.36	2600	0.5N	0.44	60	6	0.15	135(z)	0.2	0.15	5.0(z)	0.4	0.33	4.4(z)	0.45	0.15	400	0.35

[a]Distribute along short side (B) of bottom slab
[b]Distribute along long side (L) of bottom slab
[c]Distribute over area (LxB) of bottom slab
[d]Distribute along each side (d) of end wall, unit stiffness increases with depth (z, in feet)
[e]Distribute over area of end wall (Bxd), unit stiffness increases with depth (z, in feet)
[f]Distribute over area of side wall (Lxd), unit stiffness increases with depth (z, in feet)
[g]Distribute along perimeter of bottom slab (2L+2B)
[h]Degraded stiffness corresponding to cyclic shear strains expected from the Operating Design Earthquake
[i]Total Normal Force on Bottom Slab
[j]Degraded stiffness corresponding to cyclic shear strains expected from the Maximum Design Earthquake

Table 8.6-4: Elasto-Plastic Soil Spring Constants
East CC Tunnel



PLAN VIEW



SECTION

Length of Main Station Box (L) = 210 feet
Width of Main Station Box (B) = 50 feet

Depth of Station Invert (D) = 25 feet
Height of Station Wall (d) = 25 feet

k_x, k_y, k_z = Elasto-Plastic Soil Spring Constants in x, y, and z directions, respectively

Loading Case	Best Estimate of Soil Spring Constant (Due to Bearing and Friction on Bottom Slab)									Best Estimate of Soil Spring Constant (Due to Embedment Effects)											
	k_{xb} (slab friction)			k_{yb} (slab friction)			k_{zb} (bearing resistance)			k_{xe} (wall friction)			k_{xe} (passive resistance)			k_{ye} (passive resistance)			k_{ze} (wall friction)		
	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[a]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[b]	Maximum Load (kips)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[c]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[d]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[e]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft^2/ft)[f]	Maximum Elastic Deflection (inch)	Single Spring (10^6 kips/ft)	Distributed Springs (kips/ft/ft)[g]	Maximum Elastic Deflection (inch)
ODE[h]	1.23	24400	0.5N[i]	1.38	6600	0.5N	1.69	160	2.3	0.39	625(z)	0.1	0.39	25.0(z)	0.2	0.88	13.4(z)	0.4	0.52	1000	0.1
MDE[j]	0.69	13700	0.5N	0.77	3700	0.5N	0.94	90	4	0.22	350(z)	0.2	0.22	14.0(z)	0.4	0.49	7.5(z)	0.7	0.29	560	0.2

[a]Distribute along short side (B) of bottom slab
[b]Distribute along long side (L) of bottom slab
[c]Distribute over area (LxB) of bottom slab
[d]Distribute along each side (d) of end wall, unit stiffness increases with depth (z, in feet)
[e]Distribute over area of end wall (Bxd), unit stiffness increases with depth (z, in feet)
[f]Distribute over area of side wall (Lxd), unit stiffness increases with depth (z, in feet)
[g]Distribute along perimeter of bottom slab (2L+2B)
[h]Degraded stiffness corresponding to cyclic shear strains expected from the Operating Design Earthquake
[i]Total Normal Force on Bottom Slab
[j]Degraded stiffness corresponding to cyclic shear strains expected from the Maximum Design Earthquake

MTA Eastside LRT Project

If groundwater levels rise from current levels to the design levels discussed in Section 8.2, Design Groundwater Levels (up to a maximum of 25 feet above current levels), the downward vertical pressure imposed by the mat foundations will decrease, which may induce heave due to elastic and consolidation rebounds. We estimate that the corresponding total elastic and consolidation rebound of the mat foundations will be about 1 inch or less.

8.6.7.1 Bearing Values

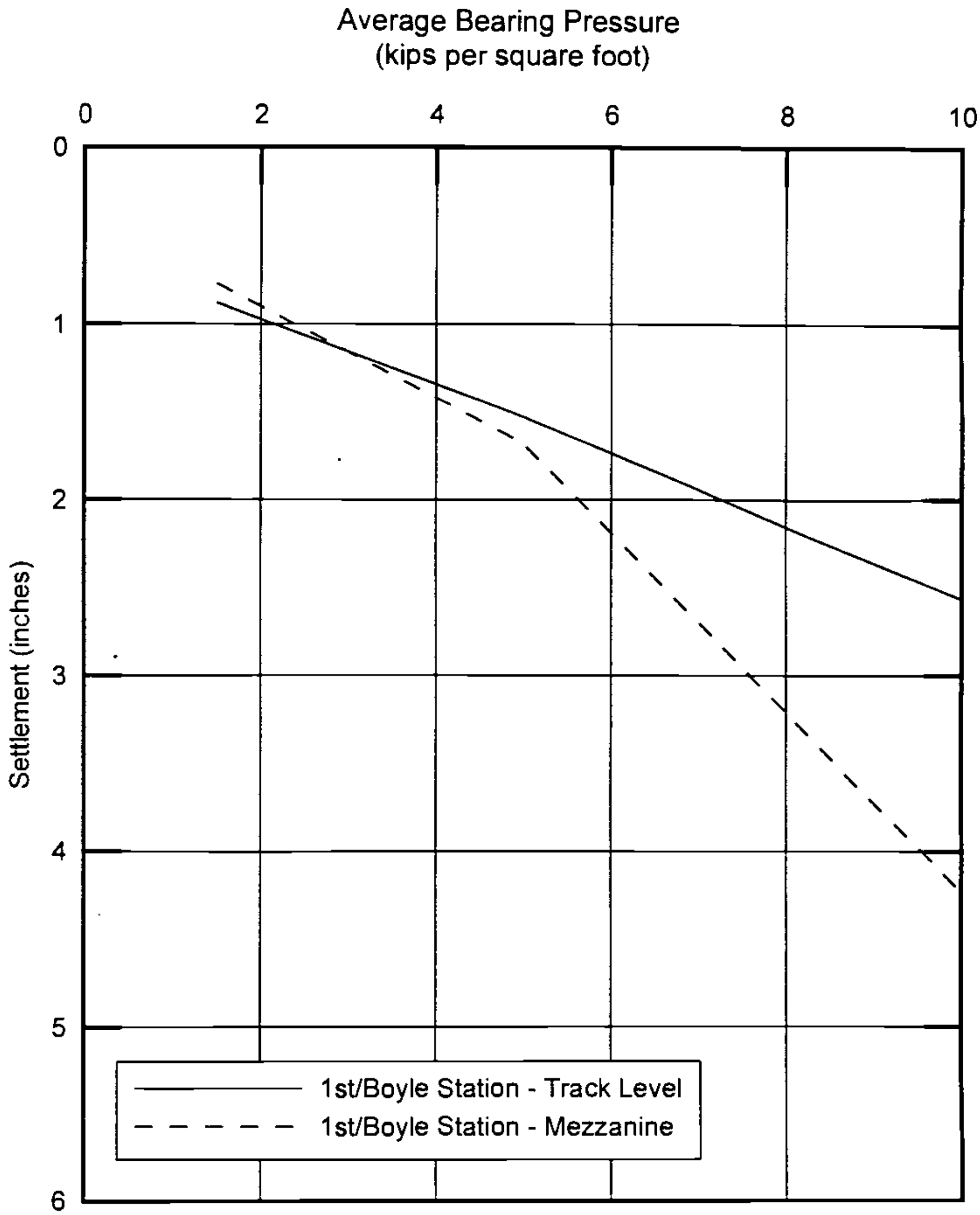
Mat foundations for the stations and mezzanine (entrance) structures established in the stiff and dense natural soils, may be designed to impose a net dead-plus-live load pressure of up to 10,000 pounds per square foot. A one-third increase in the bearing value may be used when considering wind or seismic loads.

Mat foundations for the cut-and-cover tunnel and portal structures may be designed to impose a net dead-plus-live load pressure of 2,500 pounds per square foot or the weight of soil removed by the excavation, whichever is greater. The weight of soil may be assumed as 125 pounds per cubic foot. A one-third increase in the bearing value may be used when considering wind or seismic loads.

Footings for minor structures at or near the existing ground surface that are at least 2 feet wide and are established in properly compacted fill and/or undisturbed natural soils, may be designed to impose a net dead-plus-live load pressure of 2,500 pounds per square foot. A one-third increase in the bearing value may be used when considering wind or seismic loads. Footings should extend at least 2 feet below the adjacent final grade or floor level.

8.6.7.2 Settlement

The total settlement of the proposed stations and mezzanine structures supported on mat foundations in the manner recommended is shown on Figures 8.6-3a through 8.6-3b, Estimated Settlement of Mat Foundation. Over half of the total settlement is anticipated to occur shortly after dead loads are imposed. For mat foundations with average bearing pressures of up to 5,000 pounds per square foot, the long-term consolidation settlement is not expected to exceed about ½ inch. Most of the consolidation settlement is anticipated to take place within 6 to 8 months after construction.



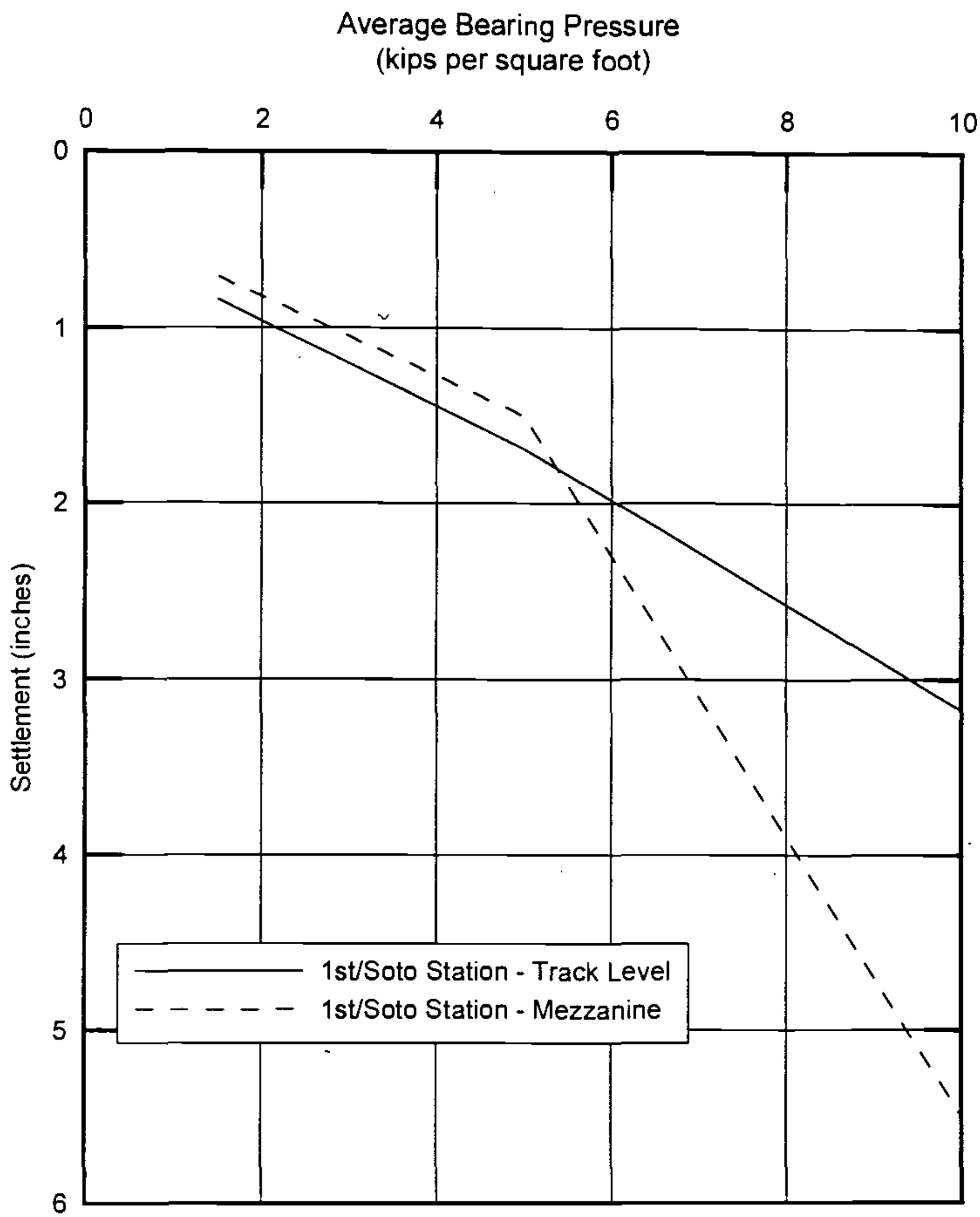
ESTIMATED SETTLEMENT OF MAT FOUNDATION

1st/Boyle Station
MTA EASTSIDE LRT PROJECT
Underground Segment

Project No. 70131-1-0138.0002 Client: Eastside LRT Partners Date: 12/07/01 Drafter: SB OE: CK Reviewer: CK

Law/Crandall

FIGURE 8.6-3a



ESTIMATED SETTLEMENT OF MAT FOUNDATION

1st/Soto Station
MTA EASTSIDE LRT PROJECT
Underground Segment

Law/Crandall

FIGURE 8.6-3b

Project No. 70131-1-0138.0002 Client: Eastside LRT Partners Date: 12/07/01 Drafter: SB OE: CK Reviewer: CK

The total settlement of the cut-and-cover tunnel section and portal structures supported on mat foundations in the manner recommended is estimated to be on the order of 1 inch or less. Over half of the total settlement is anticipated to occur shortly after dead loads are imposed. Most of the consolidation settlement is anticipated to take place within 3 to 6 months after construction.

Some differential settlement within the foundation mat should be anticipated. Differential settlements are expected to be less than ½ inch in 50 feet. If more refined deflection analysis is desired, the coefficients of subgrade reaction presented in Tables 8.6-1a thorugh 8.6-1c may be used.

The total settlement of minor structures, supported on spread footings in the manner recommended is estimated to be on the order of 1 inch or less. Most of the settlement is expected to occur during construction.

8.6.7.3 Lateral Loads

Lateral loads may be resisted by soil friction against the mats and/or footings, and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between concrete and the supporting soils. The passive resistance of the undisturbed natural soils or properly compacted fill against footings and station box walls may be assumed to be 300 pounds per cubic foot if above the groundwater level. For soils below the groundwater level, an equivalent fluid pressure of 150 pounds per cubic foot should used instead. A one-third increase in the passive value may be used for wind or seismic loads.

The passive resistance of the soils and the frictional resistance between mats and footings and the supporting soils may be combined without reduction in determining the total lateral resistance.

8.6.7.4 Soil Spring Constants and Other Geotechnical Design Parameters

Tables 8.6-1a through 8.6-1c summarize recommended geotechnical design parameters. These design parameters include lateral earth pressure coefficients, unit weight of soil, shear strength, long term maximum groundwater level, coefficient of subgrade reaction for shallow foundations, and seismic ground motion parameters. Recommended dynamic soil spring constants for design of underground structures are presented in Tables 8.6-2 through 8.6-4.

8.6.8 Earthwork

Based on the subsurface conditions encountered in this and prior investigations, the excavation of the portals and the cut-and-cover tunnel section can be accomplished relatively rapidly by conventional excavation equipment. Earthwork and site preparation activities are expected to consist of: excavations for subterranean structures; subgrade preparation for the tunnel box floor; foundation preparation for near-surface structures; excavations for utility trenches; subgrade preparation for paving; and backfill behind cut-and-cover tunnel walls, footings and utility trenches.

Due to space limitations, major excavations will probably require temporary shoring according to the recommendations presented in Section 8.6.4, Shored Excavation. All work should be in compliance with applicable city (Los Angeles), state (California), and federal (Occupational Safety and Health Act) requirements.

Materials excavated from non-contaminated granular alluvium (sand, silty sand, gravelly sand, sandy gravel, and gravel) may be stockpiled to be reused as backfill material. Excavated fine-grained alluvium or bedrock is not considered 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 licensed geotechnical engineer in the State of California.

8.6.9 Uniform Building Code Seismic Coefficients

Based on our review of the soil conditions along the alignment, the average shear wave velocities of the upper 100 feet at the underground station and portal locations correspond to the transition between S_C and S_D Soil Profile Types. However, to be appropriately conservative and for better consistency with the structures that will be designed using dynamic analyses, we recommend that at-grade structures be designed for higher ground motions using Soil Profile Type S_D as specified in Section 1629 of the Uniform Building Code (UBC), 1997 edition.

The site is located within UBC Seismic Zone 4.

According to Map M-32 in the 1998 publication from the International Conference of Building Officials entitled "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," the nearest seismic source to the alignment are the Hollywood and Raymond Faults. Both faults are classified as Type B seismic sources. The near source factors, N_a and N_v, based on Tables 16-S and 16-T of the 1997 UBC for each station location are presented in the table below.

Structure	**Soil Profile Type**	**Nearest Seismic Source**	**Seismic Source Type**	**Distance to Seismic Source (km)**	N_a	N_v
First/Boyle Station	S_D	Hollywood/Raymond	B	8	1.0	1.08
First/Soto Station	S_D	Hollywood/Raymond	B	8.8	1.0	1.05
East Portal	S_D	Hollywood/Raymond	B	9.6	1.0	1.02

8.7 PROTECTION OF EXISTING FACILITIES

The tunnels, portal structures, and stations are located within existing residential and commercial areas with 1- and 2-story buildings and aboveground and underground infrastructure, including two freeway overpasses. The tunnels and stations are generally within the public right-of-way. However, there will be a number of structures within the zone of influence of the tunnel and station excavations.

Although proper design and careful installation of shoring can typically mitigate potential disturbances to structures adjacent to excavations, the need for additional protection measures will ultimately depend on excavation methods, existing building foundations, and whether the buildings can satisfactorily accommodate the expected settlement due to excavation-related deformation.

Existing structures adjacent to the First/Soto Station may undergo settlement due to dewatering. The settlement is estimated to decrease linearly from about 1 inch adjacent to the excavation perimeter to zero at a distance of 50 feet away.

Jacobs Associates is currently performing a survey of existing buildings and infrastructure likely to be impacted by the project to help identify structures that may require the implementation of additional protection measures. Jacobs Associates will also evaluate the potential settlement of existing structures due to ground loss during tunneling.

9.0 BIBLIOGRAPHY

Abrahamson, N., 1998, "RSPMATCH: Non-Stationary Spectral Matching Program".

Abrahamson, N. and Shedlock, K., *editors*, "1997 Ground Motion Attenuation Relationships," Seismological Research Letters, Vol. 68, No. 1, January 1997 special issue.

Blake, 1995, revised 1998, "FRISKSP, A Computer Program for Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources".

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

Boore, D. et al., 1994, "Estimation of Response Spectra and Peak Accelerations From Western North American Earthquakes: An Interim Report, Part 2," U.S. Geological Survey Open-File Report 94-127.

