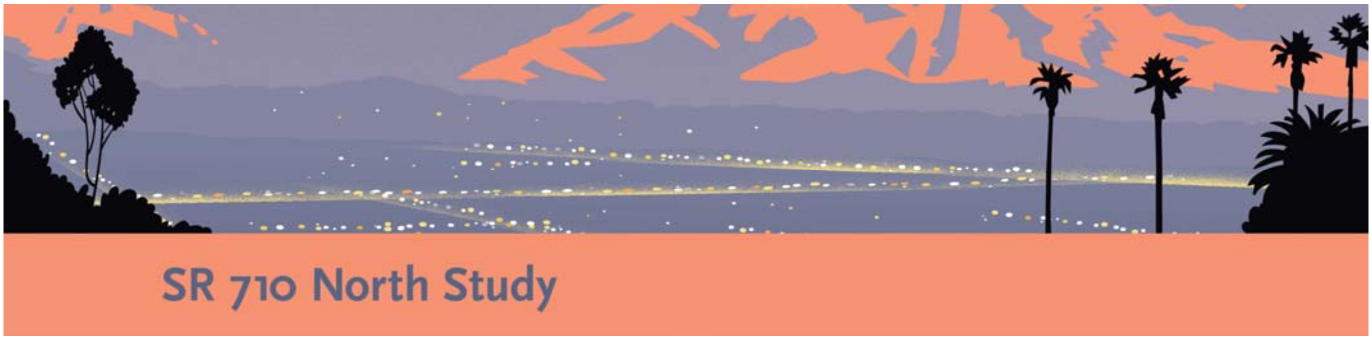


Tunnel Evaluation Report

Prepared for

Caltrans
CALIFORNIA DEPARTMENT OF TRANSPORTATION

Metro
Los Angeles County
Metropolitan Transportation Authority

September 5, 2014

Contents

Introduction

The California Department of Transportation (Caltrans), in cooperation with the Los Angeles County Metropolitan Transportation Authority (Metro) is studying transportation improvements to improve mobility and relieve congestion in the area between State Route 2 (SR 2) and Interstates 5, 10, 210 and 605 (I-5, I-10, I-210, and I-605, respectively) in east/northeast Los Angeles and the western San Gabriel Valley. At the time of the writing of this report, the Environmental Impact Report/Environmental Impact Statement (EIR/EIS) is in the process being drafted, and this report serves as a reference document to the environmental documentation process. The alternatives being considered at this stage of the study are discussed below. This report focuses on the tunnel sections associated with these alternatives.

1.1 Current Project Alternatives

The proposed alternatives selected for evaluation in the EIR/EIS phase of the study include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. Previously, during the Alternatives Analysis phase, other alternatives were considered; however, these five were selected for further study (CH2M Hill, 2012). The Freeway Tunnel and LRT Alternatives will include tunnels for significant distances over their alignments, and only these alternatives will be discussed further in this report. The other alternatives do not involve tunnel sections and are not within the scope of this report. The alignments of the LRT and Freeway Tunnel Alternatives are shown in Figures 1 and 2 and are described below.

1.1.1 LRT Alternative

The LRT Alternative (Figure 1) would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 miles long, with 3 miles of aerial segments and 4.5 miles of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with excavated tunnel diameters of approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations. Additionally, two of these underground station excavations would include additional space for a crossover (Huntington) and tail tracks (Fillmore). Additional information about the LRT Alternative, including discussions on aerial and at-grade segments, is included in the Advanced Conceptual Engineering Report (CH2M Hill, 2014a).

1.1.2 Freeway Alternative

The alignment for the Freeway Tunnel Alternative (Figure 2) starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. Both variations are approximately 6.3 miles long, with 4.2 miles of bored tunnel, 0.7 miles of cut-and-cover tunnel, and 1.4 miles of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), and each tunnel would have two levels with two lanes of traffic on each level, for a total of four lanes in each tunnel. Each bored tunnel would have an excavated diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level. Additional information about the Freeway Tunnel Alternative, including discussions on aerial and at-grade segments, is included in the Project Report (CH2M Hill, 2014c).

1.2 Scope

This report summarizes the preliminary design concepts developed for the bored tunnels and some associated underground components for the Freeway Tunnel and LRT Alternatives. Eleven technical memoranda (TMs) have been prepared to describe the preliminary design concepts developed for this stage of the study. Each of these TMs is included as a separate appendix to this report. The following TMs were prepared as part of this study:

- TM-1 Bored Tunnel Geometry
- TM-2 Tunnel Ground Characterization
- TM-3 Tunnel Excavation Methods
- TM-4A Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings
- TM-4B Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings
- TM-4C Preliminary Design Concepts for the Freeway Tunnel Internal Structure
- TM-5 Evaluation and Control of Ground Movements
- TM-6 Preliminary Design Concepts for Fault Crossings
- TM-7 Preliminary Design Concepts for the Freeway Portal Excavation Support Systems
- TM-8 Preliminary Design Concepts for the LRT Station and Portal Excavation Support Systems
- TM-9 Handling and Disposal of Excavated Materials

1.3 Purpose and Limitations

The purpose of this report, including the appended TMs, is to document the preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for a preliminary construction cost estimate developed for the tunnel sections.

The preliminary design concepts presented in each TM were considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement in future phases of this study if either of the bored tunnel alternatives is carried further.

The preliminary design concepts presented herein are limited to the geotechnical information available to date (CH2M Hill, 2014b). Concepts are expected to be modified and refined as additional geotechnical information becomes available, including the optimization of the horizontal and vertical tunnel alignments. However, such modifications or refinements are expected to be within the range of construction activities already considered and discussed in the environmental studies. It should also be noted that each TM also contains recommendations or suggestions for additional work that would need to be considered in future design phases.

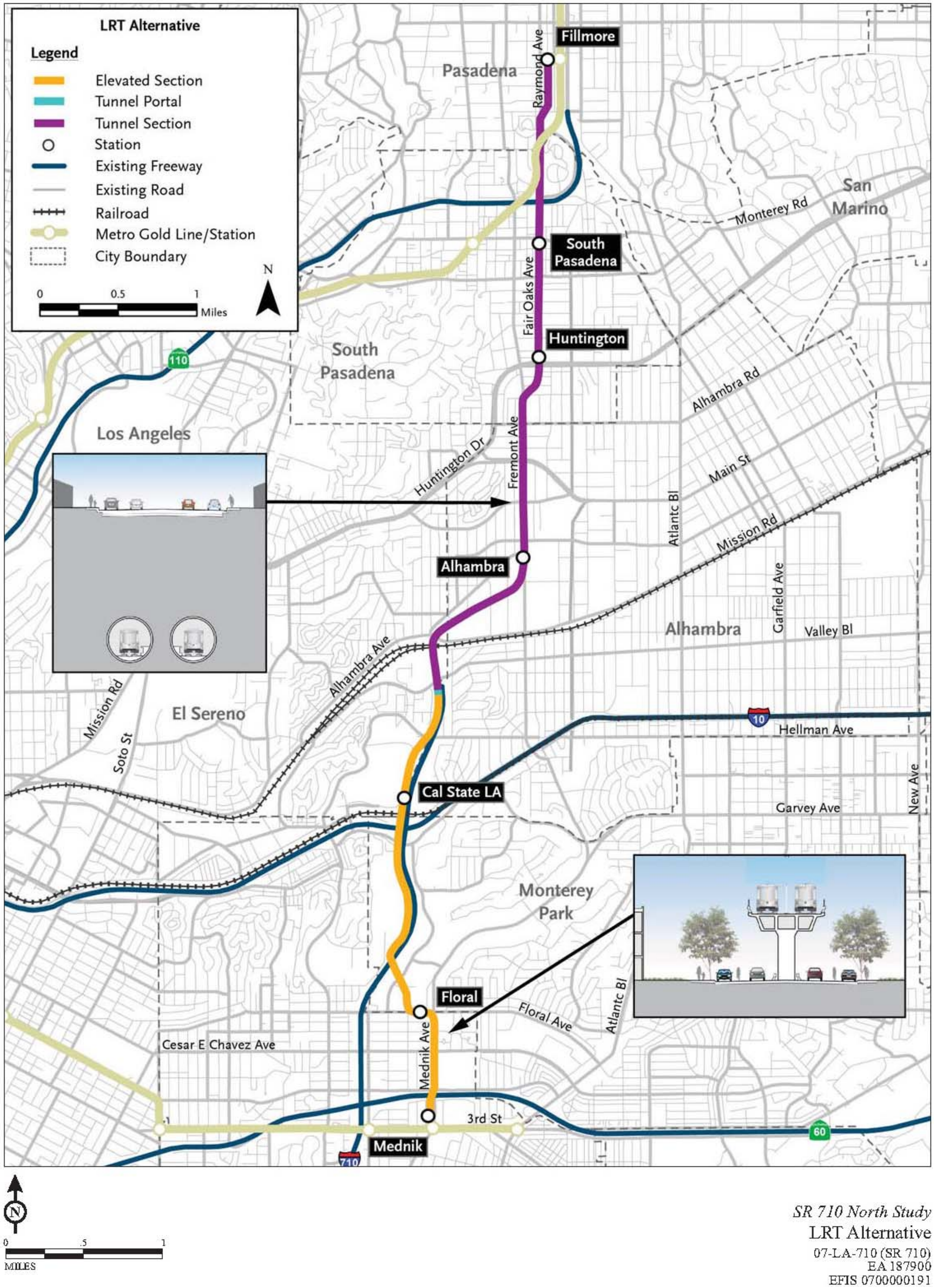


FIGURE 1
LRT Alternative

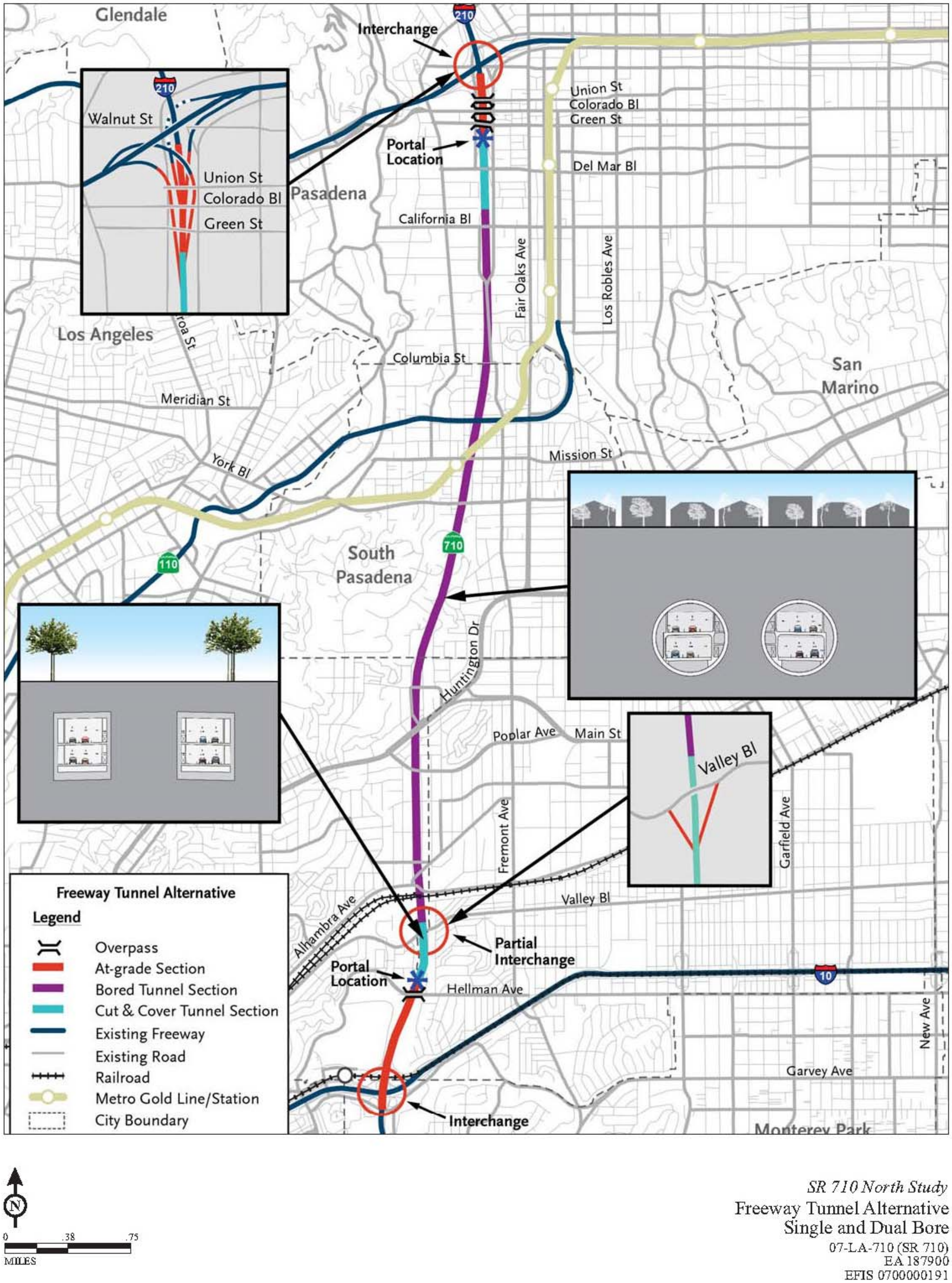


FIGURE 2
Freeway Tunnel Alternative

Summary of Technical Memoranda

The TMs prepared for the tunnel sections of this stage of the study are summarized below; each TM is provided in its entirety as separate appendices to this report.