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

California Department of Water Resources, 1966, "Planned Utilization of Ground Water Basin, San Gabriel Valley," Bulletin 104-2, Appendix A.

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

California Department of Water Resources, 1988, Hydrologic Data on Microfiche.

California Division of Mines and Geology, 1999, "Seismic Shaking Hazard Maps of California," Map Sheet 48.

California Division of Mines and Geology, 1999, "State of California Seismic Hazard Zones, Los Angeles Quadrangle, Official Map," released March 25, 1999.

California Division of Mines and Geology, 1998, "Seismic Hazard Evaluation of the Los Angeles 7.5 Minute Quadrangle, Los Angeles County, California," Open File Report 98-20."

California Division of Mines and Geology, 1997, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117.

California Division of Mines and Geology, 1996, "Probabilistic Seismic Hazard Assessment for the State of California," Open File Report 96-08.

California Division of Mines and Geology, 1986, "Faults and Earthquakes in California," DMG Note 31.

California Division of Mines and Geology, 1982, "Guidelines for Geologic/Seismic Considerations in Environmental Impact Reports," DMG Note 46.

California Division of Oil, Gas and Geothermal Resources, 1996, "Boyle Heights, Los Angeles Downtown, Union Station Oil Field Maps, Los Angeles County," Map No. 119.

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

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

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

Converse Davis and Associates, 1972, "Groundwater Investigation, Vicinity of Forest Lawn, Glendale–Los Angeles, California," Drawings 1 and 2, CDA Project 70-044-EH.

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

Cordoba Corporation, 1994, "Analysis of the Eastern Extension Alignment Addendum," internal memorandum from Byron M. Ishkanian to Genoveva Arellano, dated January 11, 1994.

Cordoba Corporation, 1993, "Analysis of the Eastern Extension Alignment," internal memorandum from Byron M. Ishkanian to Genoveva Arellano, dated January 6, 1993.

Cramer, C. H. and Petersen, M. D., 1996, "Predominant Seismic Source Distance and Magnitude Maps for Los Angeles, Orange, and Ventura Counties, California," *Bulletin of Seismological Society of America*, Vol. 86, No. 5, pp. 1645-1649.

Crook, R., Jr. and Proctor, R. J., 1992, "The Santa Monica and Hollywood Faults and the Southern Boundary of the Transverse Ranges Province," in Engineering Geology Practice in Southern California.

Crook, R., Jr., Allen, C. R., Kamb, B., Payne, C. M., and Proctor, R. J., 1987, "Quaternary Geology and Seismic Hazard of the Sierra Madre and Associated Faults, Western San Gabriel Mountains," in U.S. Geological Survey Professional Paper 1339, Ch. 2, pp. 27–63.

Crook, R., Jr., Proctor, R. J., and Lindvall, E. E., 1983, "Seismicity of the Santa Monica and Hollywood Faults Determined by Trenching," Technical Report to the U.S. Geological Survey, Contract No. 14-08-001-20523.

Davis, T. L., Namson, S., and Yerkes, R. F.,. 1989, "A Cross Section of the Los Angeles Area: Seismically Active Fold and Thrust Belt, the 1987 Whittier Narrows Earthquake, and Earthquake Hazard," *Journal of Geophysical Research*, Vol. 94, p 9644-9664.

Davis, J. F., Bennett, J. H., Borchardt, G.A., Kahle, J.E., Rice, S.J., Silva, M. A., 1982, "Earthquake Planning Scenario for a Magnitude 8.3 Earthquake on the San Andreas Fault in Southern California," California Division of Mines and Geology Special Publication 60.

Dolan, J. F., Sieh, K., and Rockwell, T. K., 2000a, "Late Quaternary Activity and Seismic Potential of the Santa Monica Fault System, Los Angeles, California," *Geological Society of America Bulletin*, Vol. 112, No. 10, October 2000.

Dolan, J. F., Stevens, D., and Rockwell, T. K., 2000b, "Paleoseismologic Evidence for an Early to Mid-Holocene Age of the Most Recent Surface Fault Rupture on the Hollywood Fault, Los Angeles, California" *Bulletin of the Seismological Society of America*, April 2000.

Dolan, J. F., Sieh, K. E., Rockwell, T. K., Guptill, P., and Miller, G., 1997, "Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California," *Geological Society of America Bulletin*, Vol. 109, No. 12.

Dolan, J. F. et al., 1995, "Prospects for Larger or More Frequent Earthquakes in the Los Angeles Metropolitan Region, California," *Science* 267, 199-205 pp.

Dolan, J. F. and Sieh K., 1993, "Tectonic Geomorphology of the Northern Los Angeles Basin: Seismic Hazards and Kinematics of Young Fault Movement."

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

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

Earth Consultants International, Inc., 2001, "Coyote Pass Escarpment Study, Eastside Light Rail Transit Project, Soto and First Streets, Los Angeles, California," dated August 6, 2001 (ECI Project No. 2019.01).

Engineering Management Consultant/Geotransit Consultants, 1998, "Los Angeles County Metropolitan Transit Authority, Metro Rail Project, Geotechnical Design Summary Report, Tunnels and Little Tokyo/Arts District Station, Contract C0502/C0541," dated June 1998, Project No. 11081.21

Engineering-Science, Inc., 1994, "Subsurface Gas Investigation, Metro Red Line - Eastside Extension, Prepared for Engineering Management Consultant," dated December 7, 1994.

Enviro-Rail, 1998a, "Environmental Summary Report, Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California," dated June 9, 1998.

Enviro-Rail, 1998b, "Stage 2 Supplemental Gas Study (Draft), Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California, Volumes I and II," dated April, 1998.

Enviro-Rail, 1998c, "Stage 1 Supplemental Gas Investigation, Metro Red Line Segment 3, East Side Extension, Contract Unit C0502, Los Angeles, California, Volumes I and II," dated March, 1998.

Enviro-Rail, 1997a, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0531 – Chavez/Soto Station and Contract Unit C0541 – Line Section First/Boyle to First/Lorena," dated May, 1997.

Enviro-Rail, 1997b, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0521 – 1st/Boyle Station," dated May, 1997.

Enviro-Rail, 1997c, "Final Report, Metro Red Line Segment 3 East Side Extension, Phase II Environmental Investigation Report, Contract Unit C0502 – Line Section Union Station to 1st/Boyle and Little Tokyo Station, Volumes I and II," dated May, 1997.

Enviro-Rail, 1995, "Baseline Environmental Study, Metro Red Line Segment 3, Eastside Extension," dated October, 1995.

Fear, C. E., and McRoberts, E. C., 1995, "Reconsideration of Initiation of Liquefaction in Sandy Soils," Journal of Geotechnical Engineering, ASCE Vol. 121, No. 3, pp 249-261.

Federal Insurance Administration, 1988, "Flood Hazard Maps," compiled by Flood Data Systems, Inc.

GeoTransit Consultants, 1996a, "Final Geotechnical Investigation for: First/Boyle Station with Crossover and Little Tokyo – First/Boyle Tunnels, Eastside Extension Metro Red Line Project, Prepared for Engineering Management Consultant," dated June 1996 (Project No. 95-8347-24).

GeoTransit Consultants, 1996b, "Final Geotechnical Investigation for: Chavez/Soto Station & Adjacent Tunnels, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated June 1996 (Project No. 95-8347-24).

GeoTransit Consultants, 1996c, "Results of Aquifer Pump Testing, At Mission Road and E. Fourth Street, Eastside Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated May 15, 1996 (Project No. 95-8347).

GeoTransit Consultants, 1996d, "Final Geotechnical Investigation for: First/Lorena Station & Tail Track Tunnels, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated May 1996 (Project No. 95-8347-15).

GeoTransit Consultants, 1996e, "Draft Final Fault Investigation for: Engineering Design Program, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated February 8, 1996 (Project No. 95-8347-18).

GeoTransit Consultants, 1995, "Final Draft Geotechnical Investigation for: Union-Little Tokyo Tunnels & Little Tokyo Station, East Side Extension, Metro Red Line Project, Volume I of II," dated December 1995 (Project No. 95-8347-04).

GeoTransit Consultants, 1994a, "Geotechnical Investigation for: Preliminary Engineering Program, Eastside Extension, Metro Red Line Project, Prepared for Engineering Management Consultant," dated February 14, 1994 (Project No. 94-1100).

GeoTransit Consultants, 1994b, "Stage II Environmental Site Assessment, Eastside Extension, Metro Red Line project, Prepared for Engineering Management Consultant," dated January 1994 (Project No. 94-1109-02).

Hart, E. W., 1973, revised 1997, "Fault-Rupture Hazard Zones in California," California Division of Mines and Geology Special Publication 42.

Hauksson, E., 1987, "Seismotectonics of the Newport-Inglewood Fault Zone in the Los Angeles Basin, Southern California," *Bulletin of the Seismological Society of America*, Vol. 77, pp. 539–561.

Hauksson, E., 1990, "Earthquakes, Faulting, and Stress in the Los Angeles Basin," *Journal of Geophysical Research*, Vol. 95, No. B10, pp. 15,365–15,394.

Hughes InSitu Engineering, Inc., 1995, "Pressuremeter Data, Eastside Extension Metro Rail Project, Submitted to Geotransit Consultants," dated October 1995.

Hummon, C., Schnieder, C. L., Yeats, R. S., Dolan, J. F., Sieh, K. E., and Huftile, G.J., 1994, "Wilshire Fault: Earthquakes in Hollywood?," *Geology*, Vol. 22, pp. 291-294.

Idriss, I. M. and Sun, J. I., 1992, "User's Manual for SHAKE91, A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits, Program modified based on the Original SHAKE program published in December 1972 by Schnabel, Lysmer & Seed," Center for Geotechnical Modeling Department of Civil & Environmental Engineering, University of California, Davis, California.

Jackson, D. D. et al., 1995, "Seismic Hazards in Southern California: Probable Earthquakes, 1994 to 2024," *Bulletin of the Seismological Society of America*, Vol. 85, No. 2.

Jennings, C. W., 1994, "Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions," California Division of Mines and Geology Map No. 6.

Kramer, S. L., 1996, "Geotechnical Earthquake Engineering," Prentice Hall.

Lamar, D. L., 1970, "Geology of the Elysian Park–Repetto Hills Area, Los Angeles County, California," California Division of Mines and Geology Special Report 101.

Law/Crandall, 2002, "Report of Aquifer Testing and Preliminary Dewatering Recommendations, Eastside LRT Project – Underground Segment, Along First Street, from Clarence Street to Lorena Street, Los Angeles, California," dated March 28, 2002, Project No. 70131-1-0138.

Law/Crandall, 2000, "Report of Fault Rupture Hazard Investigation, 1840 North Highland Avenue, Hollywood District, Los Angeles, California, Prepared for Citistate, Inc.," Project No. 70131-9-0337.0001.

Law/Crandall, 2000, "Report of Environmental Assessment–Metro Red Line Eastside Transit Corridor Study Area, prepared for Eastside Corridor Transit Consultants," Project No. 70131-9-0387.

Law/Crandall, 1997a, "Geotechnical Investigation Data Report, Pressuremeter Testing and Soil Velocity Measurements, East Side Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated April 9, 1997 (Project No. 70131-6-0552.5000).

Law/Crandall, 1997b, "Geotechnical Investigation Report–Large Diameter Borings, East Side Extension, Metro Red Line, Los Angeles, California, Prepared for Engineering Management Consultant," dated February 18, 1997 (Project No. 70131-6-0552.1000).

Law/Crandall, 1993, "Report of Potential Fault Displacements, Wastewater Treatment Plant Number 2, Huntington Beach, California, for County Sanitation Districts of Orange County," Project No. 2661.30140.0001.

LeRoy Crandall and Associates, 1987, "Report of Foundation Investigation, Proposed Classroom Building, Breed Street Elementary School, 2226 East Third Street, Los Angeles, California for the Los Angeles Unified School District," Project No. A-87104.

LeRoy Crandall and Associates, 1982, "Report of Foundation Investigation, Proposed Zody's–East Los Angeles, First and Mott Streets, Boyle Heights District, Los Angeles, California for the Herbert M. Piken Company," Project No. A-79206.

LeRoy Crandall and Associates, 1978, "Supplemental Geologic Information on Raymond Fault, San Marino High School, for the San Marino Unified School District," Project No. E-77186.

LeRoy Crandall and Associates, 1972, "Report of Foundation Investigation, Proposed East Los Angeles Neighborhood Center, Capital Project No. 9080.02, North Sunol Drive and East First Street, Los Angeles County, California for the County of Los Angeles," Project No. A-71073.

LeRoy Crandall and Associates, 1969, "Report of Preliminary Soil Investigation, Proposed Land Purchase, Indiana and First Streets, Los Angeles, California for the Glendale Federal Savings and Loan Association," Project No. A-69233.

LeRoy Crandall and Associates, 1961, "Report of Soils Investigation, Center-North 220 KV Transmission Line, Los Angeles County, California, for the Southern California Edison Company," Project No. 61105.

LeRoy Crandall and Associates, 1961, "Report of Foundation Investigation, Proposed Additions, Belvedere Elementary School, First Street and Eastman Avenue, Los Angeles County, California for the Los Angeles City Board of Education," Project No. 61494.

Los Angeles, City of, 1996, "Safety Element."

Los Angeles, County of, 1990, "Safety Element."

Los Angeles, County of, 1990, "Technical Appendix to the Safety Element of the Los Angeles County General Plan," Draft Report by Leighton and Associates with Sedway Cooke Associates.

Los Angeles County Department of Public Works, 1996, "Hydrologic Report 1994–1996".

Mark, R. K., 1977, "Application of Linear Statistical Models of Earthquake Magnitude Versus Fault Length in Estimating Maximum Expectable Earthquakes," *Geology*, Vol. 5, pp. 464–466.

Mooney, H., 1984, "Handbook of Engineering Geophysics, Vol.1," Bison Instruments, Minneapolis, MN.

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

Oskin et al., 2000, "Active Parasitic Folds on the Elysian Park Anticline: Implications for Seismic Hazard in Central Los Angeles, California," Geologic Society of America Bulletin, May 2000, v. 112, no. 5, pp. 693-707.

Petersen, M. D. and Wesnousky, S. G., 1994, "Fault Slip Rate and Earthquake Histories for Active Faults in Southern California," *Bulletin of the Seismological Society of America*, Vol. 84, pp. 1608-1649.

Petersen, M. D., Bryant, W. A., and Cramer, C. H., 1996, "Table of Fault Parameters Used by the California Division of Mines and Geology to Compile the Probabilistic Seismic Hazard Maps of California," California Division of Mines and Geology Open File Report 96-08.

Poland, J. R., Garrett, A. A., and Sinnott, A., 1959, "Geology, Hydrology, and Chemical Character of Groundwaters in the Torrance–Santa Monica Area, California," U.S. Geological Survey Water Supply Paper 1461.

Sadigh, K. et al., 1997, "Attenuation Relations for Shallow Crustal Earthquakes Based on California Strong Motion Data," *Seismological Research Letters*, Vol. 68, No. 1. pp.180-189.

Schneider, C. L., Hummon, C., Yeats, R. S., and Huftile, G. L., 1996, "Structural Evolution of the Northern Los Angeles Basin, California, Based on Growth Strata," *Tectonics,* Vol. 15, No. 2, pp. 341-355.

Seed, H. B., Tokimatsu, K., Harden, L. F., and Chung R.M., 1984, "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation," Report No. UCB/EERG-84/l 5, Earthquake Engineering Research Center, University of California, Berkeley, California.

Seed, R. B. and Idriss, I. M., 1982, "Ground Motion and Soil Liquefaction During Earthquake," Monograph No.5, Monograph Series, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Shaw, J. H., 1993, "Active Blind-Thrust Faulting and Strike-Slip Folding in California," Ph.D. Thesis, Princeton University, Princeton, New Jersey, 216 pp.

Shaw, J. H. and Suppe, J., 1996, "Earthquake Hazards of Active Blind Thrust Faults Under the Central Los Angeles Basin, California," *Journal of Geophysical Research,* Vol. 101, No. B4, pp. 8623-8642.

Sieh, K. E., 1984, "Lateral Offsets and Revised Dates of Large Pre-Historic Earthquakes at Pallett Creek, California," *Journal of Geophysical Research,* Vol. 9, pp. 7461–7670.

Slemmons, D. B., 1979, "Evaluation of Geomorphic Features of Active Faults for Engineering Design and Siting Studies," Association of Engineering Geologists Short Course.

Sommerville, P. G., Smith, N. F., Graves, R. W., and Abrahamson, N. A., 1997, "Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity," *Seismological Research Letters,* Vol. 68, No. 1.

Somerville, Paul, G., Nancy F. Smith, Robert W. Graves and Norman A. Abrahamson, "Accounting for Near-Fault Rupture Directivity Effects in the Development of Design Ground Motions." Conference Proc. Seismic, Shock and Vibration Isolation, 1995 Joint ASME/JSME Pressure Vessels and Piping Conference, Honolulu, Hawaii. PVP-Vol 1319 ASME, 1995.

Southern California Seismographic Network, 2001, "Southern California Earthquake Catalog," http://www.scecdc.scec.org/ftp/catalogs/SCSN/.

Toppozada, T. R., Bennett, J. H., Borchardt, G. A., Saul, R., and Davis, J.F., 1988, "Planning Scenario for a Major Earthquake on the Newport–Inglewood Fault Zone," California Division of Mines and Geology Special Publication 99.

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

U.S. Army Corps of Engineers, 1979, "Geophysical Exploration, Engineering Manual No. 1110-1-1802," Department of the Army, Washington, D.C.

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

Vucetic, M., and Dobry, R., 1991, "Effect of Soil Plasticity on Cyclic Response", *ASCE Journal of Geotechnical Engineering*, Vol. 117, No. 1.

Wallace, R. E., 1968, "Notes of Stream Channel Offset by San Andreas Fault, Southern Coast Ranges, California," in Dickinson, U. R., and Grantz, A., eds., Proceedings of Conference of Geologic Problems on San Andreas Fault System, Stanford University Publications, Geological Sciences, Vol. IX, p. 6–21.

Weber, F. H., Bennett, J. H., Chapman, R. H., Chase, G. W., and Saul, R. B., 1980, "Earthquake Hazards Associated with the Verdugo–Eagle Rock and Benedict Canyon Fault Zones, Los Angeles County, California," California Division of Mines and Geology Open File Report 80-10LA.

Wesnousky, S. G., 1986, "Earthquakes, Quaternary Faults, and Seismic Hazard in California," *Journal of Geophysical Research*, Vol. 91, No. B12, pp. 12,587–12,631.

Wissler, S. G., 1943, "Stratigraphic Formations of the Producing Zone of the Los Angeles Basin Oil Fields," California Division of Mines Bulletin 118.