2.1 TM-1: Bored Tunnel Geometry

TM-1 presents the criteria used and the development process followed to determine the cross-sectional geometry for the bored tunnels as well as requirements for spacing and sizing of cross passages, where applicable.

Metro and the National Fire Protection Association (NFPA) have published standards and guidelines for LRT bored tunnels that were consulted in determining the cross section of the LRT tunnels. Metro criteria specify an inside diameter (ID) for an LRT tunnel as 18.83 feet and also provide clearance envelopes for light rail bored tunnels. The resulting outer diameter (OD) of the final lining is determined to be about 20.5 feet. Cross passages are proposed along the LRT tunnel as emergency exits at a spacing not to exceed 800 feet, with a spacing of 750 feet being preferred. The clearance envelope inside the cross passages is 6.5 feet wide and 8 feet high, with a minimum height of 7 feet.

The bored tunnel configuration for the freeway is governed by regulatory agency requirements as well as the space required for ventilation, traffic operations, and equipment. The tunnel configuration is largely determined by required horizontal and vertical freeway clearances and other uses of tunnel space, such as for emergency egress, ventilation ducts, drainage, communications, and utilities. Current regulations, guidelines, and criteria established by Caltrans, FHWA, NFPA, and other regulating agencies were reviewed when developing the bored tunnel design and configuration. Based on the operational needs and the regulatory requirements reviewed at this time, it is expected that the components of the freeway tunnel fit within an ID of 52.5 feet. The thickness of the segmental lining shown at this preliminary phase is expected to be 30 inches, which results in a tunnel lining OD of about 58.5 feet.

Emergency vehicle cross passages are currently being recommend for the twin-bore Freeway Tunnel Alternative at a spacing of approximately 3,000 feet. These cross passages are expected to be used to provide first responders with an alternate method to reach an incident, possibly reducing the amount of time it takes to arrive at the location of an incident in the event of an emergency or unplanned event. The emergency vehicle cross passages are expected to have a clearance envelope that is 20 feet wide and 14.5 feet high, and constructed at a skew angles of 50 degrees between the bored tunnels. As there are two levels of traffic, there would be a separate cross passage for each of the upper and lower levels.

2.2 TM-2: Tunnel Ground Characterization

TM-2 describes the results of evaluations performed to characterize ground conditions for the bored tunnels and cross passages, including the determination of tunnel reaches and identification of ground classes. Ground characterization involves evaluating available geologic data, assessing soil/rock mass properties, identifying distinct rock mass types, estimating anticipated ground behaviors along the proposed tunnel alignments, and developing geotechnical parameters for preliminary design evaluations.

These geologic units/formations include both soft ground and rock formations consisting of artificial fill, alluvium, Fernando Formation, Puente Formation, Topanga Formation, and basement complex rocks (Wilson Quartz Diorite). Ground conditions that are expected to be encountered along the proposed Freeway and LRT tunnel alignments have been divided into four ground classes to assist in the selection of tunnel excavation and support methods. Ground classes are defined based on the physical characteristics of the ground and its potential behaviors during tunnel excavation. They include the following (refer to TM-2 for additional information):

Ground Class 1: fair to good rock conditions
Ground Class 2: poor rock conditions
Ground Class 3: soft ground or mixed-face ground conditions
Ground Class 4: faulted or sheared ground conditions

2.3 TM-3: Tunnel Excavation Methods

TM-3 presents and evaluates feasible excavation methods for the bored tunnels and cross passages. Based on the tunnel lengths and ground conditions, excavation of the freeway and LRT running tunnels are expected to be mined with tunnel boring machines (TBMs), and the cross passages are expected to be excavated using the sequential excavation method (SEM).

The LRT alternative alignment is expected to be excavated through conditions that vary between alluvium and the weak sedimentary rock formation, and the Freeway alternatives would be excavated in the alluvium, the sedimentary rock formations, and basement rock. Both bored tunnel alternatives would have portions expected to be excavated completely below the groundwater table. A pressurized-face TBM is ideally suited for the bored tunnels due to the potential for high groundwater pressures combined with the varying permeability and strength of the soil units, including mixed-face conditions. Different types of pressurized-face TBMs are described, as well as their applicability based on ground conditions and other factors.

Unlike the bored tunnels, cross passages would require installation of an initial support system followed by a final lining. The SEM is flexible for tunnels in weak and variable ground conditions and considered to be the most suitable and economical method of constructing the cross passages. Different mining methods to excavate the ground are discussed, as well as their applicability to the expected ground conditions in the cross passages. Additionally, the use of systematic ground improvement measures to stabilize the openings and control ground movements and groundwater inflows during excavation of the cross passages are discussed.

The following other tunnel excavation considerations are also briefly discussed in this TM:

- Construction power requirements
- Handling potential manmade obstructions
- Contractor’s laydown areas
- TBM abandonment (Freeway Tunnel Alternative only)

2.4 TM-4A: Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings

TM-4A describes preliminary tunnel lining and cross passage design concepts for the twin-bore and single-bore variations of the Freeway Tunnel Alternative. In this TM, the applicable codes and guidelines to tunnel and cross passage lining design are discussed as well as other criteria including but not limited to service life, durability, watertightness, and fire resistance. The design loads used in the development of the preliminary design concepts are also explained, as well as the loading combinations and load factors used.

The results of the analysis indicate that a 30-inch-thick precast concrete segmental lining is sufficient to withstand the loading conditions, given the current geotechnical information available; however, it should be expected that this design will be refined in future phases of this project. The cross passages are expected to be excavated using the SEM, as discussed in TM-3, and would consist of a two-pass lining system of the initial shotcrete lining and the cast-in-place concrete final lining. For the preliminary design concepts of the initial support, ground conditions at various cross passage locations have been categorized into three ground classes (out of the four discussed in TM-2). Each ground class is expected to respond similarly to tunneling operations and would require similar initial support types. Numerical analyses were performed to determine preliminary concepts for the excavation sequence and initial support requirements for each ground class. Additional numerical analyses were performed to determine the static and seismic requirements for the final lining of the cross passages. This TM also discusses other design considerations such as the temporary breakout support sequencing and requirements, as well as the permanent ring beam, which is expected to be required where the cross passages interface with the running tunnels.

2.5 TM-4B: Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings

TM-4B describes the preliminary design concepts developed for tunnel and cross passage lining systems for the bored tunnel portions of the LRT Alternative. These design concepts were developed from experience with other similar LRT tunnels in the Los Angeles area. The TM provides a general description of the geology along the tunnel alignment, as well as discussions of applicable design criteria, general lining requirements, and an overall design methodology that can be applied to the LRT tunnels. In this TM, the applicable codes and guidelines to tunnel and cross passage lining design are detailed, as well as other criteria including, but not limited to, service life, durability, watertightness, and fire resistance. The design loads recommended for use in the development of the preliminary design concepts are also explained, as well as recommended loading combinations and load factors.

At this stage of the design, the lining design concepts for the LRT tunnels is based primarily on relevant past experience, which includes the Regional Connector Transit Corridor and Metro Gold Line Eastside Extension projects in Los Angeles, as well as the University-Link Rail in Seattle, Washington. The preliminary design concept for the lining of the LRT alternative is a gasketed segmental precast reinforced concrete lining with an approximate thickness of 10 inches; however, a detailed analysis should be performed in future phases of this project. Similar to the cross passages of the Freeway Tunnel Alternative, the cross passages for the LRT Alternative are expected to be excavated using the SEM, and would consist of a two-pass lining system of the initial shotcrete lining and the cast-in-place concrete final lining. Different support types based on ground classes along the LRT alignment are detailed in this TM, and recommended for the cross passage initial support. The preliminary design concept for the final lining of the LRT cross passages is presented based on local experience on other Metro projects.

2.6 TM-4C: Preliminary Design Concepts for the Freeway Tunnel Internal Structure

TM-4C describes the preliminary design concepts for the internal structure elements of the twin-bore and single-bore variations of the Freeway Tunnel Alternative. The internal structure consists of the horizontal and vertical elements (mainly slabs and walls) to be constructed within the tunnel after the excavation is performed and the support installed. These elements would eventually make up the roadway decks and the walls separating the travel lanes from the emergency exit walkways and ventilation ducts. In this TM, the applicable codes and guidelines to tunnel internal design are detailed as well as other criteria including but not limited to service life, durability, watertightness, and fire resistance. The design loads used in the development of the preliminary design concepts are also explained as well as the loading combinations and load factors used.

The preliminary design concept presented at this time for the internal structure consists of a double-deck roadway constructed of cast-in-place (CIP) and precast reinforced concrete. Each deck has two 12-foot-wide travel lanes, a 1-foot shoulder, and a 10-foot shoulder for vehicles. Each deck also has a walkway, which serves as the emergency egress route in the event of an emergency such as a tunnel fire. The results of the preliminary analysis indicate that 26-inch-thick slabs for the roadway decks and 10- to 16-inch-thick walls are sufficient to withstand the anticipated loading conditions. This analysis would be refined in future phases of this project as other loadings—such as dead weight of tunnel system components that the internal structure would support and vehicle collision loads—become better defined. The performance of the internal structures would also be evaluated further to ensure adequate resilience to fires. Several measures are discussed to ensure the satisfactory performance of the internal structure when exposed to fire. The types of fire-resistant elements/provisions required and their impact on the ventilation systems would also be evaluated further.

2.7 TM-5: Evaluation and Control of Ground Movements

TM-5 discusses methods of evaluating and controlling construction-induced surface ground movements and presents a methodology to determine their potential effects along the proposed Freeway Tunnel and LRT Alternatives. A preliminary analysis of excavation-induced ground movements has been performed using semi-empirical methods to determine the extents of the zone of potential excavation-induced ground movement influence, which is identified in this TM.

The focus of these evaluations is on vertical ground movements. The induced ground movements would form a settlement trough transverse to the proposed bored tunnels, which is estimated using a semi-empirical method that assumes that the shape of the settlement trough above a single tunnel follows a Gaussian distribution and that the volume of the settlement trough is equal to the total volume of ground lost during tunneling. The total vertical ground movements caused by two tunnels are the sum of the ground movements caused by each individual tunnel. For this study, a volume loss of 0.5% was adopted for the alluvium and 0.25% for the weak sedimentary rock formations based on case histories of similar projects. For cut-and-cover excavations, associated ground movements can also be estimated using semi-empirical methods based on case histories of ground movement next to excavation support walls.

Several options for control of ground movement through prevention and mitigation are discussed in this TM. Over the past 10 to 15 years in the U.S., pressurized-face machines have been used as the primary mitigation to reduce the risk of excessive ground loss during excavation, as well as to minimize overall loss of ground, and subsequent ground movements due to tunneling, as compared to the use of open-face tunnel excavation methods. In addition to requiring the use of a pressurized-face TBM, the project could also specify requirements such as selecting a pre-qualified contractor with experience mining with pressurized-face TBMs and requiring that the ground loss be limited to a certain percentage, and that the contractor demonstrate that he can achieve that ground loss percentage with the machine selected. Additionally, it could be specified that practices such as monitoring muck volumes, integrated tail void grouting, and a real-time instrumentation and monitoring program be in place during TBM excavation.