Woodward-Clyde Consultants, 1997, "East Side Extension Stations, Seismic Hazard Evaluation, Los Angeles Metro Rail System, Los Angeles, California, Prepared for Engineering Management Consultant," dated April 10, 1997 (Project No. 964G152).

Working Group on California Earthquake Probabilities, 1995, "Seismic Hazards in Southern California: Probable Earthquakes, 1994 to 2024," *Bulletin of the Seismological Society of America*, Vol. 85, No. 2.

Wright, T. L., 1991, "Structural Geology and Tectonic Evolution of the Los Angeles Basin, California," American Association Petroleum Geologists, Memoir 52, p. 35-134.

Yerkes, R. F., Tinsley, J. C., and Williams, K. M., 1977, "Geologic Aspects of Tunneling in the Los Angeles Area, California," Miscellaneous Field Studies Map MF-866.

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

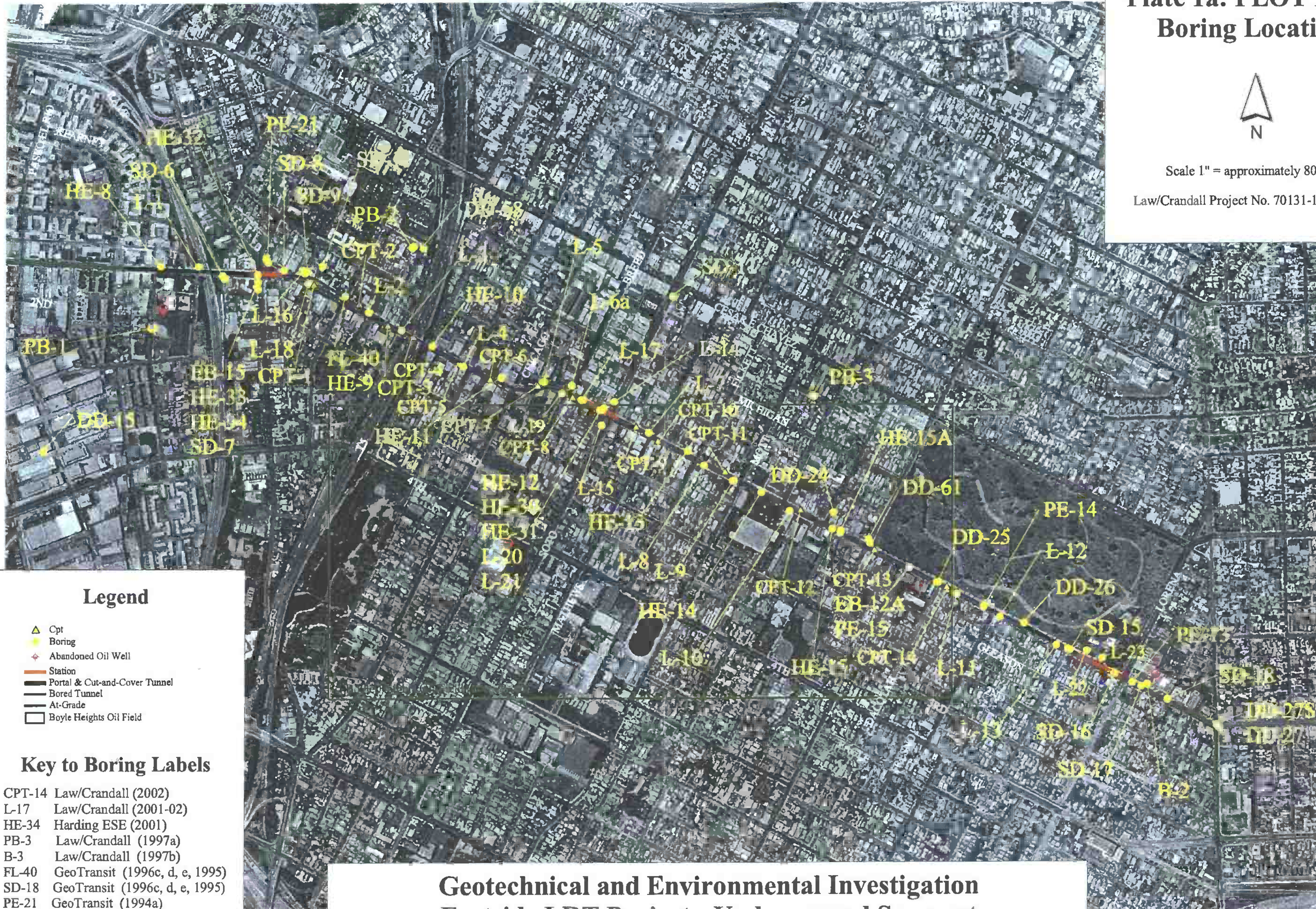
Ziony, J. I., and Jones, L. M., 1989, "Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California," U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.

Plate 1a: PLOT PLAN
Boring Locations

N

Scale 1" = approximately 800 feet

Law/Crandall Project No. 70131-1-0138.0005



Legend

- Cpt
- Boring
- Abandoned Oil Well
- Station
- Portal & Cut-and-Cover Tunnel
- Bored Tunnel
- At-Grade
- Boyle Heights Oil Field

Key to Boring Labels

CPT-14	Law/Crandall (2002)
L-17	Law/Crandall (2001-02)
HE-34	Harding ESE (2001)
PB-3	Law/Crandall (1997a)
B-3	Law/Crandall (1997b)
FL-40	GeoTransit (1996c, d, e, 1995)
SD-18	GeoTransit (1996c, d, e, 1995)
PE-21	GeoTransit (1994a)
EB-15	GeoTransit (1994b)

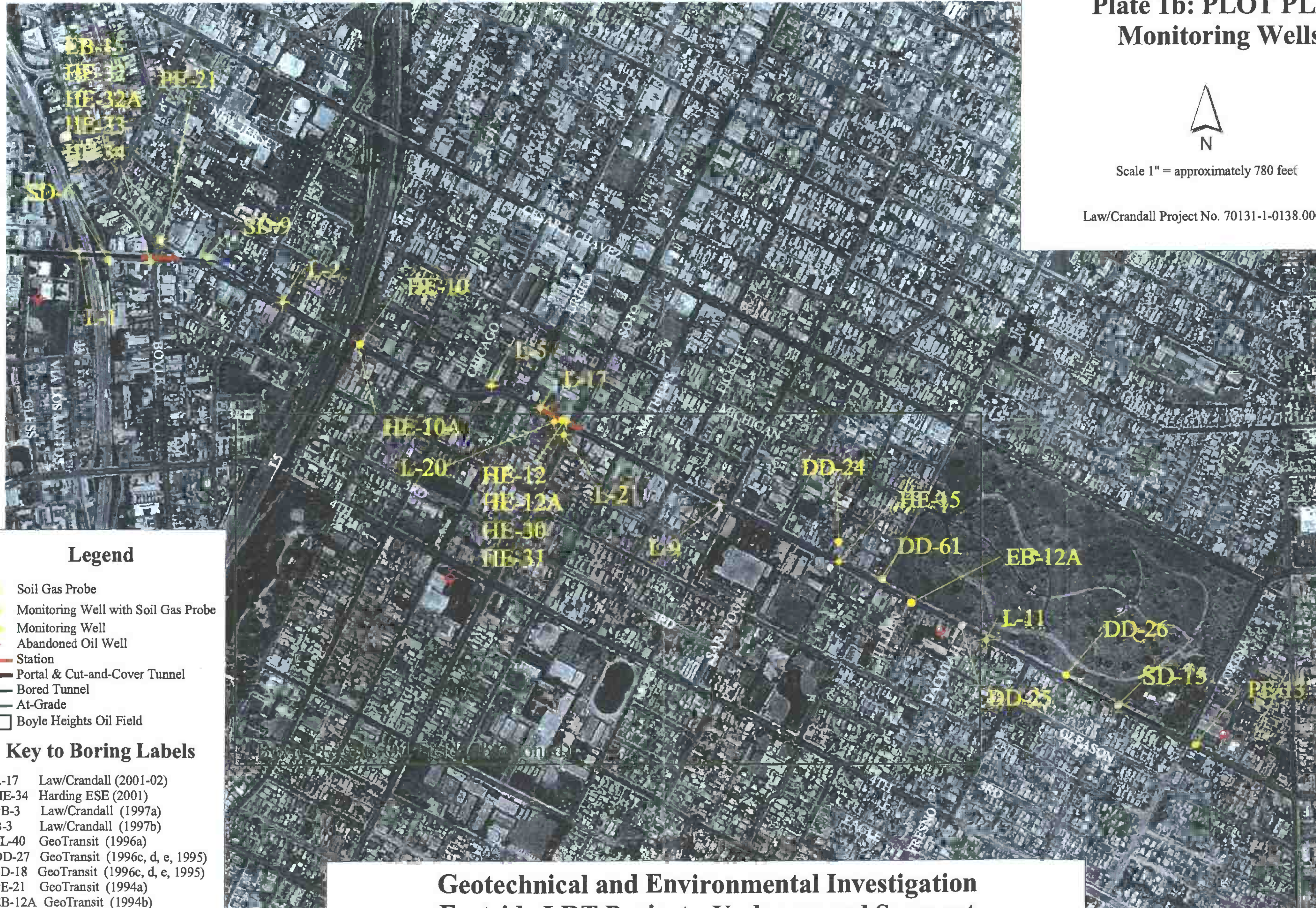
Geotechnical and Environmental Investigation
Eastside LRT Project - Underground Segment

Plate 1b: PLOT PLAN
Monitoring Wells

N

Scale 1" = approximately 780 feet

Law/Crandall Project No. 70131-1-0138.0005



Legend

- Soil Gas Probe
- Monitoring Well with Soil Gas Probe
- Monitoring Well
- Abandoned Oil Well
- Station
- Portal & Cut-and-Cover Tunnel
- Bored Tunnel
- At-Grade
- Boyle Heights Oil Field

Key to Boring Labels

L-17	Law/Crandall (2001-02)
HE-34	Harding ESE (2001)
PB-3	Law/Crandall (1997a)
B-3	Law/Crandall (1997b)
FL-40	GeoTransit (1996a)
DD-27	GeoTransit (1996c, d, e, 1995)
SD-18	GeoTransit (1996c, d, e, 1995)
PE-21	GeoTransit (1994a)
EB-12A	GeoTransit (1994b)

Geotechnical and Environmental Investigation
Eastside LRT Project - Underground Segment

Plate 1c: PLOT PLAN
Location of Seismic Velocity Surveys & Pressuremeter Tests

N

Scale 1" = approximately 850 feet

Law/Crandall Project No. 70131-1-0138.0005

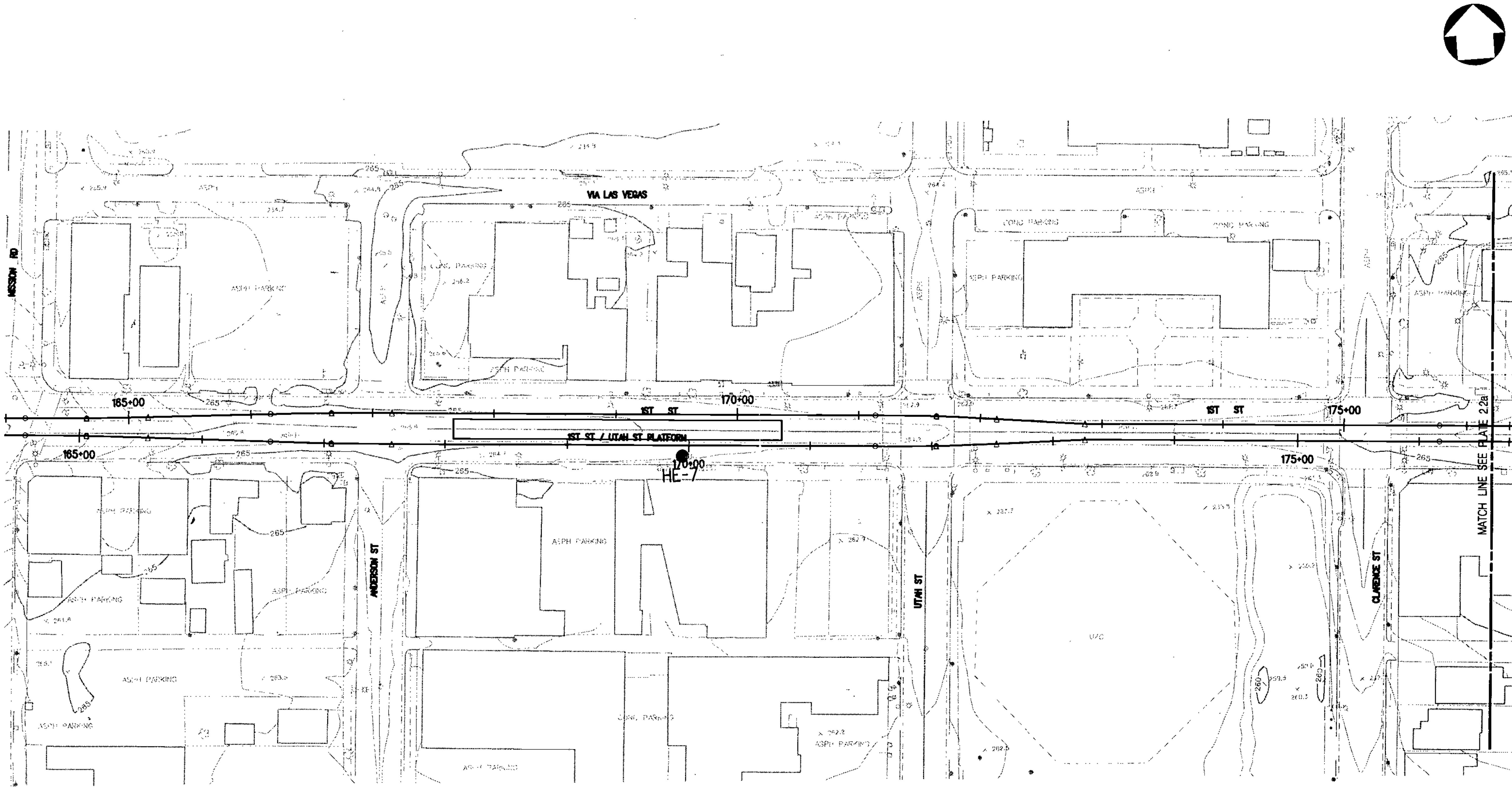
Legend

- SVS & Pressuremeter Tests
- Seismic Velocity Survey (SVS)
- Abandoned Oil Well
- Station
- Portal and Cut-and-Cover Tunnel
- Bored Tunnel
- At-Grade
- Boyle Heights Oil Field

Key to Boring Labels

L-17	Law/Crandall (2001-02)
HE-34	Harding ESE (2001)
PB-3	Law/Crandall (1997a)
B-3	Law/Crandall (1997b)
FL-40	GeoTransit (1996a)
DD-27	GeoTransit (1996c, d, e, 1995)
SD-18	GeoTransit (1996c, d, e, 1995)
PE-21	GeoTransit (1994a)

Geotechnical and Environmental Investigation
Eastside LRT Project - Underground Segment



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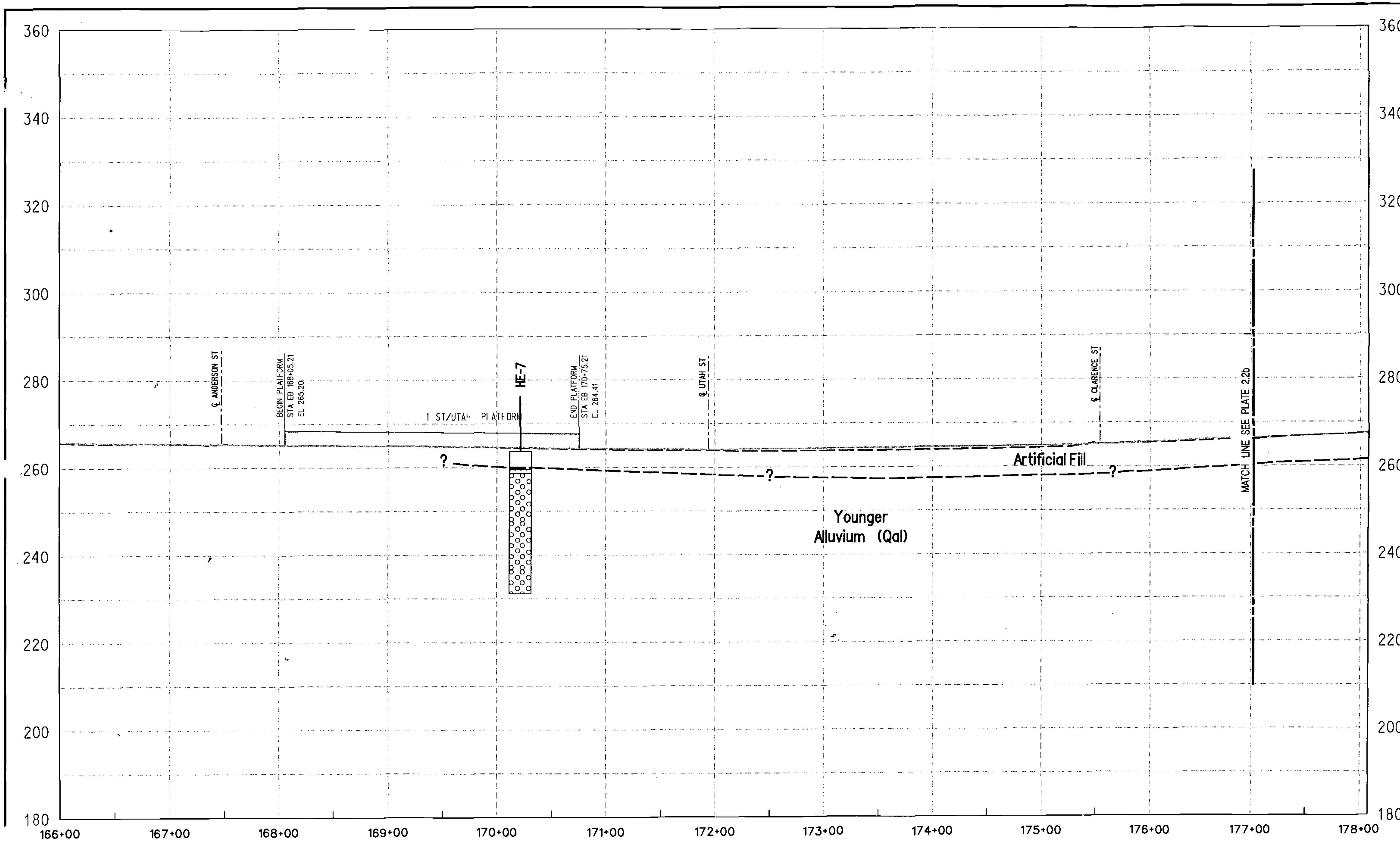
DRAWN BY: TEA	CHECKED BY: SB
SCALE: 1"-80'	PLATE 2.1a