In further phases of design, further evaluation, including structure-specific analysis, would be performed to better understand the response of the structures along the preferred alignment to the excavation-induced ground movements.

2.8 TM-6: Preliminary Design Concepts for Fault Crossings

TM-6 discusses the evaluation of the preliminary design concepts for the portions of the bored tunnels that cross active and potentially active fault zones along the tunnel alignments for the Freeway Tunnel and LRT Alternatives. Because of the fault offset expected at these fault zones, widened tunnel cross sections (called vault sections) are proposed to accommodate the fault offset. The focus of the discussion is on the design criteria and design basis related to design concepts for the fault crossings and the design methodology that is employed to evaluate a feasible fault crossing concept for each bored tunnel alternative. Site-specific geotechnical investigations have yet to be completed at each of the various fault zones; future design studies will require site-specific data to be obtained in order to refine the design concepts discussed herein.

The Freeway Tunnel Alternative is anticipated to cross one active and two potentially active faults: the Raymond fault is considered an active fault, while the Eagle Rock and San Rafael faults are designated as potentially active, but are treated as active in this study. The tunnel portion of the LRT Alternative is anticipated to cross the Raymond and San Rafael faults. The expected design horizontal and vertical offsets for these faults are discussed in this TM. These potential offsets could induce significant stresses in the tunnel linings, resulting in cracking and deformation (shearing) of the linings. To minimize the damage, special design features must be incorporated into the design to accommodate the anticipated ground offsets and minimize the impact of potential overstressing in the linings.

Because of the magnitude of the design fault offsets for this study, special design features such as an enlarged vault section or special lining design for the fault crossings are considered to be necessary. An enlarged vault section with a robust lining system has been chosen as one viable preliminary design concept to move forward with in this study. In this option, structural rings with circumferential joints between them are designed to allow slippage in the fault zone, and the enlarged cross section would accommodate the fault offset.

The concept for the Freeway Tunnel Alternative would consist of installing high-strength steel segments with a thickness of 20 inches in the fault zone, as compared to the 30-inch-thick precast concrete segments used for the remainder of the bored tunnel. The difference in the thickness is large enough to accommodate both the horizontal and vertical components of the fault offset, whilst still providing a stronger section.

Based on the anticipated fault offset, the LRT Alternative would require an oversized vault section be excavated in the fault zone after the completion of the bored tunnels. The vault section would require overexcavating the section of tunnel within the fault zone large enough so that a 36-inch-thick cast-in- place lining section could be constructed. The cast-in-place lining will be designed to accommodate the expected fault offsets. These concepts will be explored further along with other options in future phases of the project.

2.9 TM-7: Preliminary Design Concepts for the Freeway Portal Excavation Support Systems

TM-7 describes the preliminary design concepts developed for excavation support systems of the construction portals for the Freeway Tunnel Alternative. Construction portals for the Freeway Tunnel Alternative would be located at each end of the bored portion of the tunnel alignment. Because mining might occur from both portals with two TBMs excavating each bore, both portals would support construction activities and also serve as a laydown area for the contractor during construction. The portal excavations are expected to remain open for the duration of tunnel excavation and construction, and ultimately permanent roadway features (such as cut-and-cover portions of the tunnel) and permanent portal structures will be constructed at the completion of the project; however, the permanent works are not within the scope of this TM.

The north portal excavation for the twin-bore variation would be approximately 250 feet wide and 80-100 feet deep at the portal headwall due to an existing ground slope. The side walls of the portal would also be of a similar height near the head wall, and would decrease in height as the future roadway continues north and the excavation becomes shallower. The width of the excavation is sufficient to launch two TBMs and also to accommodate the permanent cut-and-cover tunnels, which would be constructed after excavation of the bored tunnels. The excavation for the north portal would be entirely in alluvium and fill and is not expected to encounter groundwater, so groundwater control measures are not expected to not be necessary for portal excavation. A soldier pile and timber lagging wall supported with tiebacks is considered suitable from a constructability and structural design standpoint, and is being assumed for this portion of the study, though other options may be feasible.

Similar to the north portal, the south portal excavation for the twin-bore variation would be approximately 250 feet wide and 125 feet deep. The subsurface conditions consist of 25 to 60 feet of loose to very dense alluvium underlain by the Puente Formation. The water table is approximately 25 feet below the current ground surface throughout the portal excavation. Primary design considerations at the south portal include groundwater control and the long-term strength and behavior of the Puente Formation. Dewatering may be considered for groundwater control, or wall systems can be used to cut off water from the excavation. The rock-like fractured and jointed portion of the Puente Formation, if encountered at the base, may require base grouting (permeation or pressure grouting) in order to seal off the joints and fractures from groundwater inflows. Slurry walls with tiebacks are considered a suitable support of excavation method given the depth of wall installation and the presence of groundwater, though other options may be feasible.

The preliminary design concepts for the portals for the single-bore variation are similar in depth and concept to those described for the twin-bore variation, but the width is reduced by approximately 115 feet to accommodate the single tunnel bore and only one TBM per portal.

2.10 TM-8: Preliminary Design Concepts for the LRT Station and Portal Excavation Support Systems

TM-8 describes the preliminary design concepts developed for the excavation support systems for the LRT south construction portal and four underground LRT station excavations. The bored tunnels of the LRT Alternative are expected to be mined using two TBMs launched from the south portal, and that portal would also be used to stage construction activities for the tunneling work. Four underground stations are expected to be excavated using cut-and-cover techniques in advance of the TBM arrival at each location; it is common that the stations are excavated first, but is not necessary and will depend on project requirements. The alignment terminates at the Fillmore Station, and the TBMs would be retrieved from that location. The final, permanent structures to be constructed within the temporary excavations discussed in this TM are not addressed in this TM.

The portal excavation is approximately 50 feet high and 70 feet wide at the headwall. The side walls of the temporary excavation would be of a similar height near the headwall, and would decrease in height to the south as the excavation becomes shallower. The portal would ramp down from the existing ground surface to gain enough cover to launch the TBMs; both TBMs for the LRT alternative would be launched at the headwall of this portal. Geotechnical conditions indicate alluvial soils within the excavated height and that groundwater could potentially be encountered in the deeper portion of the excavation near the portal headwall. Preliminary design considerations would be preventing the alluvium from sloughing into the excavation, and a secondary consideration would be controlling groundwater near the portal headwall, if encountered. A soldier pile and timber lagging wall supported with tiebacks is considered suitable from a constructability and structural design standpoint; however, other options may be feasible.

There are four underground stations, which are expected to be excavated with cut-and-cover techniques; these include the Alhambra Station, Huntington Station, South Pasadena Station, and Fillmore Station. The station excavations are approximately 80 to 100 feet deep and 80 feet wide, with the length of each station excavation varying from 400 feet for a standard station, up to over 1,000 feet if there is a crossover or tail tracks adjacent to it (the Huntington Station includes an excavation for a crossover and the Fillmore Station includes an excavation for tail tracks). At the four stations, the geotechnical conditions indicate that they are expected to be excavated wholly in alluvial soils or in a combination of alluvial soils and weak sedimentary rock. The Alhambra Station is expected to have groundwater in the bottom 20 feet of the excavation. All stations are excavated in an urban setting in the public right-of-way with buildings and structures immediately adjacent to the excavations.

Soldier piles and lagging have been successfully used for past Metro projects under similar conditions and therefore have been selected as the wall type at this conceptual design level. Tiebacks and/or internal bracing can be used for lateral wall support. Tiebacks can be installed through the soldier pile itself, and internal bracing, if used, would consist of walers and struts. Similar to TM-7, this TM also includes a discussion of ground support at the portal, and station, headwalls to support break-in and break-out of the TBMs.

2.11 TM-9: Handling and Disposal of Excavated Materials

TM-9 describes several aspects of handling and disposal of excavated materials for the Freeway and LRT alternatives. The Freeway and LRT bored tunnels would be mined with TBMs, and the excavated material generated from the tunneling operations would be removed at the portals. In addition to the bored tunnels, excavation of construction portals, cross passages, and LRT stations would also generate spoil material.

This TM evaluates the anticipated properties (including bulking factors) for excavated materials based on the geologic formations along the tunnel alignments. Bulking factors and unit weights have been estimated for each geologic unit expected to be encountered in the excavation; assumed bulking factors range from 1.3 to 1.6. Estimates of approximate quantities and weights of excavated material that would be generated from the tunnel, portal, and LRT station excavations for each alternative are presented based on the percentage of the excavation in each geologic unit. These estimates are presented as total volumes and weights for each component of the excavation process.

A discussion of the anticipated excavated material conditions from TBM tunneling operations (including those resulting from conditioning additives) and the handling of excavated material at the work areas is also included in this TM.

References

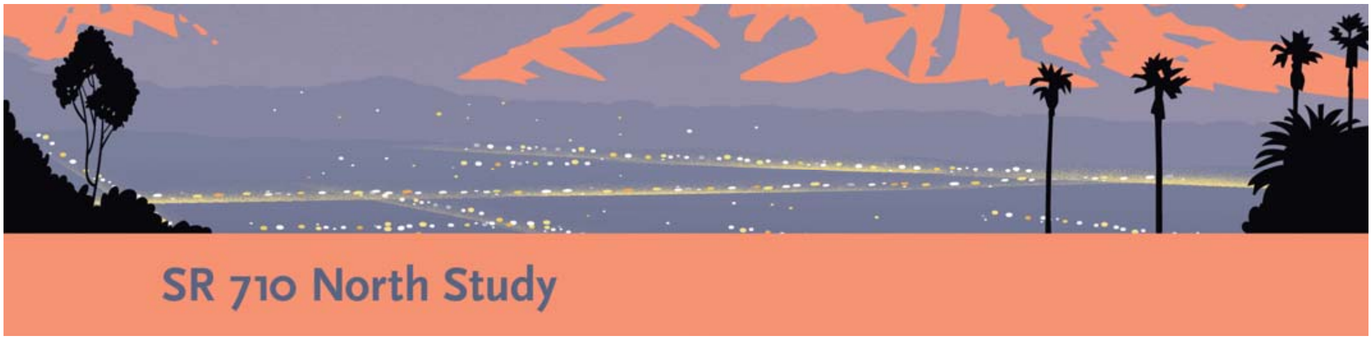
CH2M Hill (2012), *State Route 710 Study Alternatives Analysis Report*. Prepared for Metro. December.

CH2M Hill (2014a), *SR 710 North Study Advanced Conceptual Engineering Report LRT Alternative*. Prepared for Metro. April.

CH2M Hill (2014b), *SR 710 North Study Draft Preliminary Geotechnical Report*. Prepared for Metro. August.

CH2M Hill (2014c), *SR 710 North Study Draft Project Report*. Prepared for Metro. August.

Appendix A
TM- 1 Bored Tunnel Geometry



TECHNICAL MEMORANDUM 1

Bored Tunnel Geometry

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) presents the background on the development of the cross section for the bored tunnel for both the Freeway Tunnel and LRT Alternatives. Applicable codes, standards and assumptions made during the process are documented herein. Requirements for cross passages for the twin-bore freeway tunnel and the LRT tunnel will also be discussed. This TM does not document the roadway design, ventilation, or tunnel systems designs except where such requirements affect the diameter of the bored tunnel. The cross section of the cut-and-cover portions of the freeway and LRT tunnels is not within the scope of this TM.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Freeway Tunnel Cross Section Geometric Requirements

For the Freeway Tunnel Alternative, the bored tunnel configuration is governed by regulatory agency requirements as well as the space required for ventilation, traffic operations, and equipment. The tunnel configuration is largely determined by required horizontal and vertical freeway clearances and other uses of tunnel space, such as for emergency egress, ventilation ducts, drainage, communications, and utilities.