NOTE:
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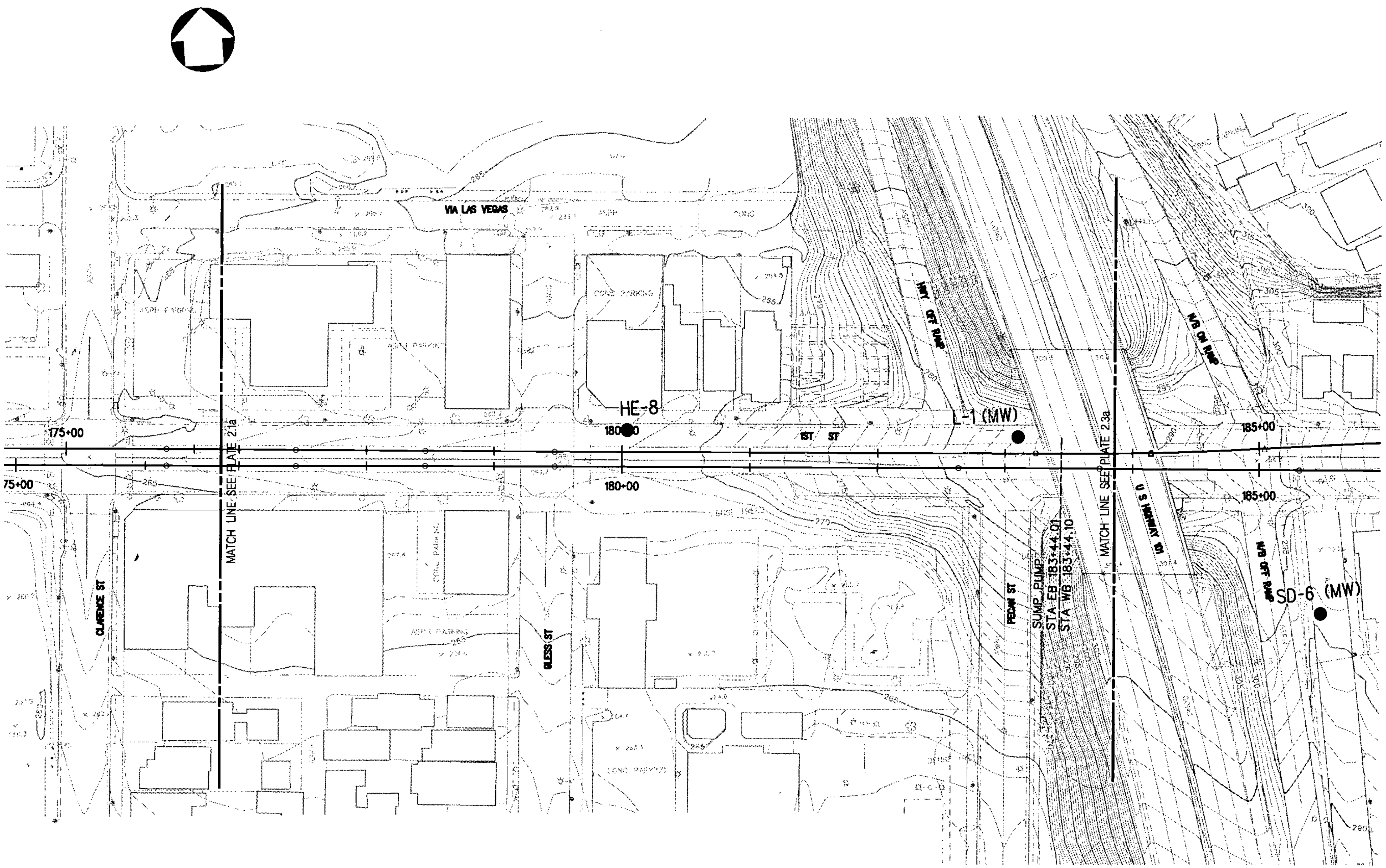
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.1b



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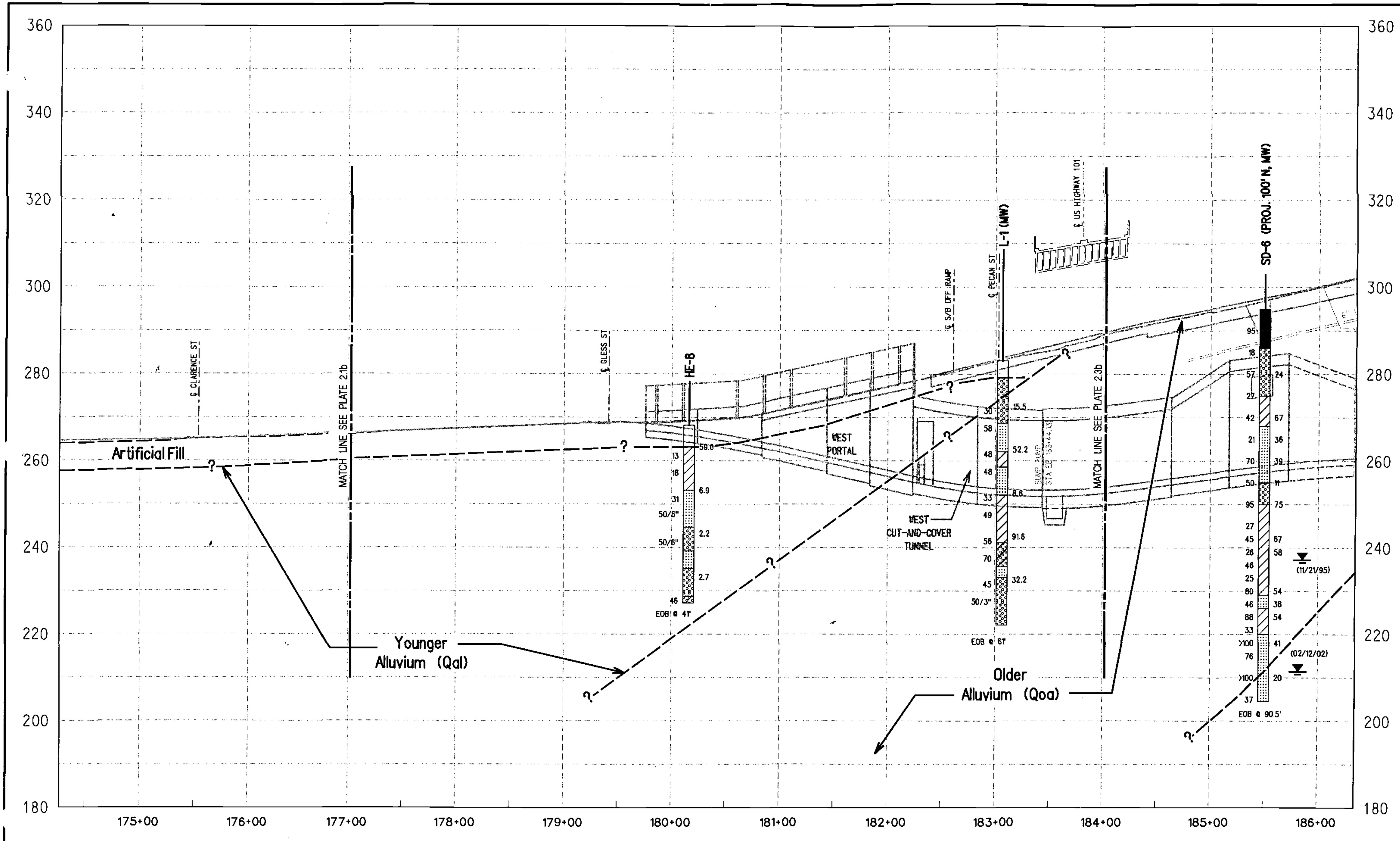
SCALE: 1"=80'	PLATE 2.2a



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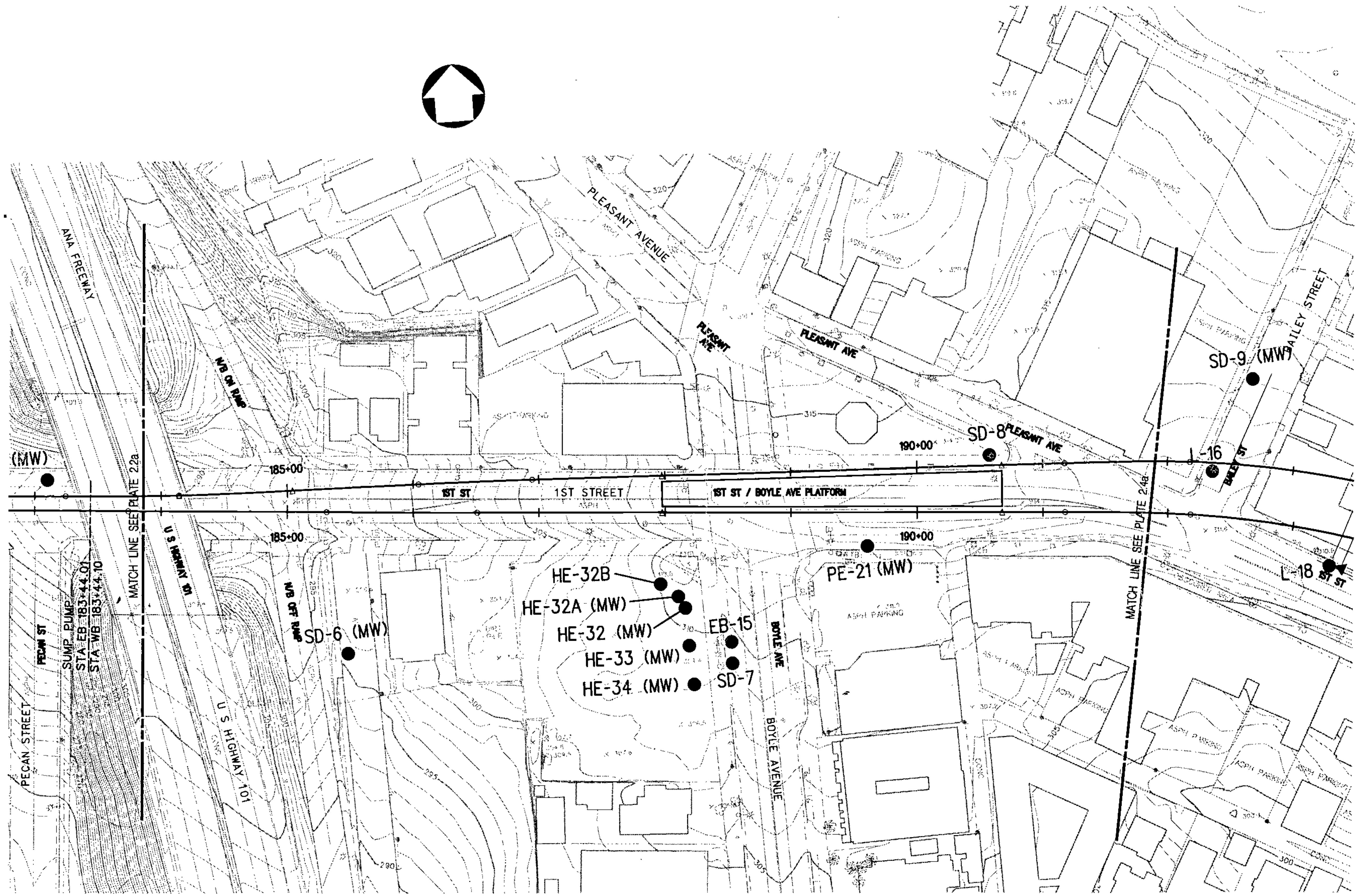
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.2b



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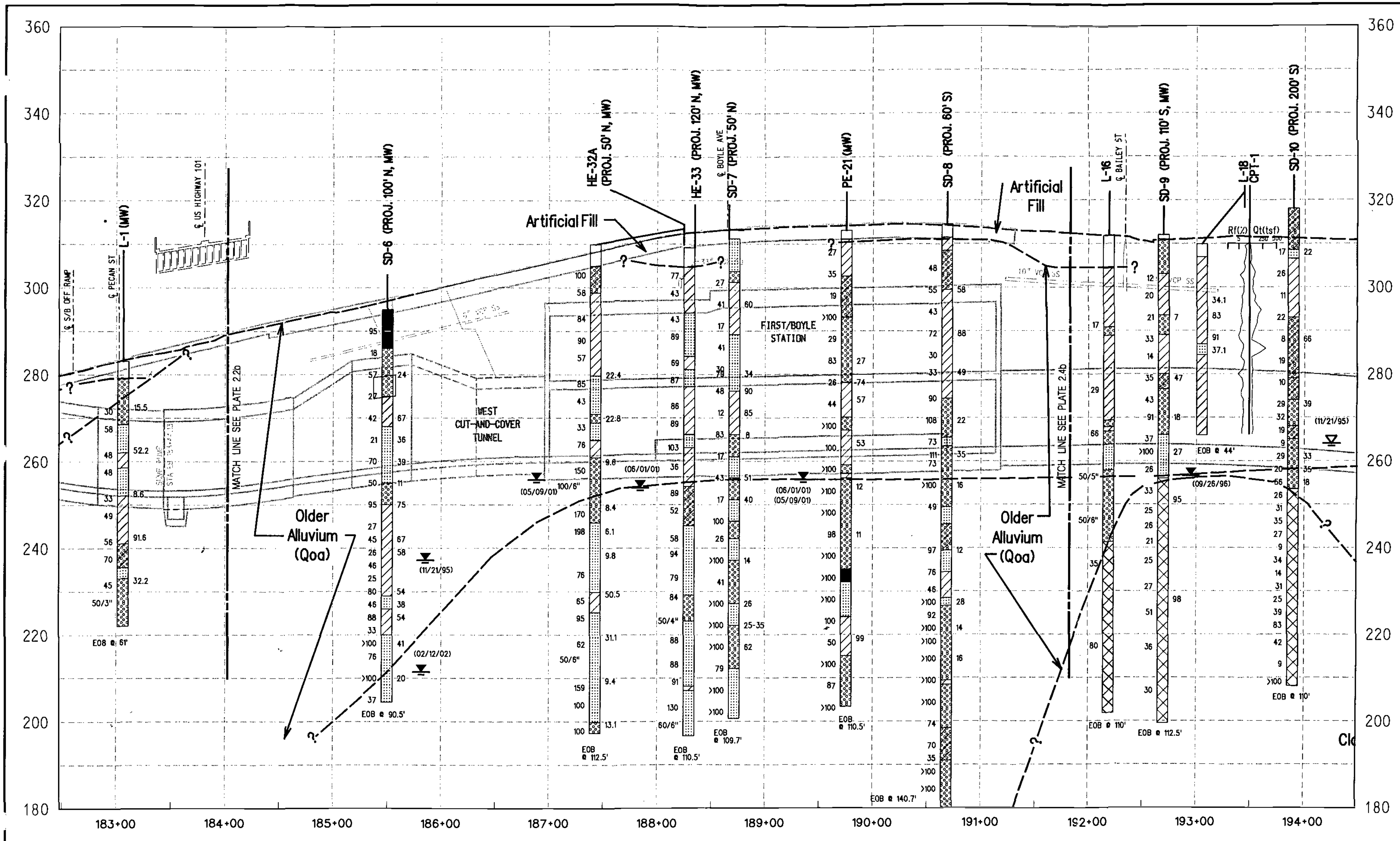
SCALE: 1"=80'	PLATE 2.3a



NOTES:
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SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.3b

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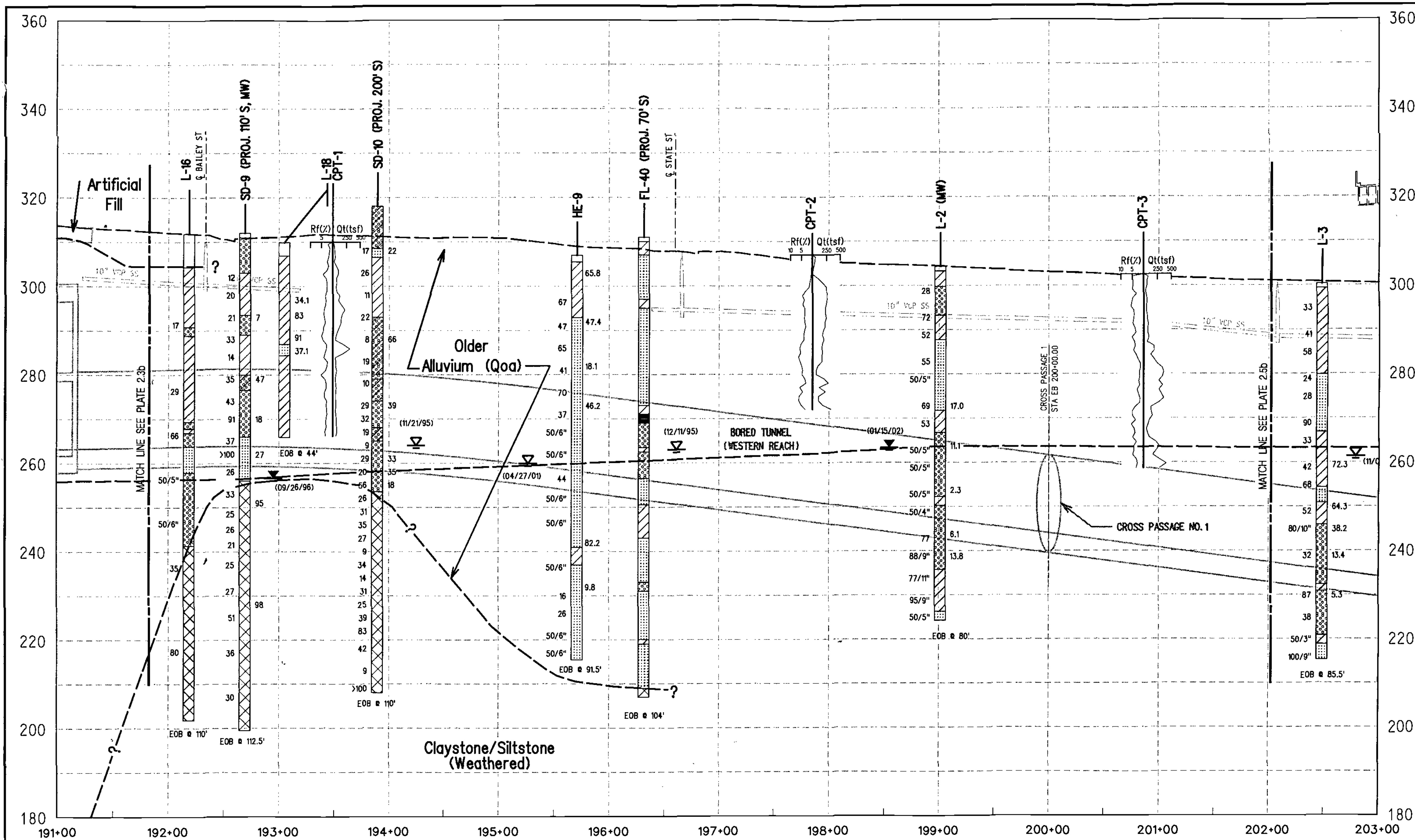
SCALE: 1"=80'	PLATE 2.4a