Current regulations, guidelines, and criteria established by Caltrans, FHWA, and other regulating agencies, outlined below, were reviewed when developing the bored tunnel design and configuration. It should be recognized that tunnels of this size and length are not routine, so there is little precedent to draw from. Two tunnels that were considered as a basis for some of the allowances were the Caldecott 4th Bore Tunnel and Devil’s Slide Tunnel, both in northern California; these are Caltrans’ most recent highway tunnels. Additionally, the SR-99 Tunnel, which is currently under construction in Seattle, Washington, was also referenced as a basis for comparison. It is important to note that engineering standards and applicable regulations that do pertain to tunnels of this size change with time; therefore, it will be important to revisit the criteria as the project proceeds through the planning, design, and environmental review phases.

Chapter 300 of the Caltrans Highway Design Manual (HDM), “Geometric Cross Section,” provides guidance on dimensions for roadway width, shoulders, and other horizontal and vertical clearances (Caltrans, 2012). Additionally, the US Department of Transportation Federal Highway Administration’s (FHWA) Technical Manual for Design and Construction of Road Tunnels (2009) was reviewed. Requirements from the Americans with Disabilities Act (ADA, 2002) and National Fire Protection Association (NFPA) were also consulted.

The following sections document the requirements that were considered in determining the conceptual cross section for the Freeway Tunnel Alternative.

2.1 Project Requirements

Two options are being considered for this study – a twin-bore tunnel and a single-bore tunnel. In the twin-bore configuration, the project would have four lanes of traffic in each direction (two lanes on each level), one direction per bore, for a total of eight lanes. In the single-bore option, there would be two lanes of traffic in either direction, (two lanes on the upper level and two lanes on the lower level) for a total of four lanes.

2.2 Travel Lanes

In accordance with Index 301.1 of the Caltrans HDM (2012), the standard lane width should be 12 feet per lane. This is consistent with the recommendation from the FHWA (2009). A travel lane width of 12 feet has been adopted for this study.

2.3 Shoulders

The FHWA and Caltrans guidelines as well as other existing tunnels were referenced in determining requirements for shoulder widths. Section 2.4.2, "Travel Lane and Shoulder" of The FHWA Technical Manual for Design and Construction of Road Tunnels states:

> Although the Green Book states that it is preferable to carry the full left- and right-shoulder widths of the approach freeway through the tunnel, it also recognizes that the cost of providing full shoulder widths may be prohibitive. Reduction of shoulder width in road tunnels is usual. In certain situations narrow shoulders are provided on one or both sides. Sometimes shoulders are eliminated completely and replaced by barriers. Based on a study conducted by World Road Association (PIARC) and published a report entitled "Cross Section Geometry in Unidirectional Road Tunnels" 2001; shoulder widths vary from country to country and they range from 0 to 2.75 m (9 ft). They are generally in the range of 1 m (3.3 ft). It is suggested for unidirectional road tunnels that the right shoulder be at 4 ft (1-2 m) and left shoulder at least 2 ft (0.6 m).

Shoulder requirements from Index 309.3, "Tunnel Clearances," of the Caltrans HDM were also consulted. Index 309.3 states the following for tunnels:

> Tunnel construction is so infrequent and costly that the horizontal width should be considered on an individual basis. For minimum width standards for freeway tunnels see Index 309.1.

At the direction of Metro, which considers the direction from the FHWA and the Caltrans HDM as well as precedent from existing tunnels, the design widths of the shoulders for this study will be 1 and 10 feet.

To allow for the wider shoulder to be on the side of the roadway with access to emergency egress, the 10-foot shoulder may be located on the left side of the travel lanes depending on the configuration of the lanes. Emergency egress is discussed in following sections of this TM.

2.4 Edge Treatment

A Caltrans Type 60D-shaped barrier would be incorporated into the wall shape on the outer edges of both the left and right shoulders. The Type 60D-shaped barrier would be made an integral part of the vertical walls which separate the traveled way from other areas of the tunnel as seen in Attachment A. The dimensions of the Caltrans Type 60D barrier are shown in Figure 1. The barrier extends 5.75 inches from the vertical wall at its base, and the 4-inch horizontal surface would be included as part of the vertical walls.

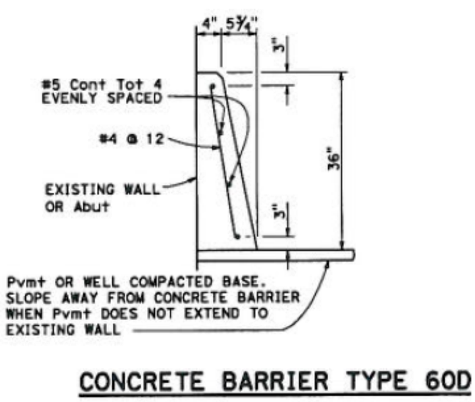


FIGURE 1
Caltrans Type 60D Barrier

2.5 Roadway Cross Slope

A cross slope of 2% is being shown for both the upper and lower roadways. The roadways slope away from the emergency egress areas.

2.6 Vertical Clearance

The vertical clearance is determined by the clear height above the highway grade for traffic. Caltrans HDM Table 309.2A, "Minimum Vertical Clearances," indicates that a freeway (new construction) is required to have a minimum vertical clearance above the travel lanes and shoulders of 16.5 feet. Additionally, the vertical clearance required to signs and minor structures is 18.5 feet for a normal at-grade freeway. For the purposes of developing the tunnel cross sections for this study, a vertical clearance of 15.5 feet, with a reduced clearance of 14 feet 8 inches for a width of 11 inches above the shoulder on the upper deck, was used per Metro's direction (refer to Attachment A).

If a maximum posted vehicle height of 14 feet is used for the tunnels, this would provide a clearance of about 1.5 feet between the top of a vehicle and the lowest of any appurtenance installed in the tunnel based on the configuration proposed. Outside of this vertical clearance envelope, an additional 2 feet (in the vertical direction above the clearance envelope) have been provided for signage, lighting, and other tunnel systems equipment.

2.7 Emergency Egress

A safe evacuation route for vehicle occupants is essential in the case of an emergency inside a freeway tunnel. The NFPA states that emergency exits must be provided throughout the tunnel to minimize the exposure of the evacuating vehicle occupants to an untenable environment (NFPA 502, 2011). In addition, it must be recognized that some of the vehicle occupants may be disabled, requiring special provisions. The requirements that form the basis for the egress concepts of the Freeway Tunnel Alternative are based on both NFPA (2011) and the discussions with the City of Los Angeles fire marshal and other stakeholders (CH2M HILL 2013), and they include the following:

- Spacing between the emergency exits shall not exceed 656 feet.
- The egress passageways shall be separated from the tunnel with a minimum of a 2-hour fire-rated door/enclosure.
- The minimum clear width of the egress passageway must be 3.6 feet.
- The minimum headroom in the egress passageways shall be 7.5 feet, with projections not less than 6.67 feet above the floor.
- Where the portals of the tunnel are below surface grade, the surface grade shall be made accessible by a stair, ramp, or elevator.

The design team has taken these requirements into consideration to develop the concept for emergency egress in both the twin-bore and single-bore freeway tunnel options.

2.8 Area for Tunnel Systems Equipment and Ventilation

The design of the tunnel section has been checked to ensure that there is sufficient area in the cross section for fire life safety, ventilation, and tunnel systems equipment. There is a minimum of 2 feet of clear space above the clearance envelope over the travel lanes for tunnel lighting, variable message signs, and other necessary utilities. Additionally, the ventilation design was checked to ensure enough area is provided for the exhaust air duct and the rooms for tunnel equipment; No jet fans are necessary in the bored section of this tunnel.

2.9 Internal Structure

The internal structure that would make up the road decks and the walls which separate the travel lanes from other areas of the tunnels (emergency egress walkways and utility corridors) will also add to the space requirements of the tunnel. The thicknesses of the vertical and horizontal members which make up the internal structure are somewhat dependent of the final diameter of the tunnel, making it an iterative process. At this preliminary stage of the design, the following thickness allowances were used as a basis for the space requirements:

- Roadway slab thickness: 26 inches
- Lower wall thickness: 16 inches

- Upper wall thickness: 10 inches

Additional details of the internal structure can be found in *Preliminary Design Concepts for the Freeway Internal Structure* (JA 2014c).

3 Freeway Tunnel Cross Section

3.1 Development Process

The requirements discussed in the previous section were considered when determining the cross section of the freeway tunnel. The activity of fitting together the tunnel cross section in the most efficient way possible is an iterative process – the task was to fit the components together in a way that met the requirements and fit in the smallest diameter possible. The geometry that resulted from this process has sufficient space for the tunnel systems equipment and ventilation and therefore a cross section was established. Refer to Attachment A for the cross sections of both the twin and single-bore freeway tunnel options.

3.2 Tunnel Diameter

The components of the freeway tunnel fit within a diameter that is 52.5 feet; these are the design limits of the cross section (refer to Attachment A). To account for deviations from line and grade during TBM excavation, a tolerance of 6 inches was allowed, and therefore the segmental lining is shown with an inside diameter (ID) of 53.5 feet (6-inch tolerance on the radius results in 1-foot tolerance on the diameter).

The outside diameter (OD) of the tunnel segmental lining is dependent on the ID and the thickness of the segmental lining. The thickness of the segmental lining shown at this preliminary phase is 30 inches (JA, 2014a), which results in a lining OD of 58.5 feet.

While the freeway tunnel cross section has an ID and OD of the segmental lining of 53.5 and 58.5 feet, respectively, the diameter of the TBM used to excavate a tunnel of this size would be larger. The TBM is generally larger than the OD of the lining to account for the thickness of the TBM shield, shield clearance/gap, and overcut. For example, the SR-99 tunnel currently under construction in Seattle, Washington, the excavated diameter is about 18 inches larger than the OD of the lining. Conceptually, a TBM with an excavated diameter of approximately 60 feet (i.e., 58.5 feet + 18 inches) could be used to excavate the freeway tunnels. This is the diameter being used for this phase of the study and should be optimized where possible in future stages.

3.3 Tunnel Bore Spacing

The two tunnel bores are separated by approximately one tunnel diameter for the majority of the alignment, which is approximately 60 feet. For reasons to do with the roadway development, the bores are separated by approximately 70 feet as they approach the north TBM launch portal. This spacing should be revisited in subsequent phases of the design; if the spacing could be reduced, the length of the cross passages would be reduced.

4 Freeway Tunnel Emergency Vehicle Cross Passages

4.1 Background

Emergency vehicle cross passages are being recommend to be included with the twin-bore variation; the following section does not apply to the single-bore variation. There are no regulations in the Caltrans Highway Design Manual or set by National Fire Protection Association (NFPA) or any other agency requiring cross passages for emergency vehicles. However, it was decided in a coordination meeting with the City of Los Angeles fire marshal and other stakeholders that emergency vehicle cross passages would be positioned along the twin-bore variation at a spacing of approximately 3,000 feet for this study. (CH2M HILL 2013)

The emergency vehicle cross passages could reduce the amount of time it takes first responders to arrive at the location of an incident in the event of an emergency or unplanned event in a tunnel of this length depending on the location of the incident with respect to the portals and cross passages. Vehicular cross passages would provide first responders with another option to reach an incident in addition to the shoulders of the travel lanes, and also provide a means for vehicles behind the incident to exit the tunnel. The need for these cross passages will continue to be discussed in future phases of this project.

4.2 Geometry and Configuration

The size of the cross passages depends on the type of vehicles expected to use them. The cross passages are primarily intended for emergency response vehicles; however, in a meeting with the fire marshal it was determined that the cross passages should be able to accommodate vehicles traveling in the tunnel in the event of an emergency, which include larger trucks (CH2M HILL 2013).

Angled cross passages have been proposed to accommodate the turning radius of a larger truck. An iterative process was used to determine the width of the clearance envelope and the angle of the cross passage with respect to the bored tunnel to optimize the design. The emergency vehicle cross passages are conceptually shown to have a clearance envelope that is 20 feet wide and 14.5 feet high, angled 50 degrees from the bored tunnels (Mitry 2013). The angle of the cross passages should be revisited when better ground information is available, and the operational need should analyzed to optimize the safe and economic construction of the cross passages in future phases of this study. Refer to Attachment A for drawings of the cross passages in cross section.

As there are two levels of traffic, there would be a separate cross passage for each of the upper and lower levels; one cross passage excavation to accommodate both levels would require too large of an excavation. Refer to Figure 2 for a schematic of the cross passage configuration used in this study. In future phases of this project, if the cross passages are carried forward, coordination will be needed to determine how the cross passages will work together with the emergency egress passageways when they are in use. .

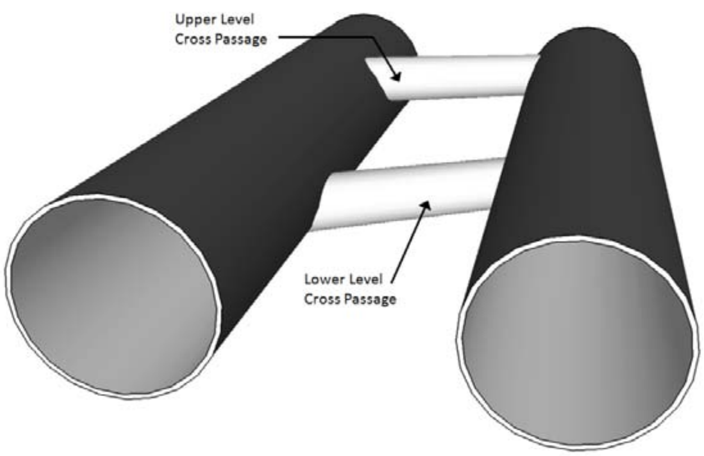


FIGURE 2
3D Model of Emergency Vehicle Cross Passage Configuration

5 LRT Tunnel Geometric Requirements

5.1 Bored Tunnels

Metro has published standards and guidelines for LRT circular (bored) tunnels (Metro, 2012). In addition, the NFPA 130 code for passenger rail systems (2010) was also consulted in determining the cross section of these tunnels. The Metro Rail Design Criteria (2012) specifies that the ID of an LRT tunnel is 18.83 feet and presents figures showing clearance envelopes. The drawings in Attachment B show the conceptual LRT Tunnel cross section. Currently, the OD of the final lining is shown to be 20.5 feet based on a segmental lining that is 10 inches thick (refer to JA, 2014b for details on the segmental lining of the LRT tunnel). The TBM that would excavate a tunnel with a final lining of this size would be approximately 12 to 14 inches greater in diameter than the OD of the final lining, making it just over 21.5 feet in diameter.

5.2 Emergency Egress

A walkway running the entire longitudinal length of the tunnel is necessary to provide passengers access to egress locations in the event of an emergency. NFPA 130 (2012) requires that the minimum unobstructed walkway be at least 30 inches wide and 80 inches high. The Fire/Life Safety Criteria of the Metro Rail Design Criteria (2012) note that a 30-inch clear width is acceptable, and a 36-inch clear width is preferable. The proposed clearance envelope for the LRT walkway meets these requirements (refer to Attachment B).

Cross passages are proposed along the bored tunnel portion of the LRT Alternative as emergency exits at a spacing not to exceed 800 feet, with a spacing of 750 feet being preferred. The clearance envelope inside the cross passages is 6.5 feet wide and 8 feet high, with a minimum height of 7 feet (Metro Rail Design Criteria, 2012). These requirements exceed the NFPA requirements for cross passage spacing and clearance envelope size. Details regarding the excavation and support of these cross passages are in *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014b).

5.3 Bored Tunnel Separation

The separation of the LRT tunnels varies from approximately 14 to 21 feet along the alignment. The two bores are separated by approximately one tunnel diameter for the majority of the alignment, which makes the distance between the centerlines of each tunnel bore approximately 42 feet. The separation is reduced near to the four underground stations where the bores taper into the stations (which is dictated by the platform width); the distance between the centerlines of the bores at these locations is approximately 34.5 feet. In addition, the bored tunnels taper together at the north portal, where the tunnels transition from bored tunnel to cut-and-cover tunnel.

6 References

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Jacobs Associates (JA). 2014b. *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings*. Prepared for Metro..

Jacobs Associates (JA). 2014c. *Preliminary Design Concepts for the Freeway Internal Structure*. Prepared for Metro..

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National Fire Protection Association (NFPA), 2011. NFPA 502: Standard for Road Tunnels, Bridges, and Other Limited Access Highways, 2011 Edition.

National Fire Protection Association (NFPA), 2009. NFPA 101: Life Safety Code, 2009 Edition.

7 Revision Log

Revision 0	January 23, 2014	Internal Review
Revision 1	February 6, 2014	Internal Review
Revision 2	February 18, 2014	Metro/Caltrans Review
Revision 3	June 3, 2014	Metro/Caltrans Review
Revision 4	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

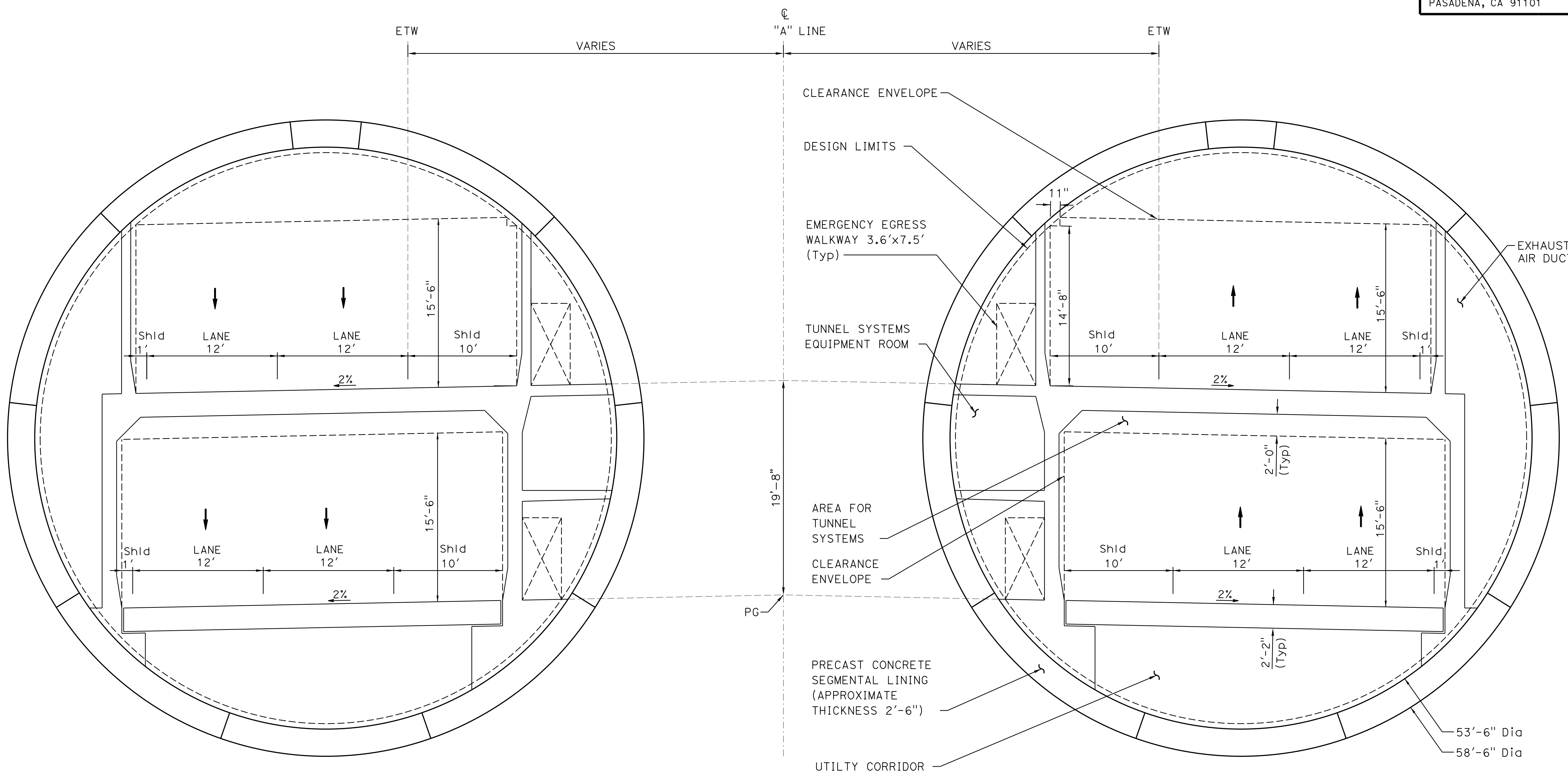
Freeway Tunnel Drawings

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



SOUTHBOUND SECTION

NORTHBOUND SECTION

S2.01

DESIGNED BY	M. TORSIELLO	DATE	12-19-2013
DRAWN BY	J. TOLES	DATE	12-19-2013
CHECKED BY	S. KLEIN	DATE	2-7-2014
APPROVED	S. DUBNEWYCH	DATE	2-7-2014

X
PROJECT ENGINEER

SR 710 NORTH STUDY

TUNNEL-TYPICAL SECTION

BRIDGE NO. TBD
UNIT:
SCALE: 1"-10'
PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT
SIGN OFF DATE

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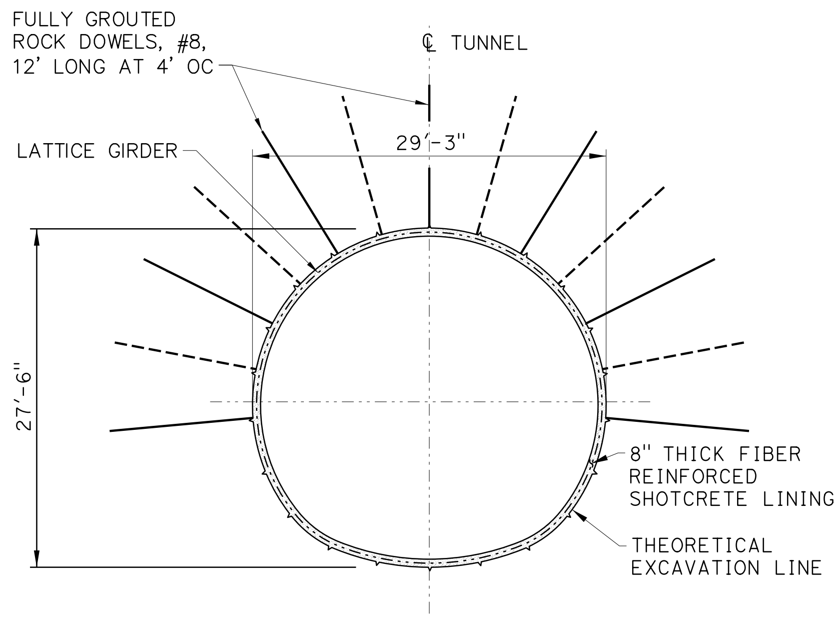
PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



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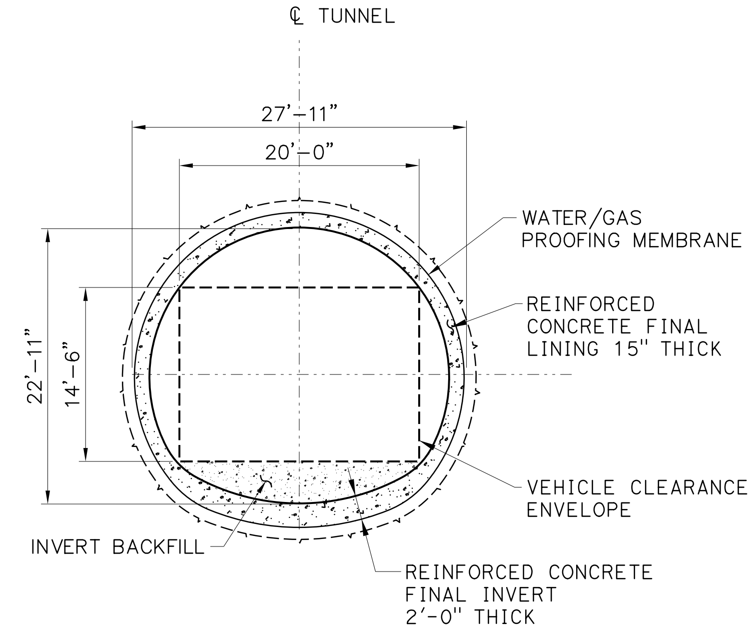