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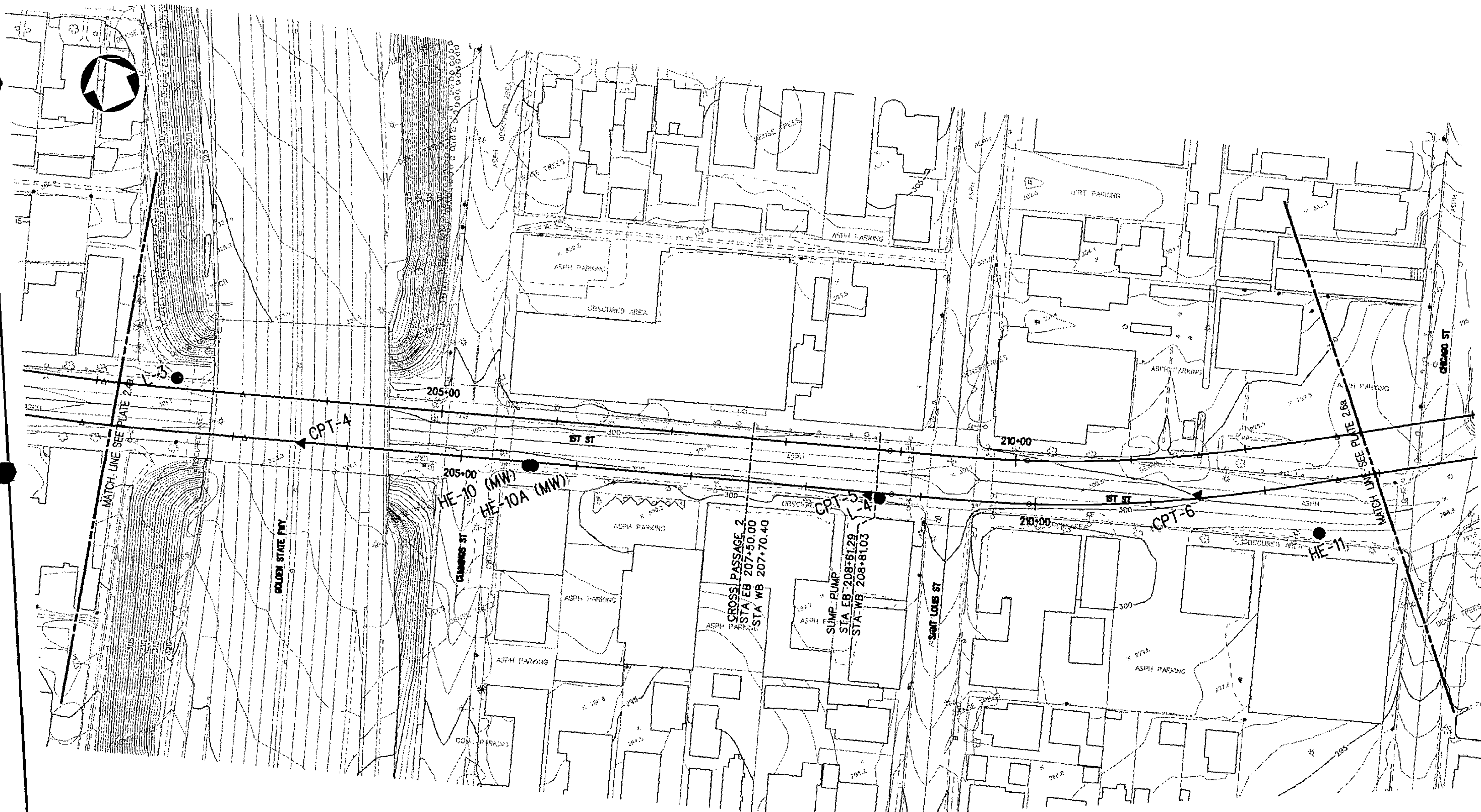
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.4b



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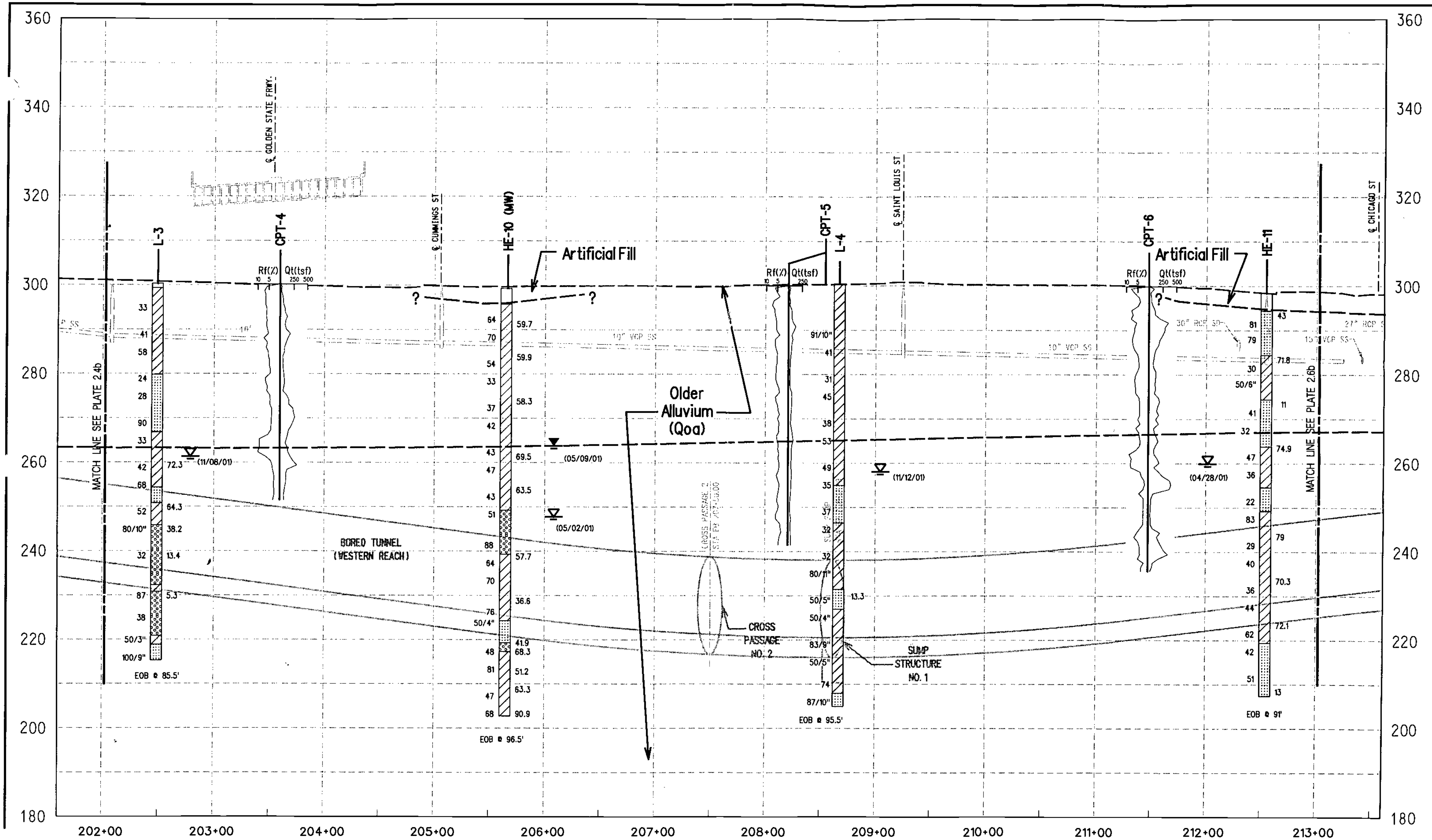
SCALE: 1"=80'	PLATE 2.5a



NOTES:
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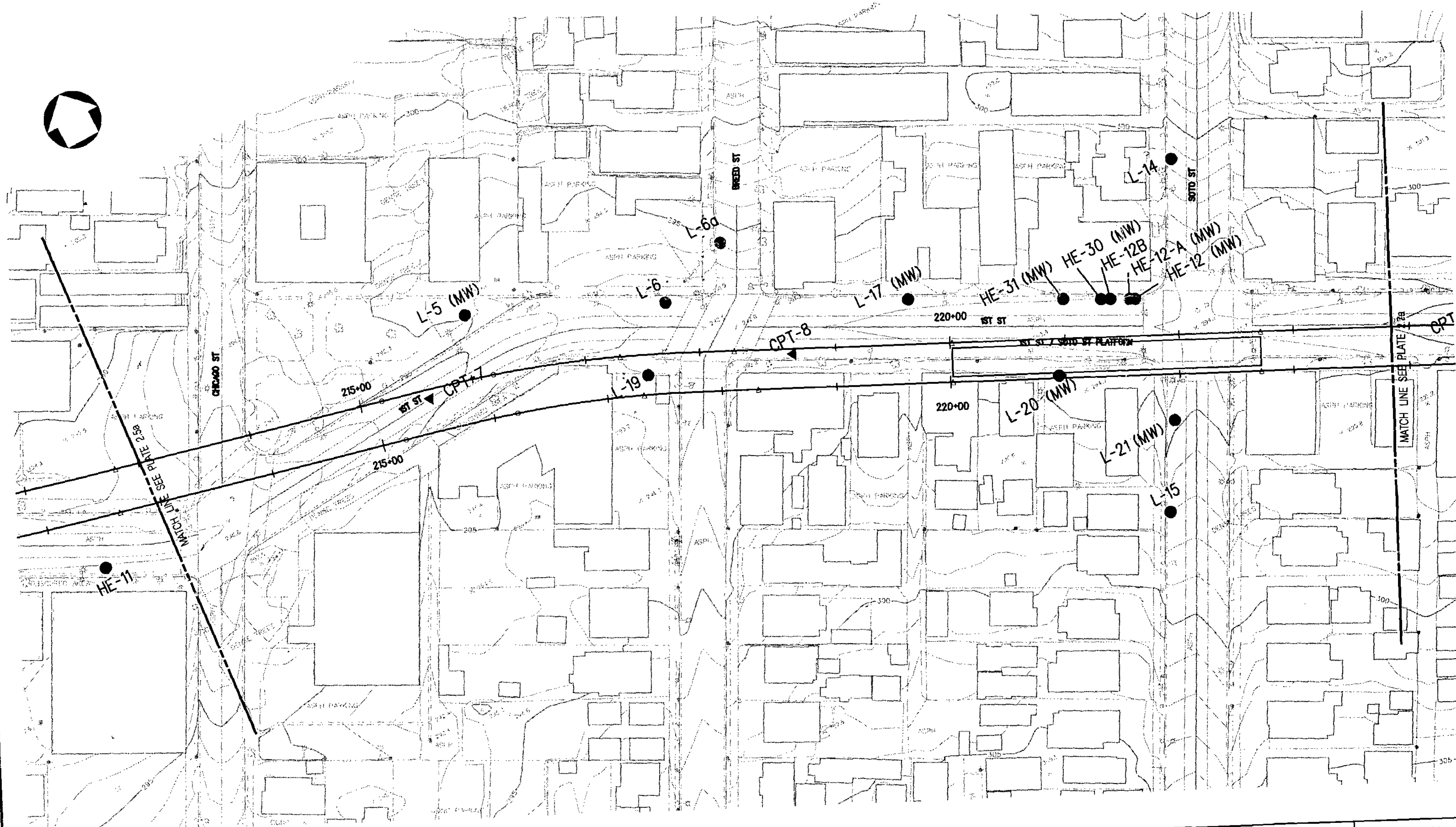

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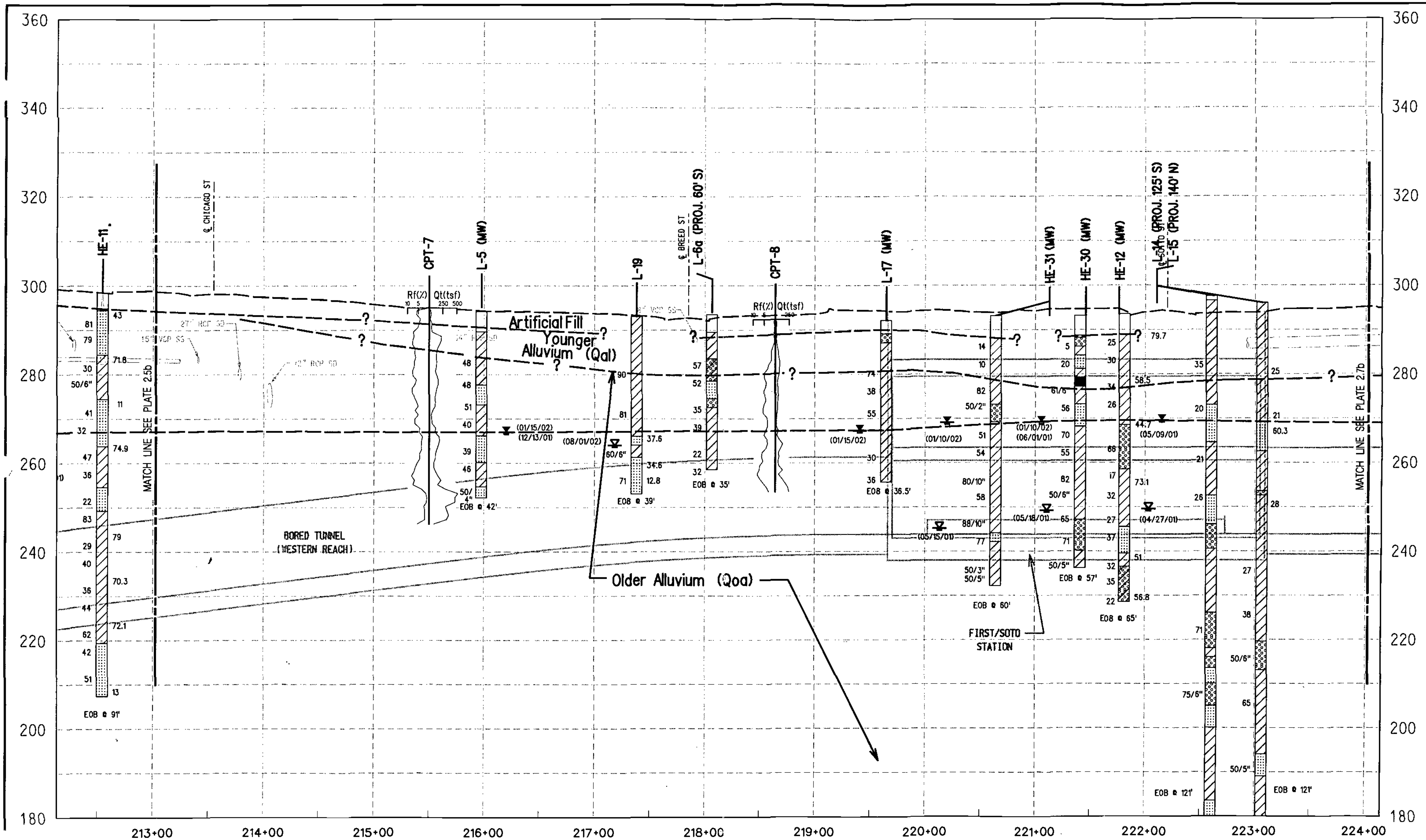
DRAWN BY: TEA	CHECKED BY: SB
SCALE: 1"-80'	PLATE 2.6a



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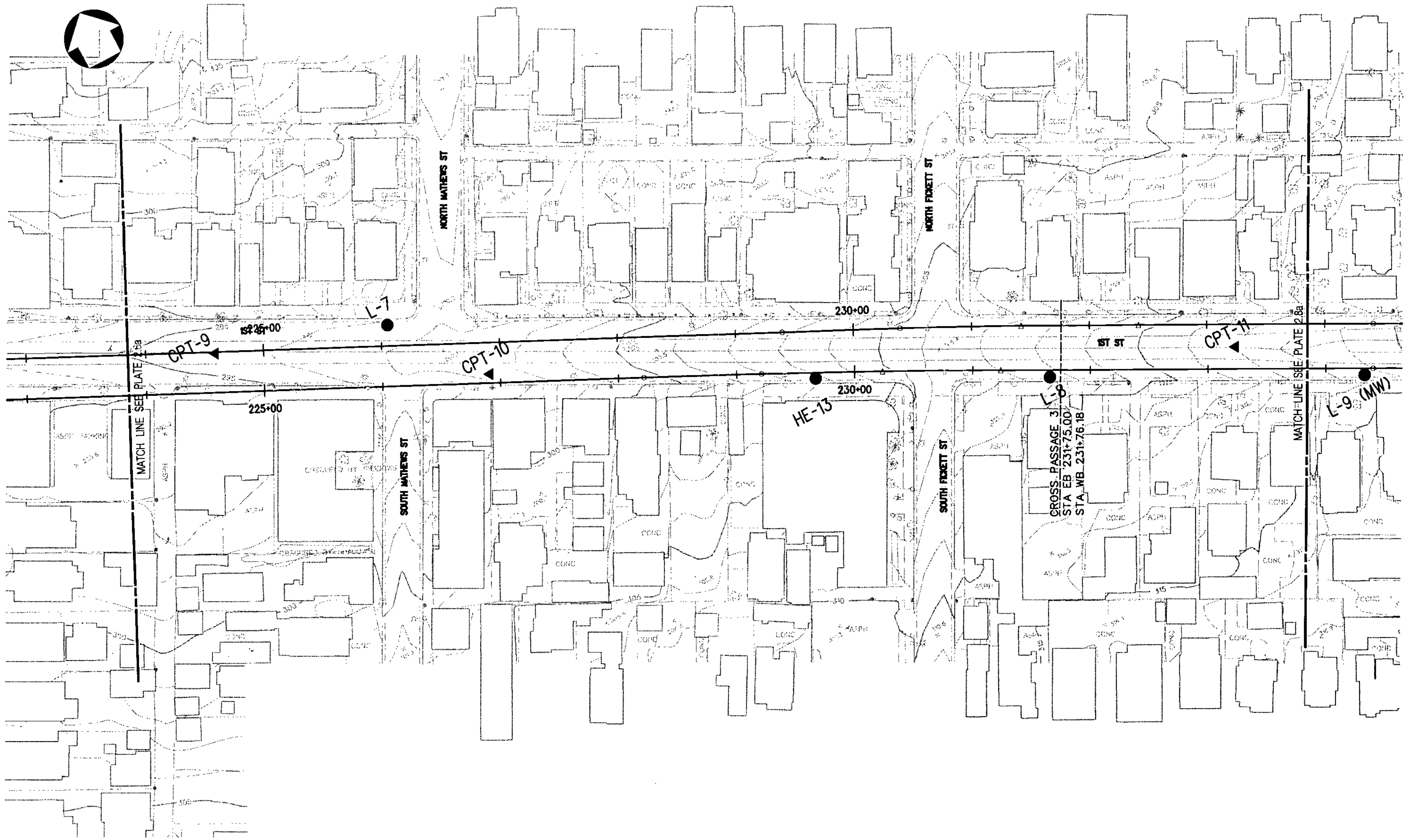
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.6b