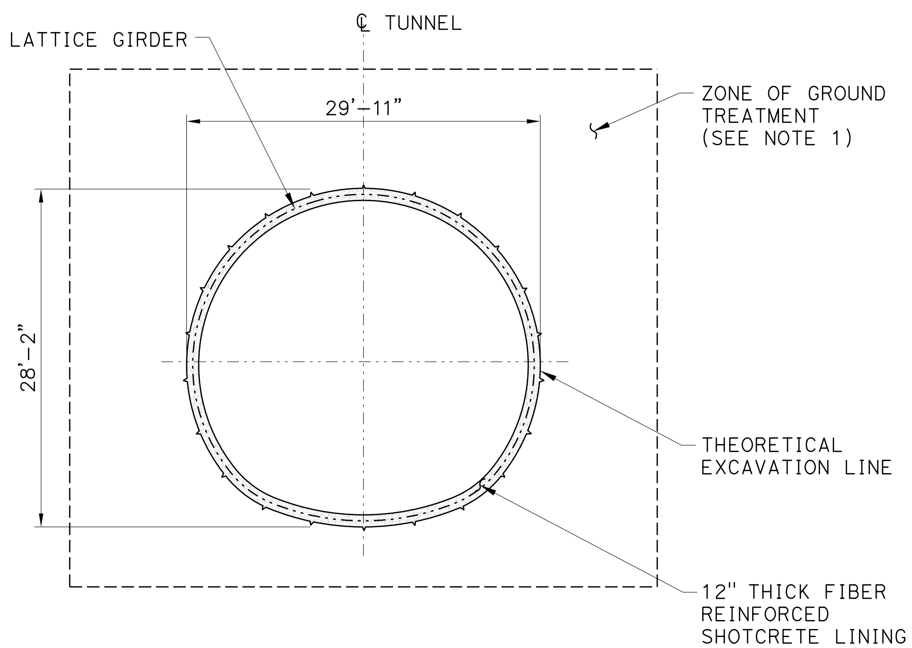
FINAL LINING SECTION

NTS

CROSS PASSAGE IN ROCK

NOTES

1. GROUND TREATMENT METHODS INCLUDE, BUT ARE NOT LIMITED TO, PERMEATION GROUTING, CHEMICAL GROUTING, OR GROUND FREEZING. TREATED GROUND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 400 PSI AT 28 DAYS.



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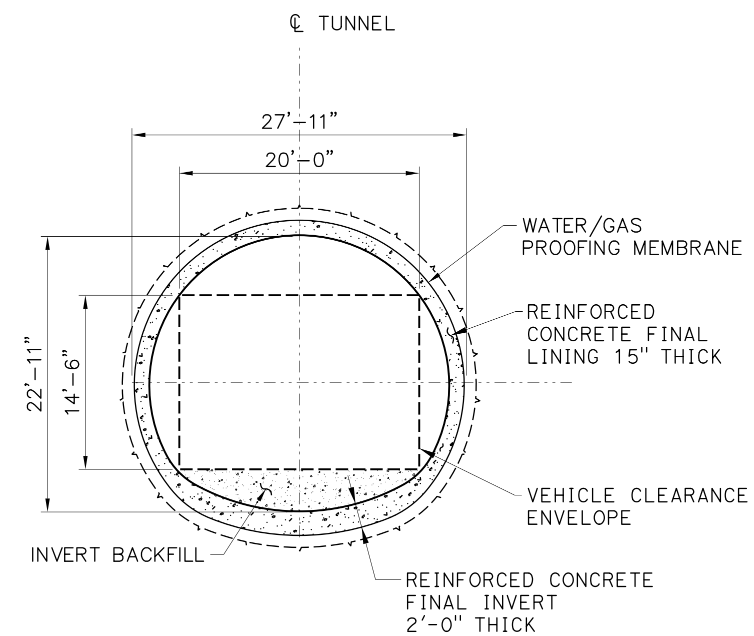

FINAL LINING SECTION

NTS

CROSS PASSAGE IN SOIL

S3.01

X
DESIGN OVERSIGHT
X
SIGN OFF DATE

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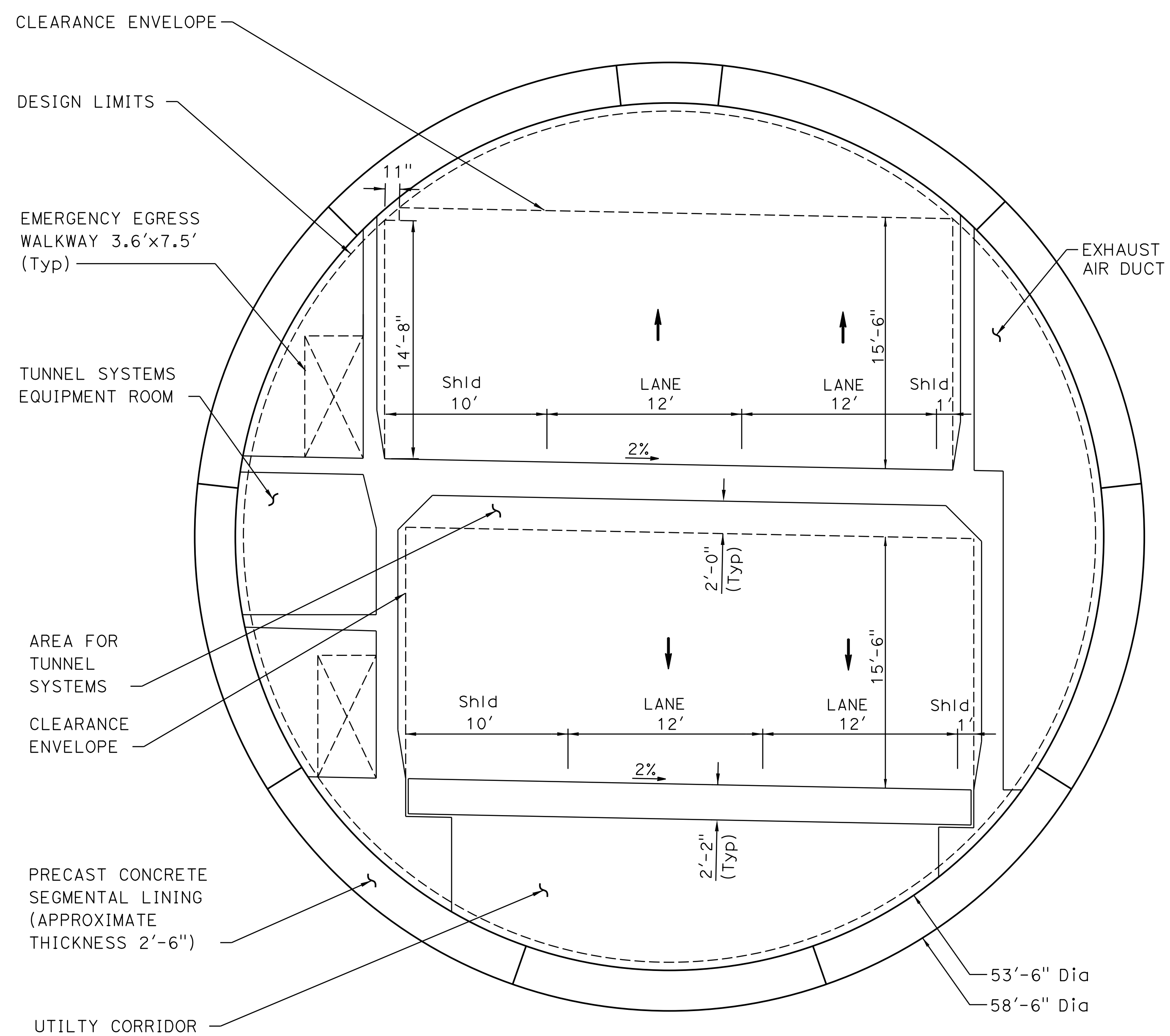
DRAWN BY	J. TOLES	DATE 7-23-2013	PROJECT ENGINEER	CROSS PASSAGE 1 OF 7
CHECKED BY	S. KLEIN	DATE 2-7-2014		BRIDGE NO. TBD / UNIT:
APPROVED	S. DUBNEWYCH	DATE 2-7-2014		SCALE: AS SHOWN / PROJECT NUMBER & PHASE:

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



TYPICAL SECTION

S2.01

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DRAWN BY	J. TOLES	DATE	12-19-2013
CHECKED BY	S. KLEIN	DATE	2-7-2014
APPROVED	S. DUBNEWYCH	DATE	2-7-2014

X
PROJECT ENGINEER

SR 710 NORTH STUDY

TUNNEL-TYPICAL SECTION

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SCALE: 1"-10'	PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

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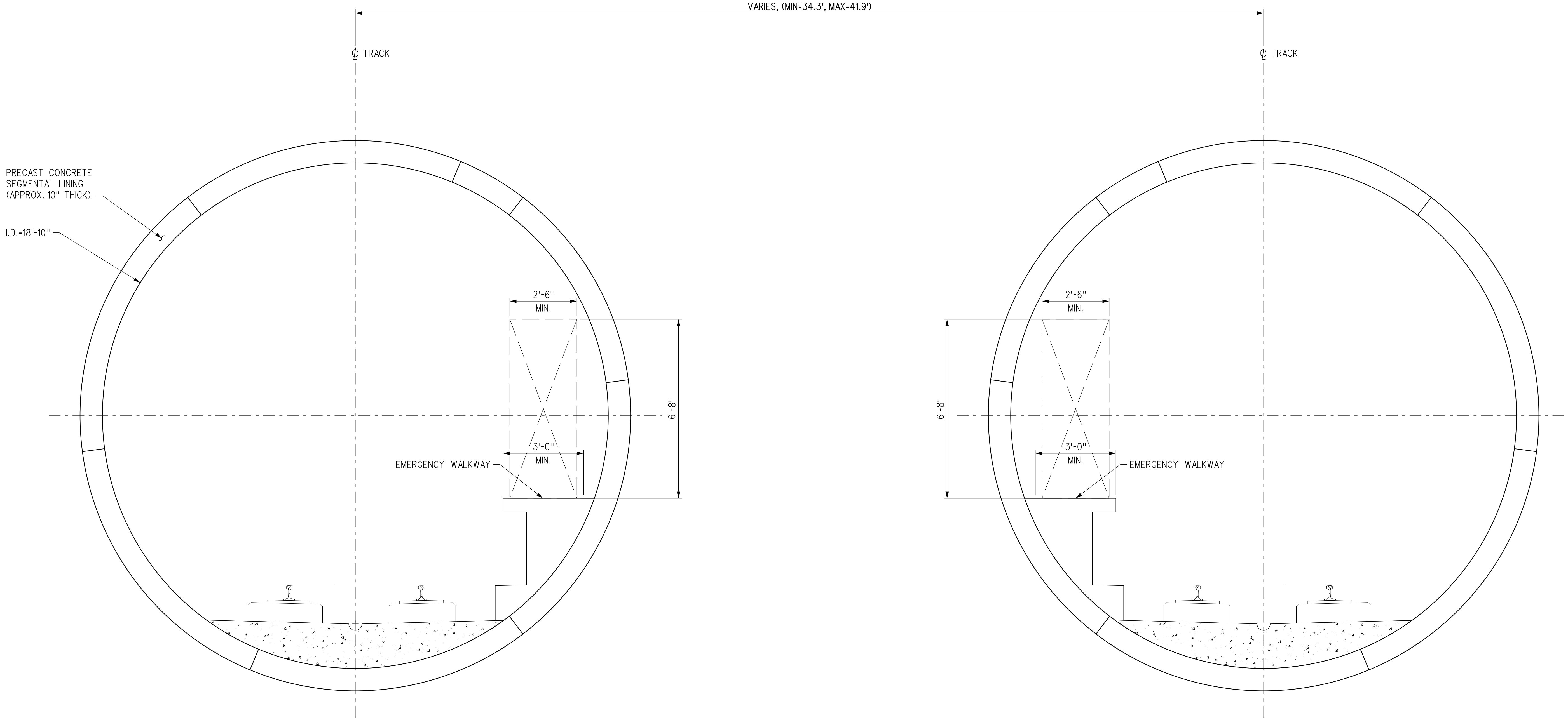
Attachment B

LRT Tunnel Drawings

Metro®

Caltrans



TYPICAL SECTION – BORED TUNNELS

NOTES:

1. METRO RAIL DESIGN CRITERIA AND NFPA 130 USED AS REFERENCE FOR TYPICAL CROSS SECTIONS AND CLEARANCES.

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PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION
-	2/7/14						METRO COMMENTS