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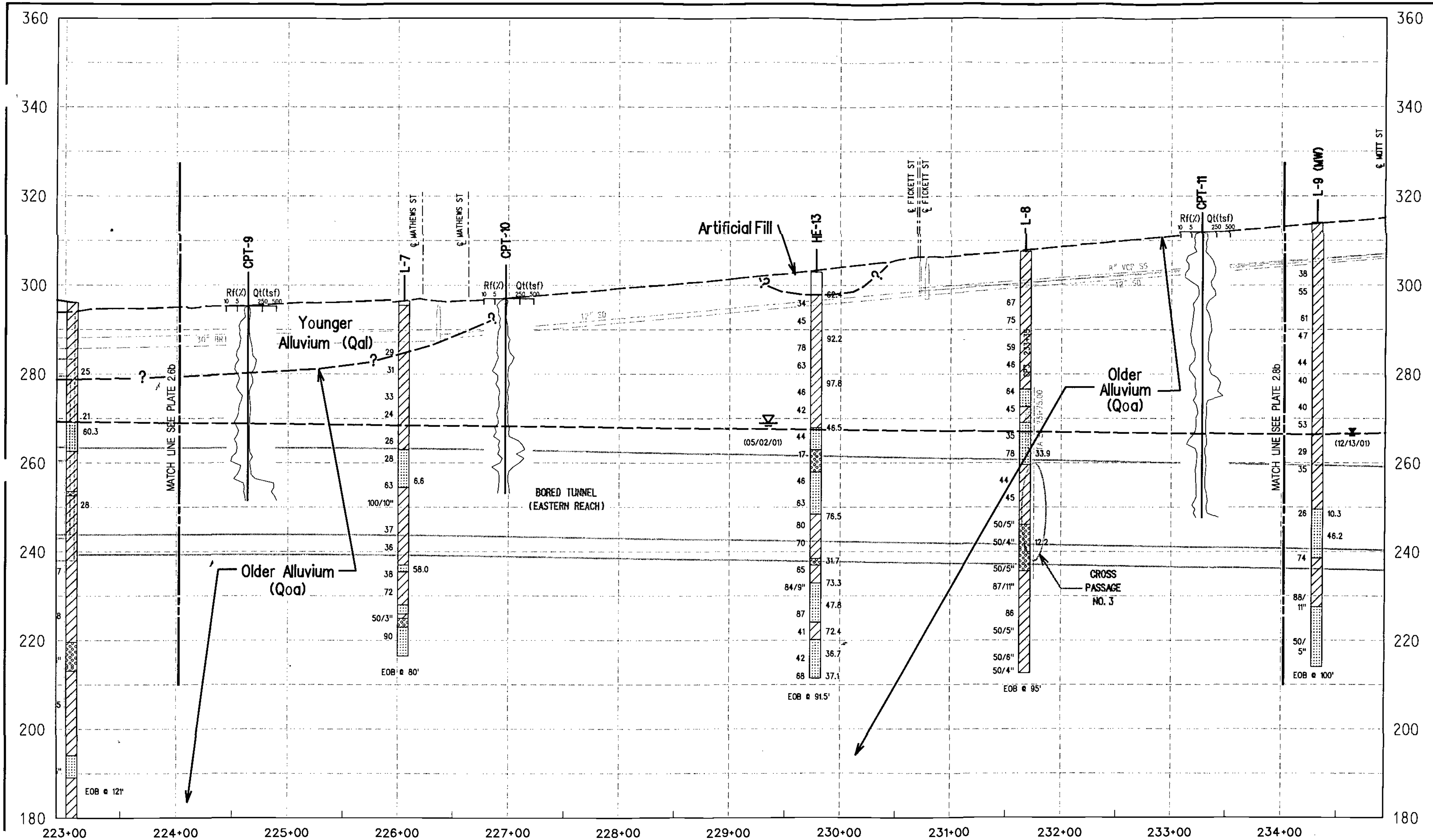
		SCALE: 1"=80'	PLATE 2.7a



NOTE:
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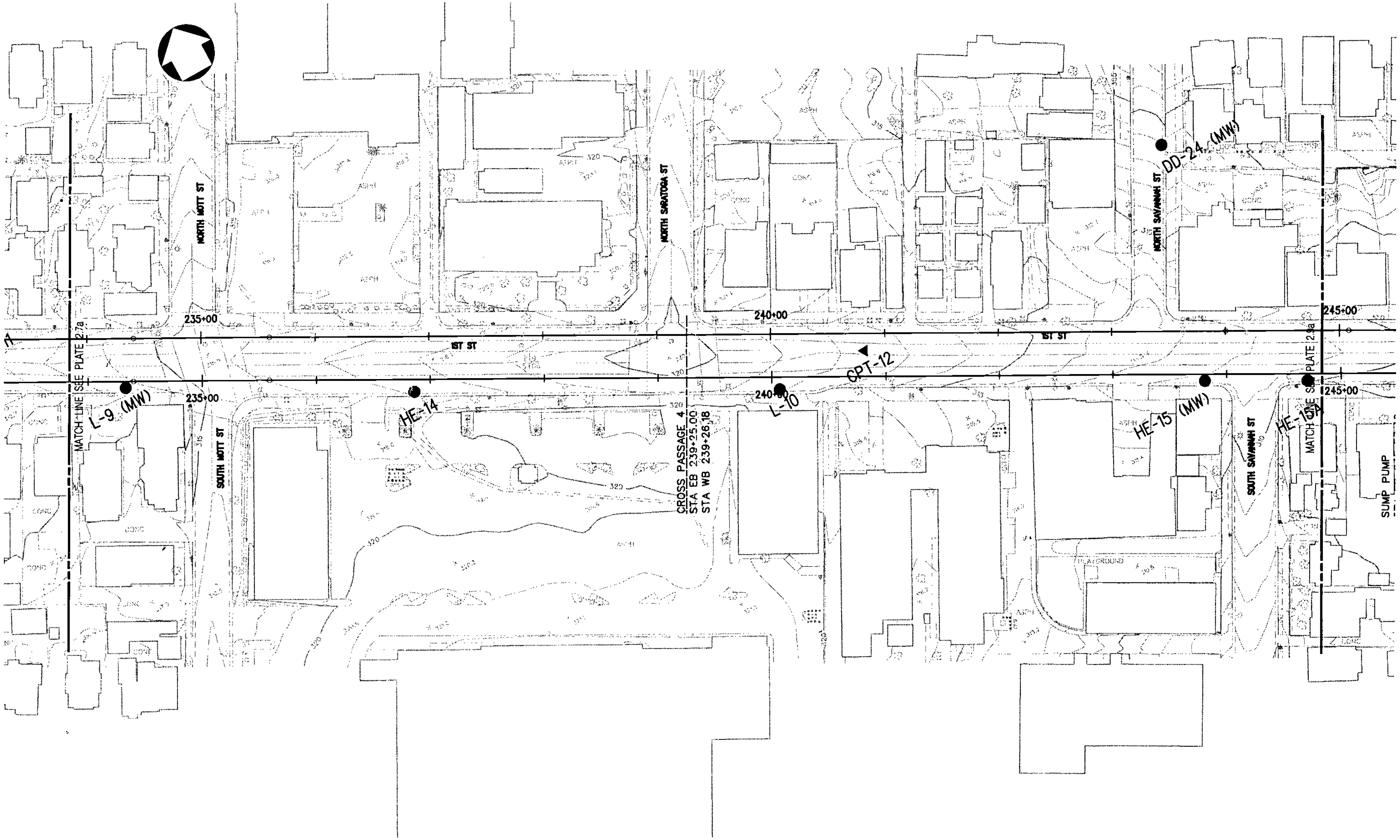
SCALE: HORIZ: 1"-80' VERT: 1"-20'	PLATE 2.7b



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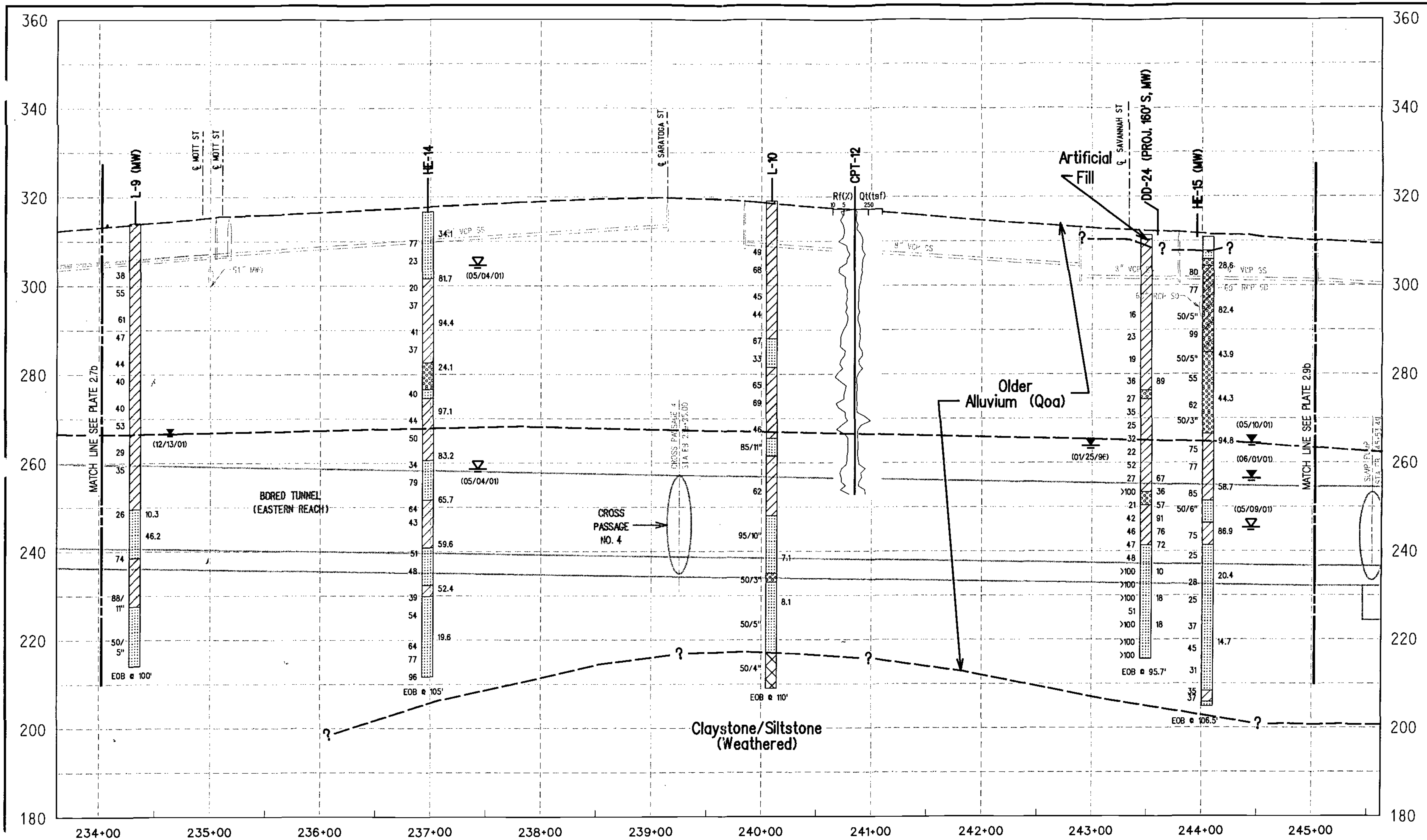
SCALE: 1"=80'	PLATE 2.8a



NOTES:
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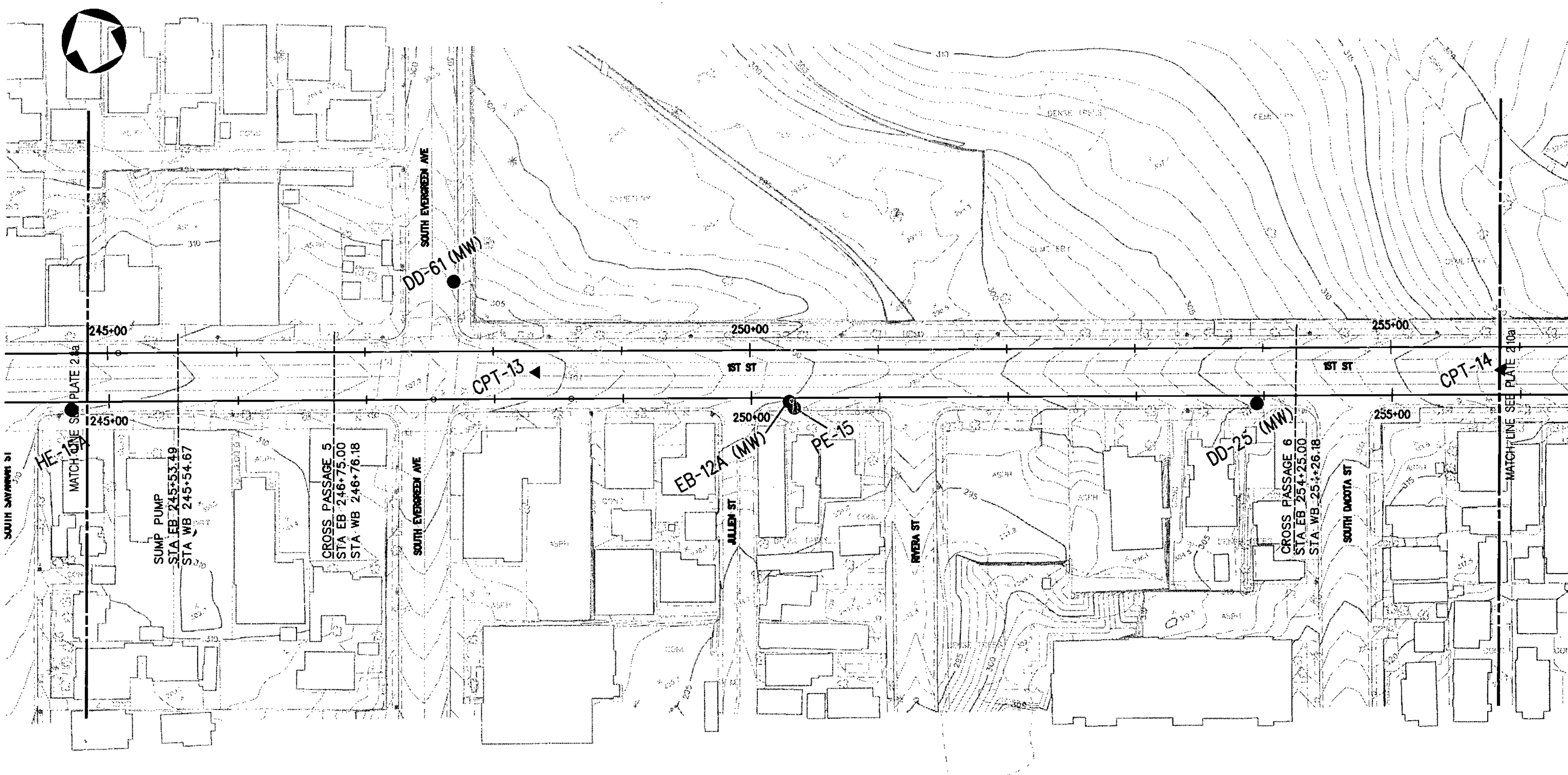

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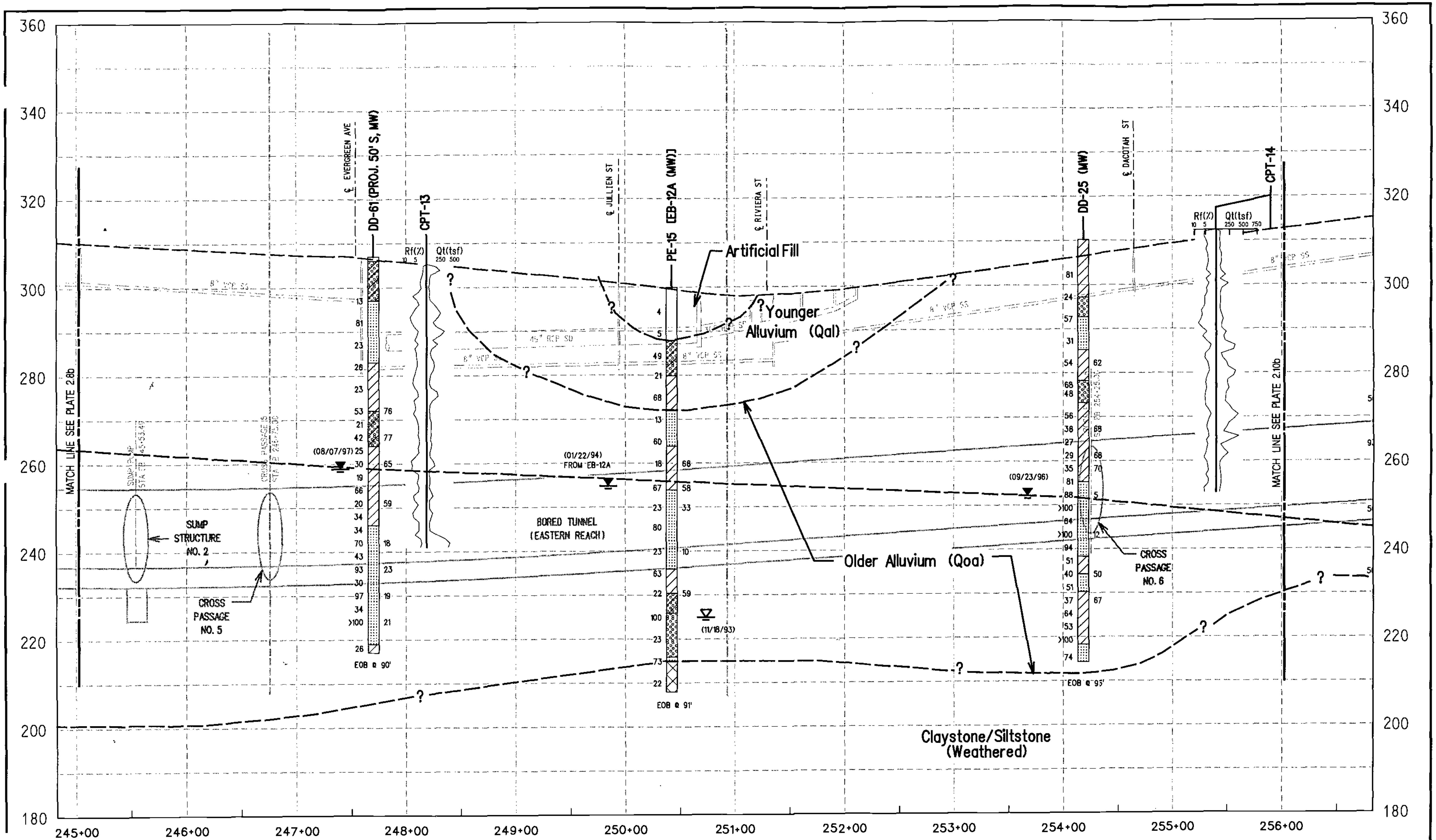
SCALE: 1"=80'	PLATE 2.9a



NOTES:
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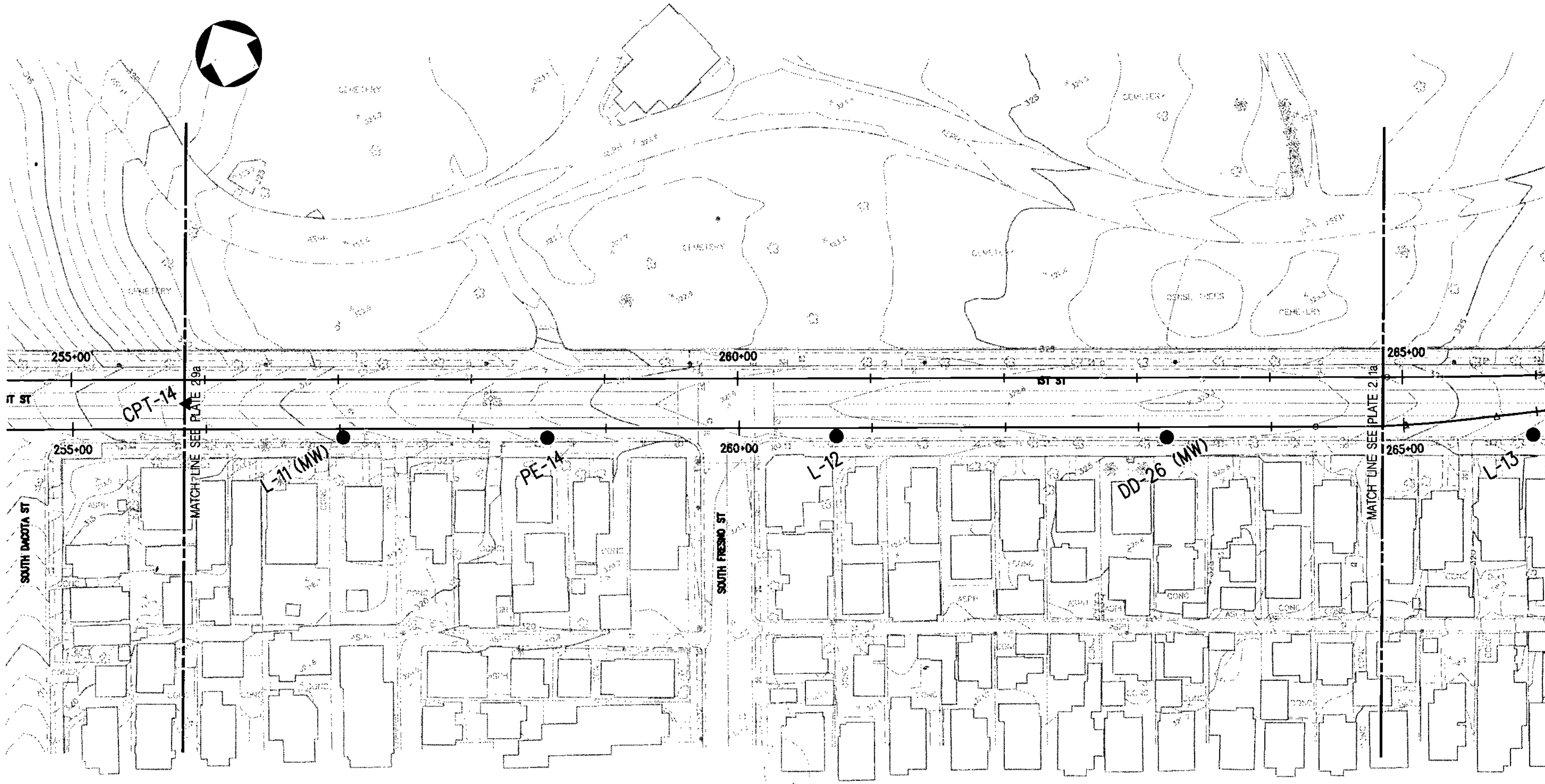
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.9b



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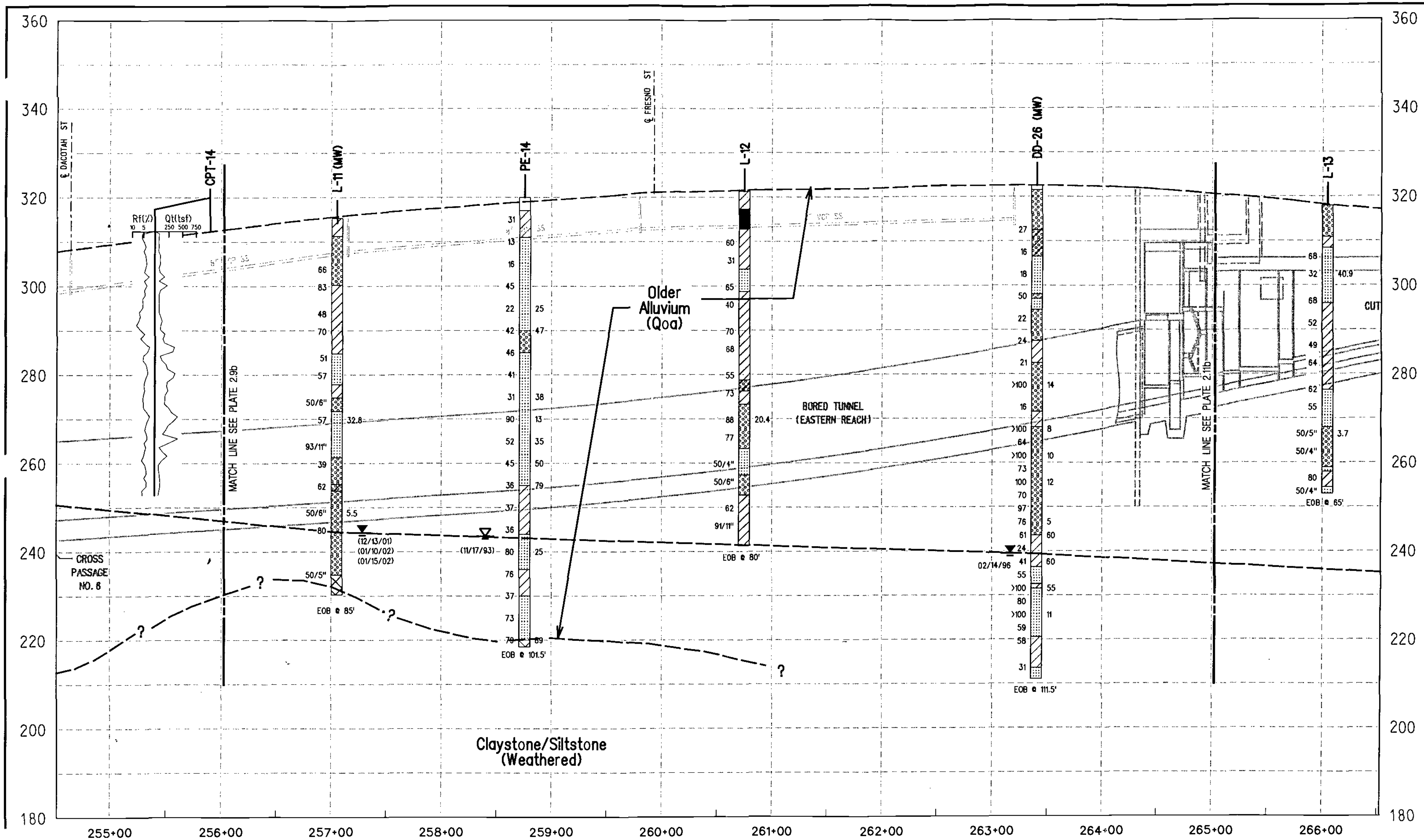
SCALE: 1"-80'	PLATE 2.10a



NOTE:
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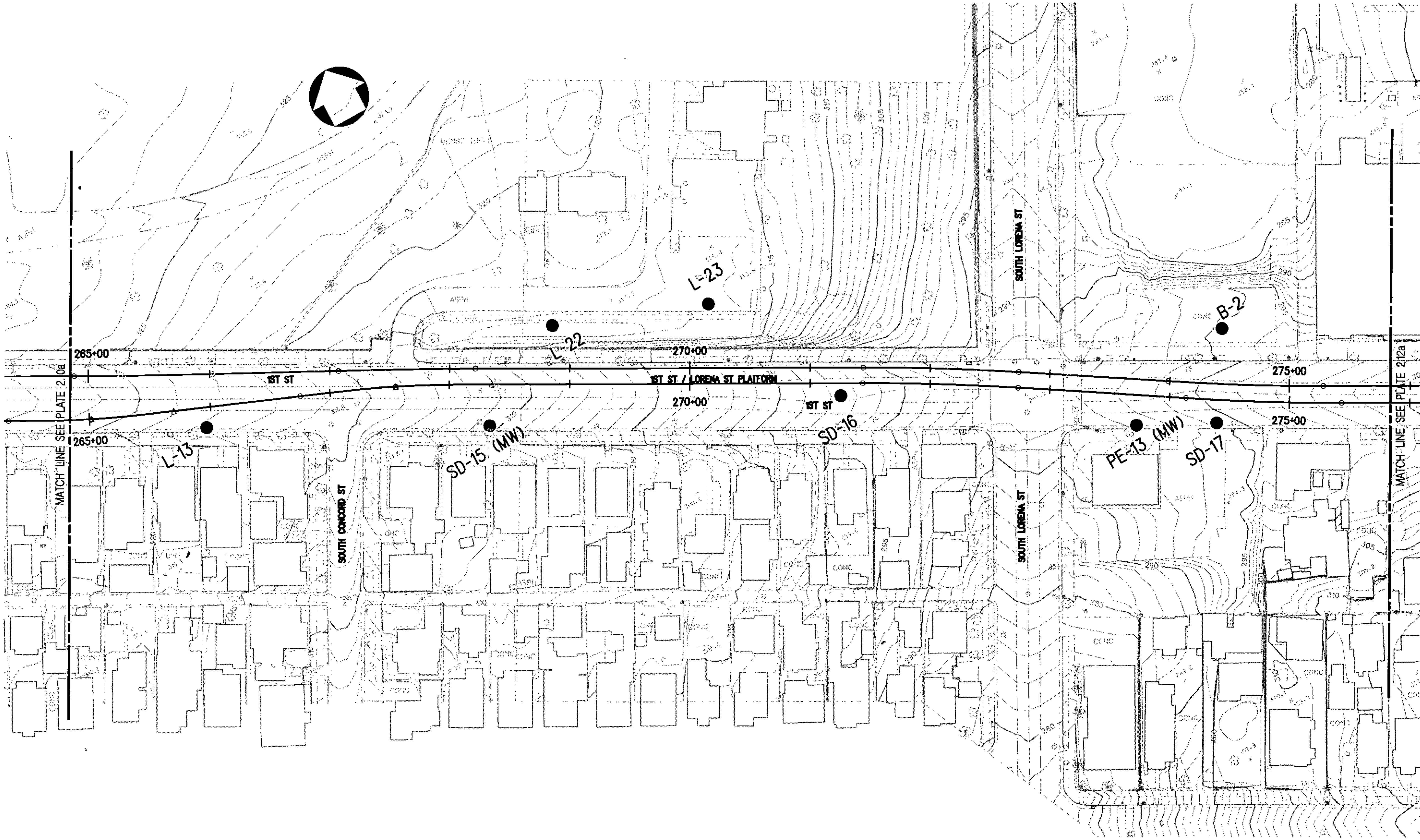
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.10b



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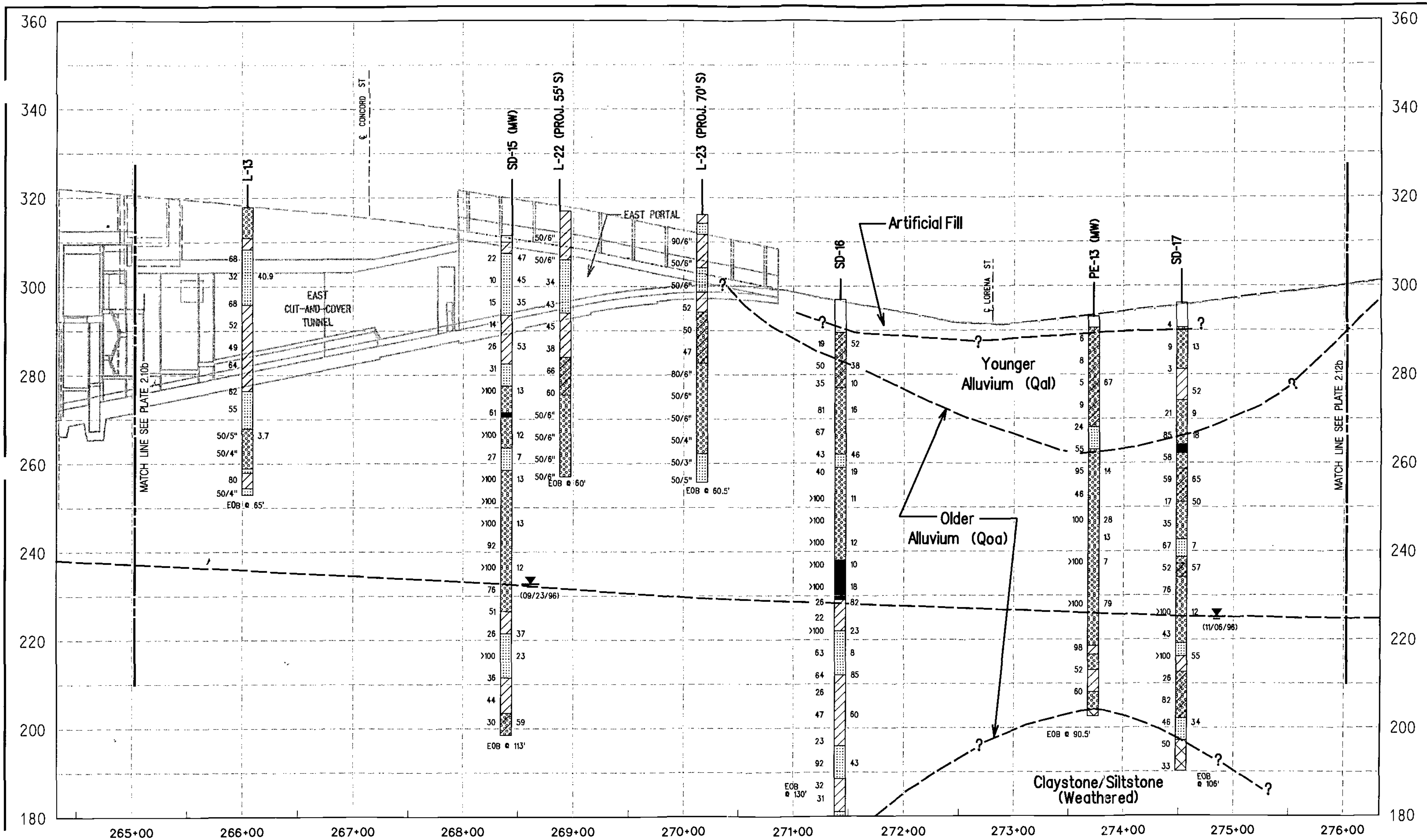
SCALE: 1"=80'	PLATE 2.11a



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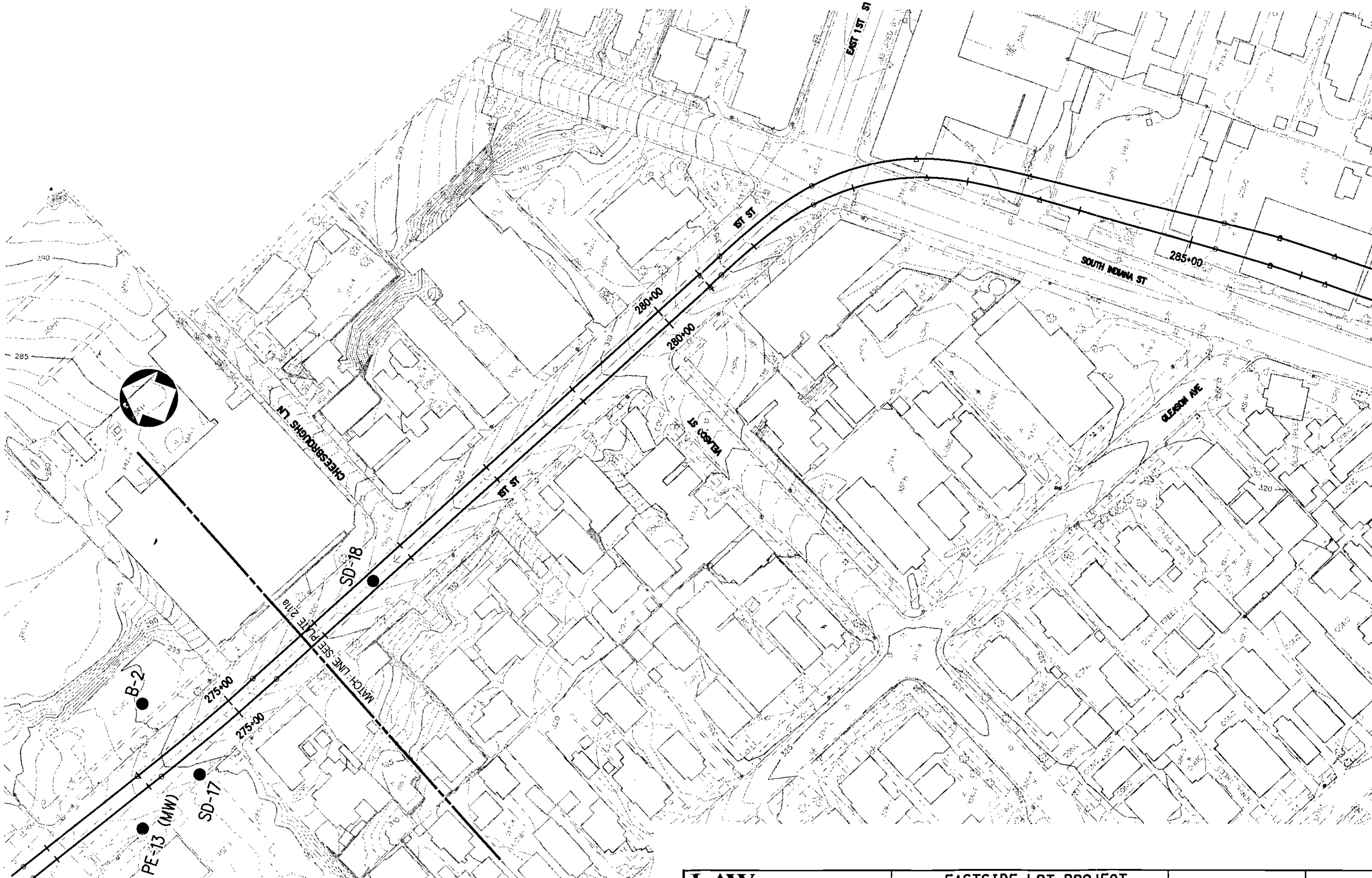
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.11b