DESIGNED BY M. TORSIELLO
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TYPICAL BORED TUNNEL SECTION
SHEET 1 OF 1

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SHEET NO

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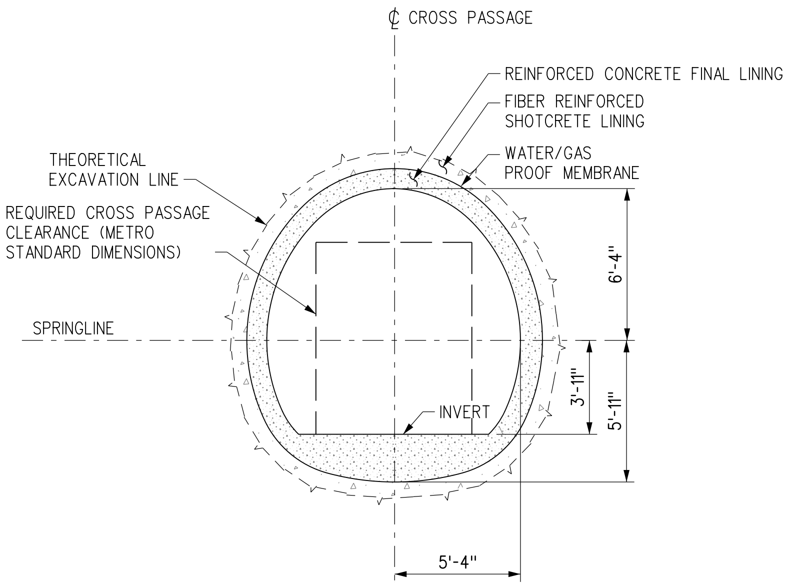
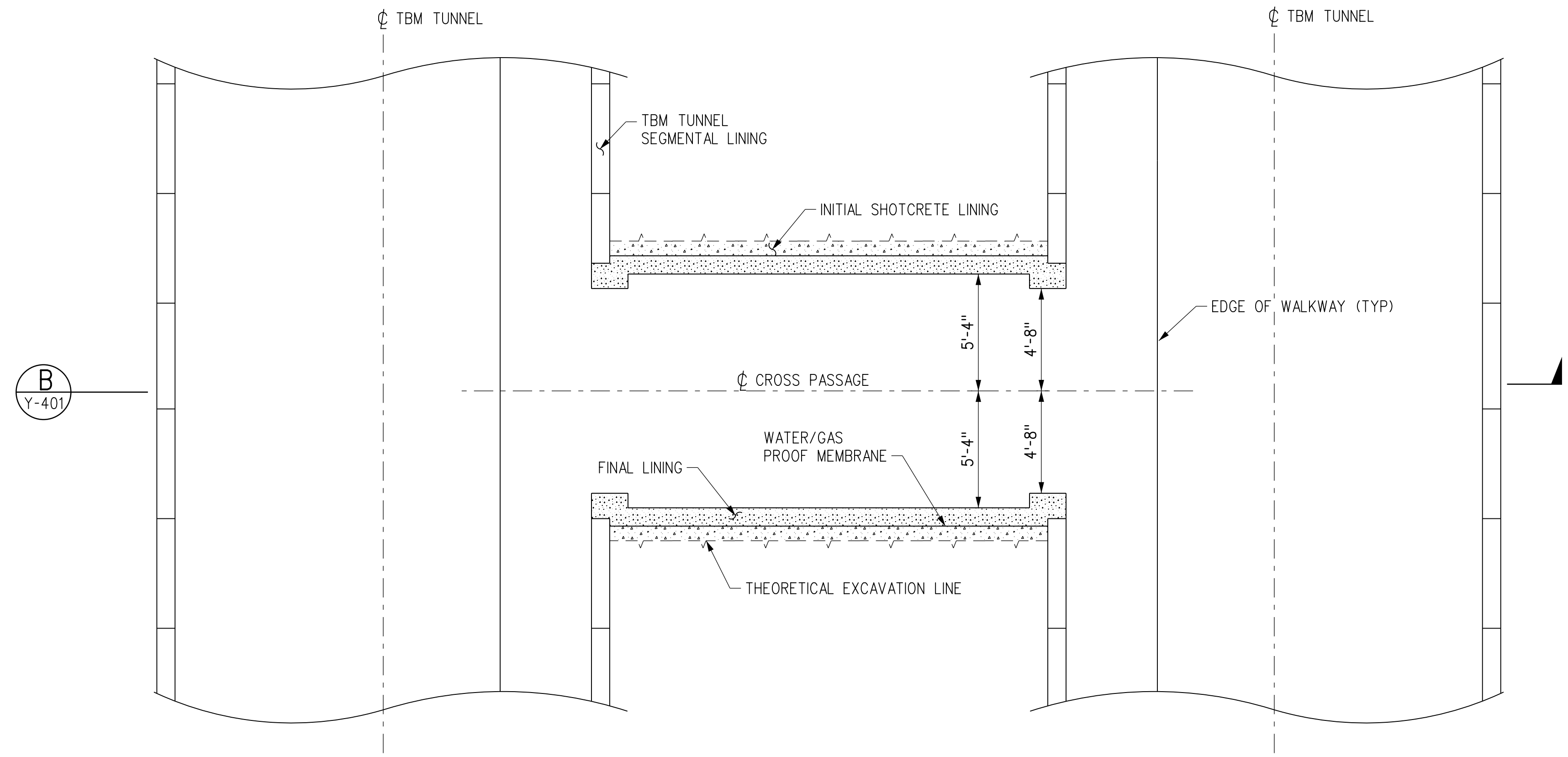
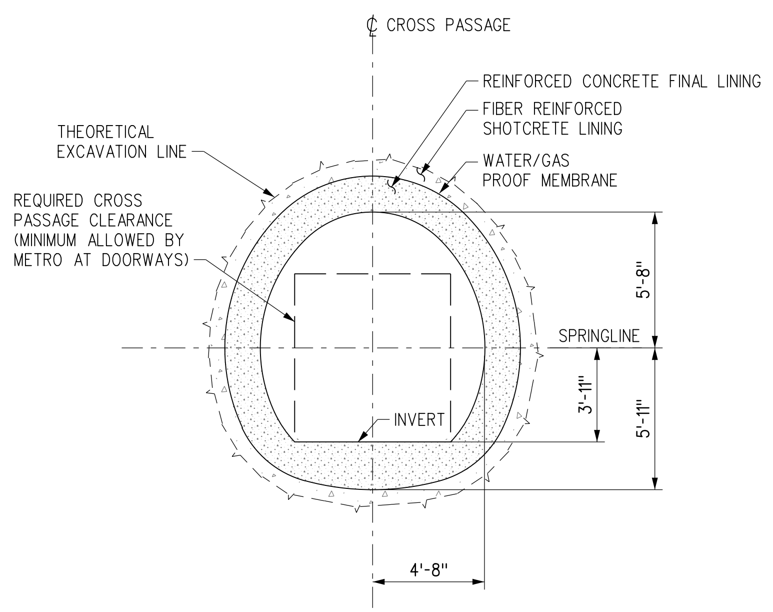


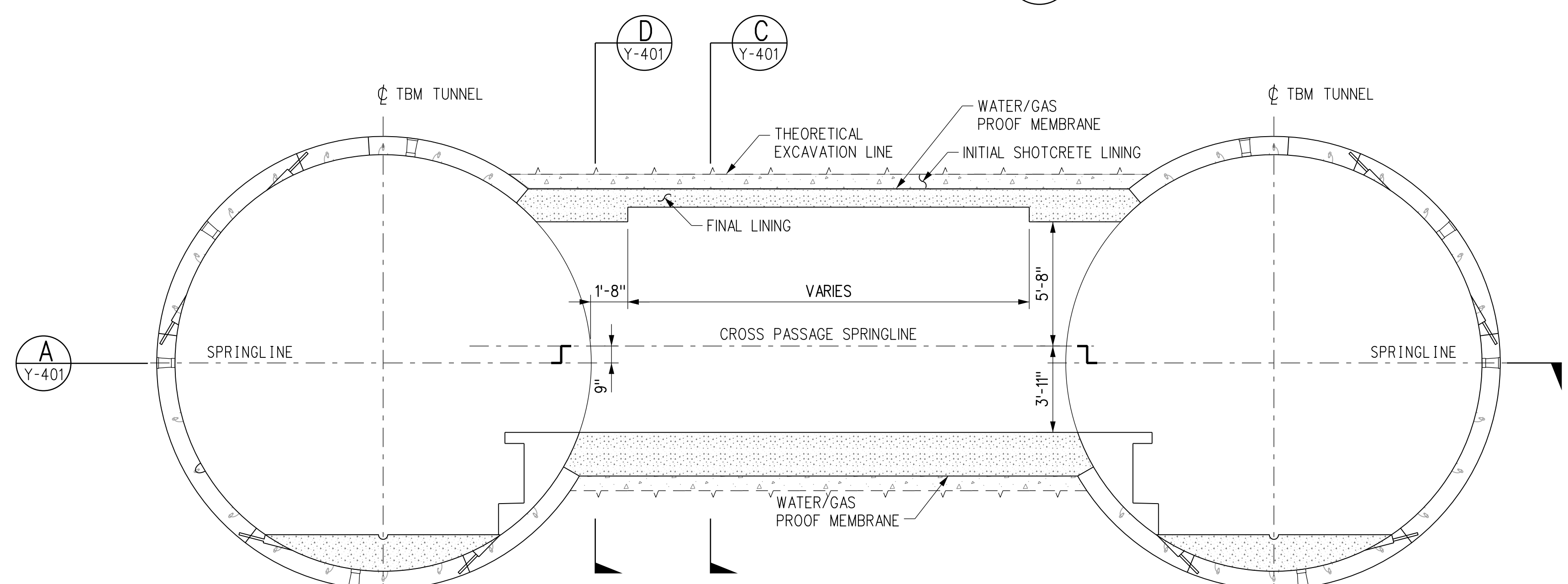
SECTION C Y-401



PLAN A Y-401



SECTION D Y-401



SECTION B Y-401

NOTES:

1. METRO RAIL DESIGN CRITERIA USED AS REFERENCE FOR CROSS PASSAGE CLEARANCE REQUIREMENTS.

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PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION
-	2/7/14						METRO COMMENTS

DESIGNED BY Y. SUN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro
LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE
PLANS AND SECTIONS

CONTRACT NO
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REV
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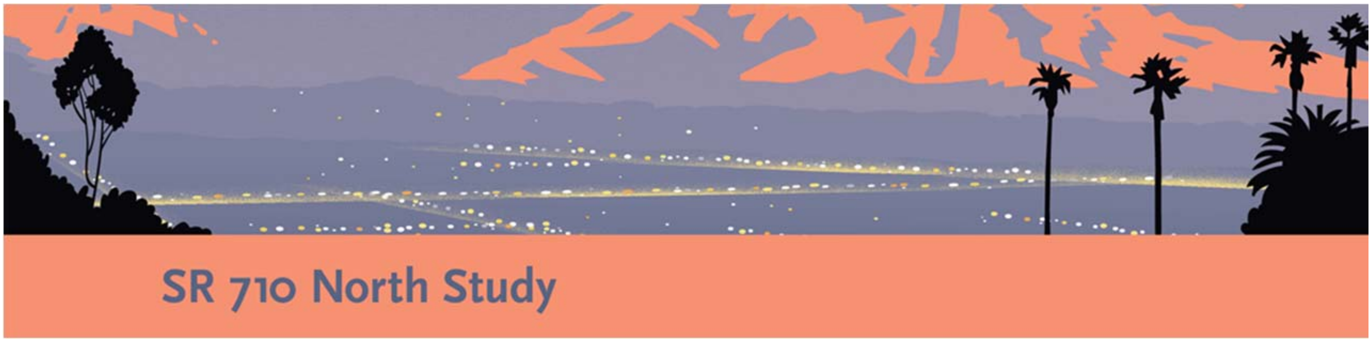
SHEET NO

Appendix B
TM- 2 Tunnel Ground Characterization



TECHNICAL MEMORANDUM 2

Tunnel Ground Characterization

PREPARED FOR: Metro

COPY TO: Caltrans

PREPARED BY: Jacobs Associates/CH2M HILL

DATE: August 22, 2014

PROJECT NUMBER: 428908

1. Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) presents a preliminary assessment of the ground conditions along the bored portion of the proposed alignment of the freeway and LRT tunnels, as well as the cross passages associated with these tunnels. The findings of this TM have been used in the development of the preliminary design evaluations for the bored tunnel and cross passage excavation and support.