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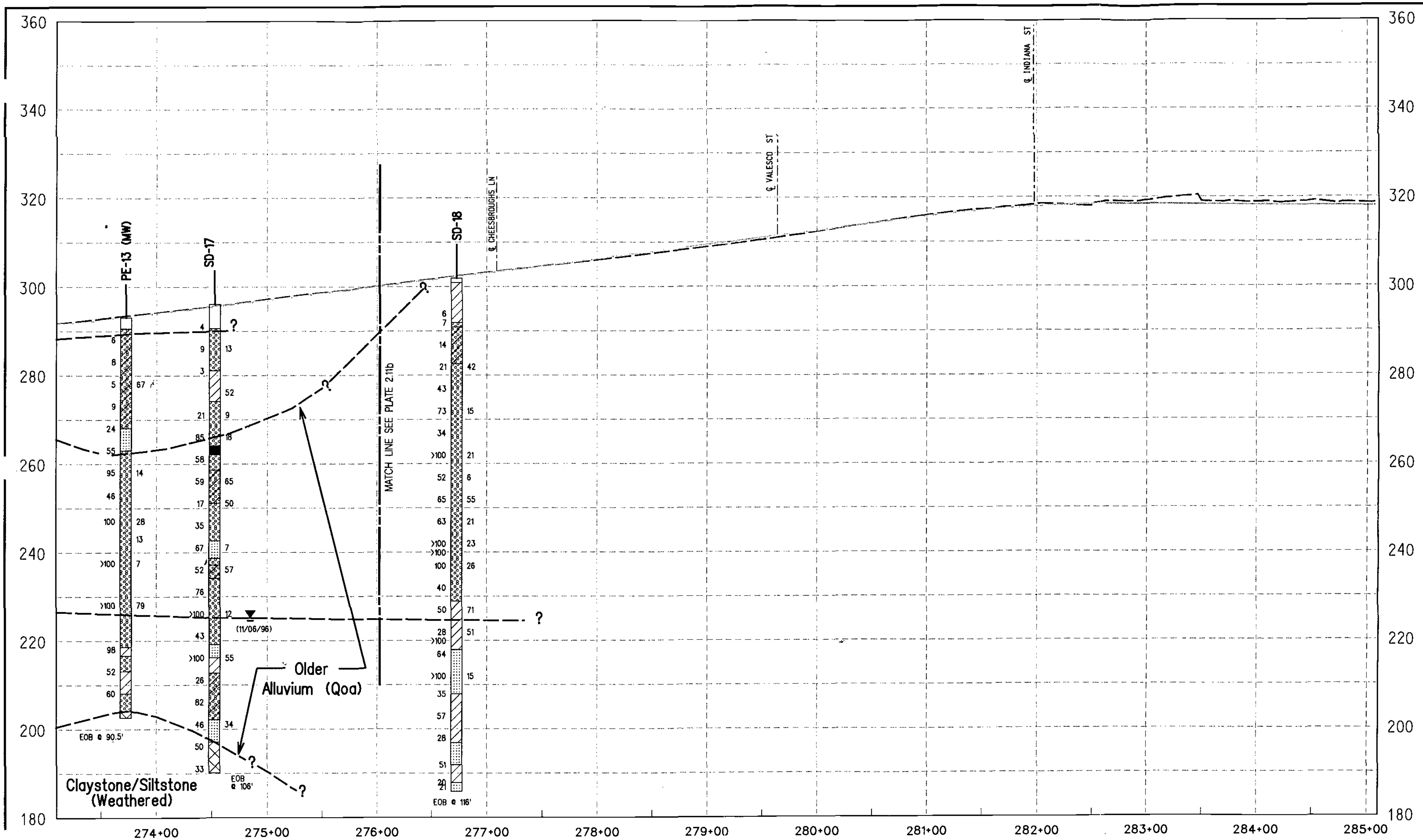
DRAWN BY: TEA	CHECKED BY: SB
SCALE: 1"-80'	PLATE 2.12a



NOTES:
LEGEND SHOWN ON PLATE 2.13
SOME BORINGS NOT SHOWN FOR CLARITY

BASE PROFILE PROVIDED BY:
EASTSIDE LRT PARTNERS
DATED OCTOBER 20, 2002

LAW Crandall
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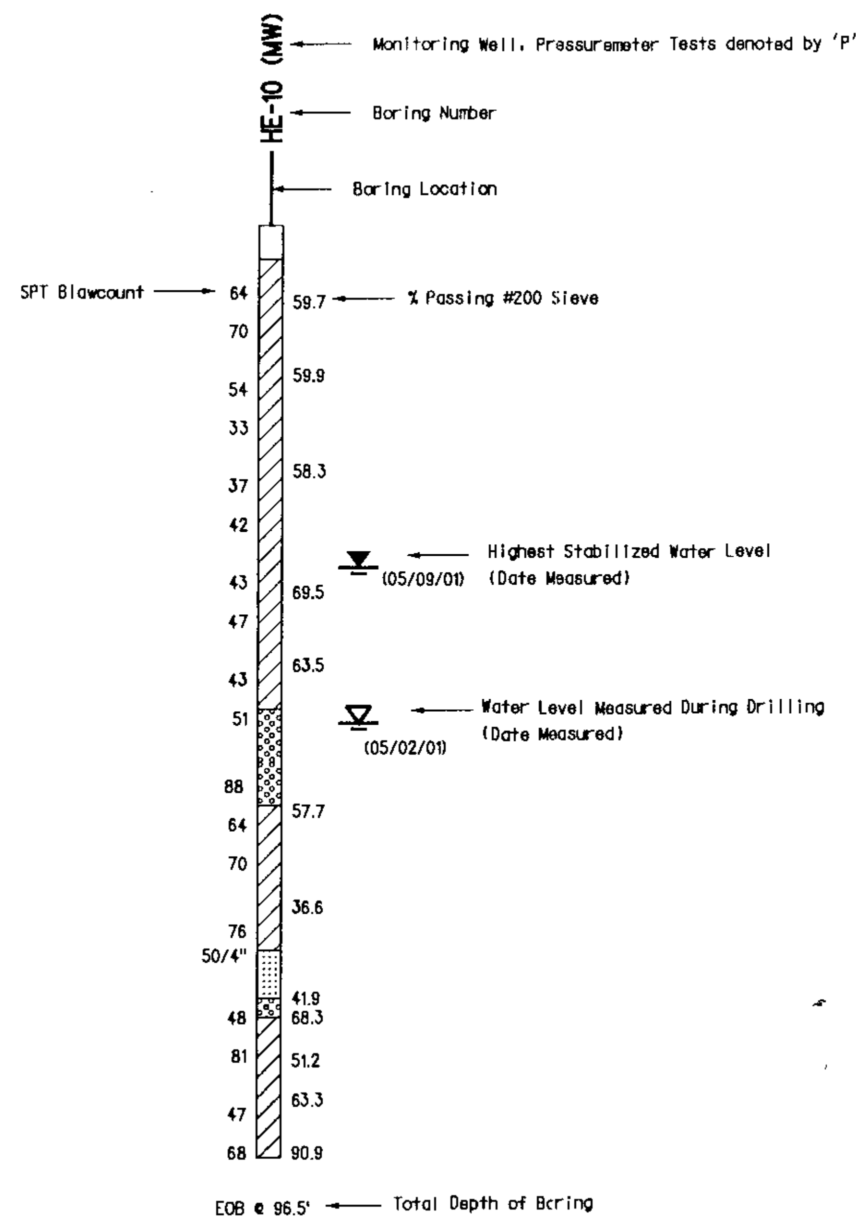
200 CITADEL DRIVE
LOS ANGELES, CALIFORNIA 90040
Tel. 323.889.5300 Fax. 323.889.5398

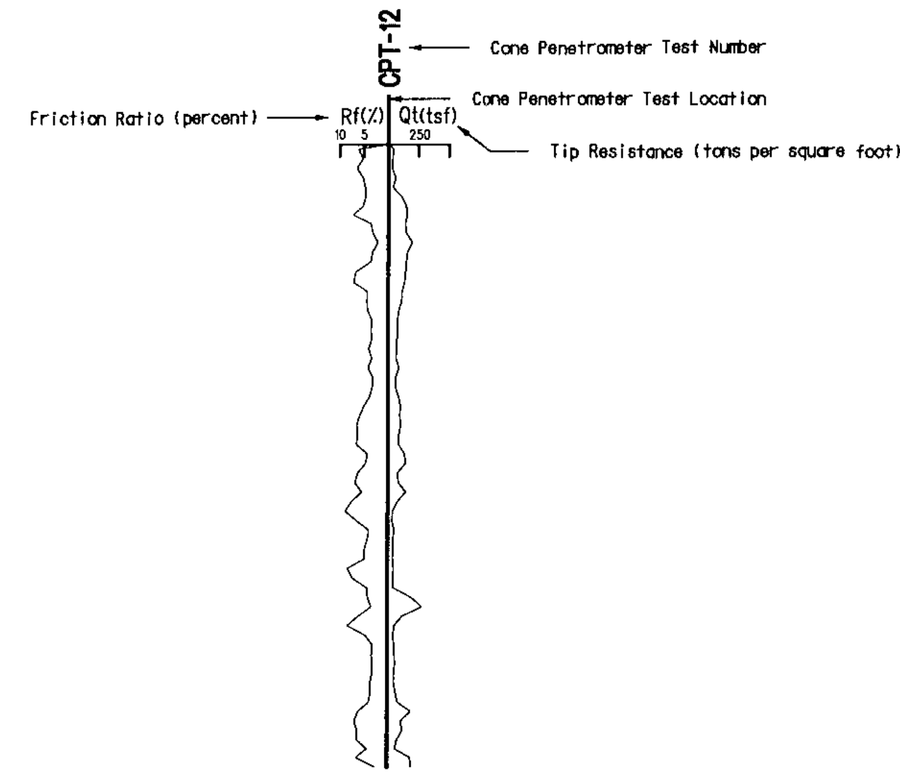
EASTSIDE LRT PROJECT
UNDERGROUND SEGMENT
GEOLOGIC PROFILE

JOB NO: 70131-1-0138.0005	DATE: 10/22/2002
DRAWN BY: TEA	CHECKED BY: SB
SCALE: HORIZ: 1"=80' VERT: 1"=20'	PLATE 2.12b

LEGEND:

	Artificial Fill (see boring log for details)
	Alluvium - fine-grained (greater than 50% fines) (predominantly low-plasticity clays)
	Alluvium - fine-grained (greater than 50% fines) with varying amounts of gravel (less than 3" in size) (see boring log for details)
	Alluvium - coarse-grained (less than 50% fines) (predominantly poorly graded sands and silty sands)
	Alluvium - coarse-grained (less than 50% fines) with varying amounts of gravel (less than 3" in size) (see boring logs for details)
	Alluvium - gravelly with cobbles (3" to 12" in size) (see boring logs for details)
	Bedrock - Claystone/Siltstone (weathered)





LAW Crandall LAWGIBB Group Member A Division of LAW Engineering and Environmental Services, Inc. 200 CITADEL DRIVE LOS ANGELES, CALIFORNIA 90040 Tel. 323.889.5300 Fax. 323.889.5398	EASTSIDE LRT PROJECT UNDERGROUND SEGMENT LEGEND FOR GEOLOGIC PROFILE	JOB NO: 70131-1-0138.0005	DATE: 10/22/2002
		DRAWN BY: TEA	CHECKED BY: SB
		SCALE: HORIZ: 1"-80' VERT: 1"-20'	PLATE 2.13

REFERENCE:

U.S.G.S. 7.5 MINUTE LOS ANGELES QUADRANGLE, DATED 1966, PHOTOREVISED 1981.
KEY PLANS AND GEOLOGY, DATED 5-96, BY GEO TRANSIT CONSULTANTS.

EXPLANATION:

Qya	YOUNGER ALLUVIUM	Q_4 and Q_5 of Bullard and Lettis (1993)
Qoa_u (Qalo of Lamar (1970)); 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	

GEOLOGIC CONTACT, short dashed between subunits of older alluvium

UNDERGROUND SEGMENT

FAULT / ESCARPMENT

FAULT, dotted where concealed, queried where uncertain

LOCAL GEOLOGY

SCALE 1" = 1000'

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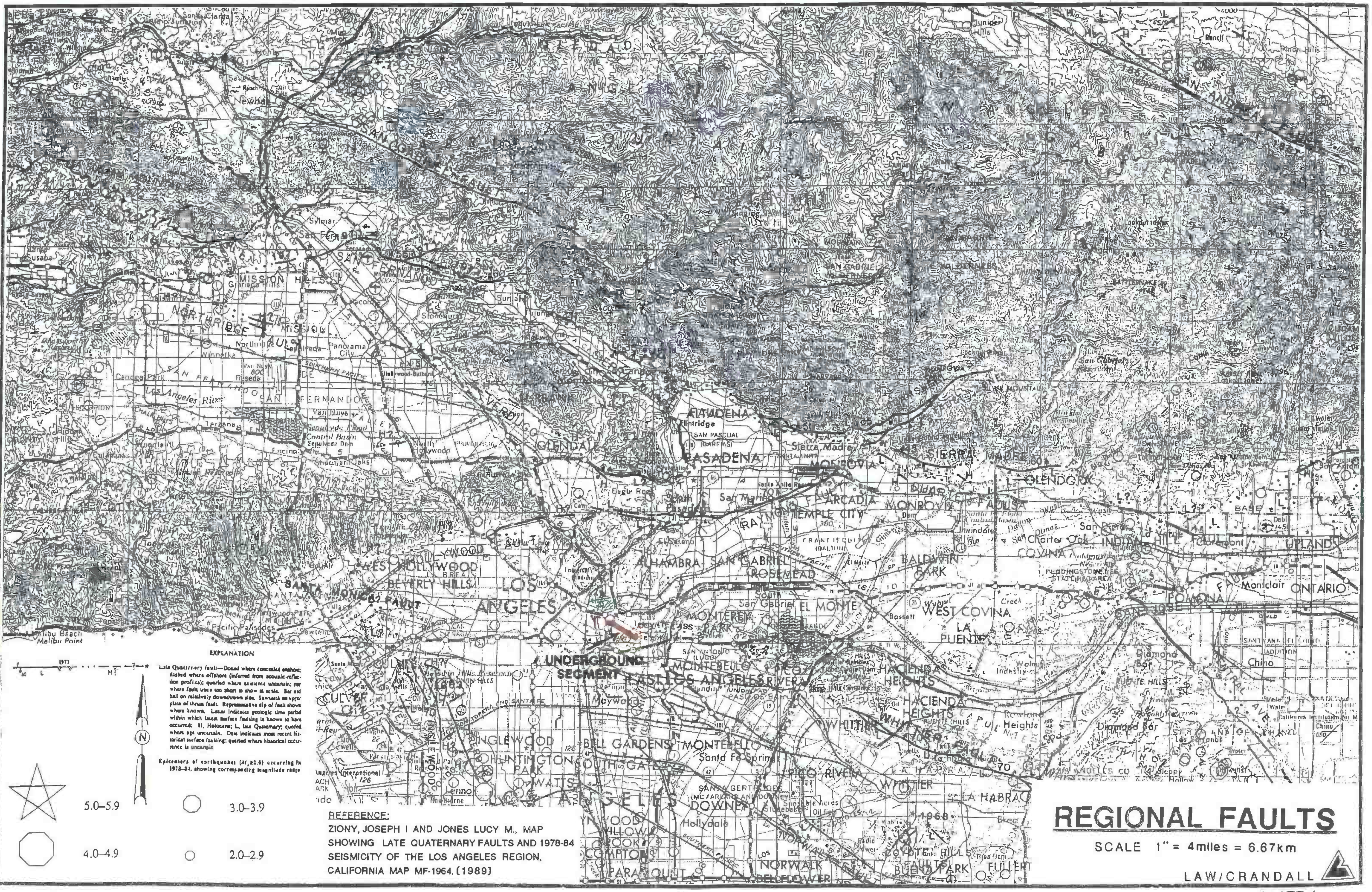
PLATE 3

JOB 70131-1-0138 DATE 06-28-01 F.T. ___ DR. ___ O.E. ___ CHKD ___



REFERENCE:
ZIONY, JOSEPH I AND JONES LUCY M., MAP SHOWING LATE QUATERNARY FAULTS AND 1978-84 SEISMICITY OF THE LOS ANGELES REGION, CALIFORNIA MAP MF-1964. (1989)

REGIONAL FAULTS

SCALE 1" = 4miles = 6.67km

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PLATE 4

FORM 137-A

JOB 70191-1-0193 DATE DR. O.E. CHKD.

EXPLANATION:

HISTORIC FAULT DISPLACEMENT

HOLOCENE FAULT DISPLACEMENT WITHOUT HISTORIC RECORD

YEAR M 8+

YEAR M 7-8

YEAR M 6-7

YEAR M 5-6

APPROXIMATE EPICENTRAL AREA OF EARTHQUAKE

0 20
SCALE IN KILOMETERS

SCALE 1: 750,000

0 12 24
SCALE IN MILES

REFERENCES:

1.) JENNINGS, C.W., 1994, "FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS WITH LOCATION AND AGES OF RECENT VOLCANIC ERUPTIONS", CALIFORNIA DIVISION OF MINES AND GEOLOGY, GDM-6.

2.) EARTHQUAKE CATALOGS: RICHTER, 1812-1932, NATIONAL OCEANIC ATMOSPHERIC ADMINISTRATION, 1812-1931; CALTECH, 1932-1997.

REGIONAL SEISMICITY

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PLATE 5

JOB DATE 06-28-01 F.T. DR. O.E. CHKD

FORM 137-A