This TM describes the results of evaluations performed to characterize ground conditions for the tunnel alternatives, including the determination of tunnel reaches (TRs), and identification of ground classes (GCs). Ground characterization involves evaluating available geologic data, assessing soil/rock mass properties, identifying distinct Rock Mass Types (RMTs), estimating anticipated ground behaviors along the proposed tunnel alignments, and developing geotechnical parameters for preliminary design evaluations. Determination of TRs involves dividing each of the tunnel alignments into a number of reaches of similar conditions based on the anticipated ground conditions and potential ground behavior. RMTs are determined based on the lithology, fracture frequency and condition, weathering, and strength of the rock formations. The identification of ground classes involves grouping the RMTs into several GCs based on similar ground behaviors anticipated during tunnel excavation. It is assumed that the running tunnels for each alternative are excavated with pressurized-face tunnel boring machines (TBMs). Excavation and initial support of the cross passages are anticipated to be constructed using the Sequential Excavation Method (SEM).

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Geologic Conditions

The project area encompasses portions of the San Gabriel Valley, the southern San Rafael Hills, the Elysian Hills, and the Repetto Hills (CH2M HILL, 2014). The Repetto Hills and San Rafael Hills in the western part of the project area are characterized by small- and medium-sized rounded hills and intervening valleys. The San Gabriel Valley, which encompasses the eastern part of the project area, is essentially a flat, gently south-sloping surface. A major geomorphic feature is Arroyo Seco, which is a steep-walled, flat-floored ravine about 600 to 1,000 feet wide and 50 feet deep. The project area is principally underlain by Quaternary-age alluvium, Tertiary-age sedimentary rocks, and Mesozoic-age crystalline igneous and metamorphic rocks. The sedimentary rocks in the project area consist of claystone, siltstone, mudstone, sandstone, conglomerate, and breccia.

2.1 Sources of Geologic Data

The Preliminary Geotechnical Report for the SR-710 North Study (CH2M HILL, 2014) was reviewed and used for the development of this technical memorandum. Some additional information from previous studies (CH2M HILL, 2010) were also used as a reference.

2.2 Primary Geologic Units

The geologic units along the proposed alignments for the Freeway Tunnel and LRT Alternatives are shown in Figure 1 and Figure 2, respectively. These geologic units/formations include both soft ground and rock formations consisting of artificial fill, alluvium, Fernando Formation, Puente Formation, Topanga Formation, and basement complex rocks (Wilson Quartz Diorite). Terms and definitions used to describe the characteristics of the rock formations are presented in Tables 1 through 5.

Artificial Fill. Along the tunnel alignments, artificial fill ranging approximately from 0 to 40 feet thick overlies the alluvium at the southern end of the alignments. The bored tunnels of the Freeway Tunnel and LRT Alternatives are located generally below the fill. The fill consists of heterogeneous mixtures of gravel, sand, silt, and clay, with variable amounts of debris. Standard Penetration Test (SPT) N values are 8 and 11 based on two tests from Boring RC-13-005. Blow counts per foot using Standard California sampler are 15 and 26 based on two tests from the same boring. Table 6 summarizes the laboratory test results of undrained shear strength and the plasticity and liquidity indices for the fine-grained soils of the fill. Based on the results of the triaxial tests, the consistency of the fine-grained soils is stiff to hard.

Alluvium. The alluvium deposits consist of interbedded lenses and/or discontinuous layers of fine-grained soils (clay and silt) and coarse-grained materials (sand and gravel) that include a wide range of soil types. For example, alluvial soils retrieved from the borings include GP, GW, SP, SW, SM, SW-SM, SM-SC, ML, MH, CL-ML, CL, and CH, as classified based on Unified Soil Classification System (USCS). Hard to very hard cobble-size rock clasts are common locally within the alluvium, and some hard to very hard boulders may be scattered locally throughout the unit. Table 6 summarizes the laboratory test results of undrained shear strength, plasticity index, and liquid limit for the fine-grained soils of the alluvium deposits. Triaxial tests indicate undrained shear strength in the range of 1.2 to 6.7 ksf for the fine-grained soils. Histograms summarizing the SPT N values of the alluvium within the tunnel zone (i.e., between one tunnel diameter above the tunnel crown and one tunnel diameter below the tunnel invert) are shown in Figure 3. The SPT N values range from 0 to 50 blows for 1/2 inch for the Freeway Tunnel Alternative, and from 9 to 200 blows for 11 inches for the LRT Alternative. These results generally indicate the fine-grained soils are stiff to hard, and the coarse-grained soils are very loose to very dense. Some of the boring logs noted rock fragments, gravels, and/or cobbles for the SPT refusals. Laboratory test results for grain size distribution are shown in Figure 4 for the alluvium to be encountered along both the Freeway and LRT alternative alignments.

Fernando Formation. Along the Freeway Tunnel and LRT Alternatives, the Pliocene-age Fernando Formation consists primarily of weak, massive, marine claystone, and siltstone. Scattered, hard concretions and very thin to thin hard layers, occur within the Fernando Formation Siltstone Member. Figure 5 shows core photographs of the moderately to slightly weathered Fernando Formation taken from the exploration program. Average shear wave velocity measured in this formation ranges from 880 to 2,800 feet per second. Results of laboratory testing are summarized in Table 7 for the Fernando Formation. The Fernando Formation exhibits an unconfined compressive strength (UCS) in the range of approximately 50 to 500 psi. The RQD values ranges from 0 to 100. The rock mass is generally slightly to very slightly fractured (i.e., fracture spacing between 1 and 10 feet). Laboratory test results for grain size distribution of the Fernando Formation are shown in Figure 6.

Puente Formation. The marine rocks of the late Miocene Puente Formation expected to be encountered along the tunnel alignments consists predominantly of the siltstone unit. This unit is a thinly bedded to laminated siltstones with medium to thick interbeds to laminations of fine-grained sandstone. The rocks generally are weak with locally strongly cemented interbeds and concretions. The observed cemented zones and concretions were generally strong and can be hard to very hard. Figure 7 shows core photographs of the moderately to slightly weathered Puente Formation taken from the exploration program. Average shear wave velocity measured in this formation ranges from 900 to 3,360 feet per second. Results of laboratory testing are summarized in Table 7 for the Puente Formation. The Puente Formation exhibits an unconfined compressive strength (UCS) in the range of approximately 40 to 500 psi. The RQD values ranges from 0 to 100. The rock mass is generally intensely fractured to very slightly fractured (i.e., fracture spacing between 1 inch and 10 feet). Laboratory test results for grain size distribution are shown in Figure 8 for the siltstone member of the Puente Formation.

Topanga Formation. The middle-Miocene-age Topanga Formation includes a wide variety of rock types ranging from coarse-grained rocks such as breccia, conglomerate, and sandstone to fine-grained sandstone and siltstone with minor claystone. The formation consists predominantly of the siltstone unit south of the Raymond fault. This unit consists of thinly bedded to laminated and fissile siltstones and shales, with fine- to coarse-grained sandstone interbeds. Localized, strongly cemented layers and/or concretions were encountered through the formation. The cemented zones, layers and concretions are generally strong and can be hard to very hard. Figure 9 shows photos of the cores of moderately to slightly weathered Topanga Formation from the exploration program. Average shear wave velocity measured in this formation ranges from 1,170 to 5,400 feet per second. Results of laboratory testing are summarized in Table 7 for the Topanga Formation. The Topanga Formation exhibits an unconfined compressive strength (UCS) in the range of approximately 10 to 5,000 psi. The RQD values ranges from 7 to 100. The rock mass is generally intensely fractured to very slightly fractured (i.e., fracture spacing between 1 inch and 10 feet). Laboratory test results for grain size distribution are shown in Figure 10 for the siltstone and conglomerate members of the Topanga Formation.

Basement Complex Rocks. Basement Complex Rocks in the project area include several types of igneous and metamorphic rocks such as: diorite, monzonite, quartz diorite, quartz monzonite, and gneissic diorite. The rock mineralogy is primarily plagioclase feldspars with quartz, hornblende, and biotite. Figure 11 shows examples of Basement Complex Rocks in a moderately to slightly weathered state. Average shear wave velocity measured in this formation ranges from 1,450 to 6,700 feet per second. Results of laboratory testing are summarized in Table 7. The Basement Complex Rocks exhibits an unconfined compressive strength (UCS) in the range of approximately 35 to 1,600 psi. The RQD values ranges from 0 to 76. The rock mass is generally intensely fractured to very slightly fractured (i.e., fracture spacing between 1 inch and 10 feet).

Histograms summarizing the laboratory results of unconfined compressive strength (UCS) and point load index (PLI) tests for the various rock formations are shown in Figures 12 and 13, respectively. It should be noted there are limited numbers of UCS and PLI tests. Significantly more UCS and PLI tests, as well as other tests in general, are required to obtain a representative distribution of rock strength and other geotechnical parameters for each of the rock formations.

Histograms summarizing the Rock Quality Designation (RQD) values of the rocks within the tunnel zone are shown in Figures 14 and 15 for the freeway and LRT tunnels, respectively. It should be noted that these histograms were prepared based on the limited number of borings and tests available along the tunnel alignments within the tunnel zone; therefore, the distributions shown in these histograms may change as additional data becomes available. It is also noted that RQD values from the borings are based on intact core pieces obtained between natural discontinuities. However, due to the relatively weak nature of the rock formations, a significant portion of the cores included in the RQD calculation do not necessarily meet the “sound core” definition provided in the standard test method for RQD (ASTM D6032), so it could be unconservative to use these RQD values to evaluate the overall rock mass quality. This needs to be taken into consideration when reviewing the RQD histograms. Further geotechnical investigations in the future phases of this study are necessary to better understand the correlation between the RQD values and the anticipated rock mass quality.

2.3 Geologic Structure

According to the Preliminary Geotechnical Report (CH2M HILL, 2014), the geologic strata along the tunnel alignments is deformed into series’ of folds and faults. These folds and faults are a result of ongoing regional tectonic forces which are present in the SR 710 North Study Area and the Los Angeles Basin. South of the Raymond fault, these structures generally trend southeasterly through the Repetto Hills and continue below the flat-lying Quaternary alluvium of the San Gabriel Valley in the vicinity of the tunnels. These folds include the Elysian Park Anticline and the South Pasadena Anticline. Frequent changes in bedding orientation due to folding and faulting are expected at the depths of the Freeway and LRT tunnels. Many of the faults mapped in the SR 710 North Study Area are inactive and intraformational features (meaning the faults offset rocks of the same geologic formation). However, some of these inactive faults juxtapose rocks of different formations as well. In either case, the width of these faults along the tunnel alignments is expected to vary widely, including narrow to wide zones of highly fractured rock and/or clayey gouge.

Several faults are shown on the geologic profiles in Figure 1 and Figure 2. According to the Preliminary Geotechnical Report (CH2M HILL, 2014), Raymond fault is identified as an active fault, and the San Rafael and Eagle Rock faults are identified as potentially active faults. The other faults shown in Figures 1 and 2 are considered inactive faults. Only a few of the borings actually penetrated through the fault zones; therefore, limited information is available on the geotechnical conditions, and exact widths and locations of the fault zones. However, the horizontal zone of uncertainty for the location of each active and potentially fault has been estimated in the preliminary fault investigation conducted for the SR 710 North Study (CH2M HILL, 2014). A combined zone of uncertainty of about 240 feet is estimated for the three Raymond fault strands anticipated at the freeway and LRT tunnel depths. The tunnels could intersect additional fault strands for an additional 200 feet north of the main fault zone, but it is unlikely that the zone of active faulting would extend that far north. The San Rafael fault zone occurs on the north side of Raymond Hill and separates basement complex rocks from the Topanga Formation. The fault zone is mapped as having a main strand and two potential secondary strands. The horizontal zone of uncertainty for each of the fault strands ranges from 75 to 120 feet and 100 to 260 feet at the freeway and LRT tunnel depths, respectively. The Eagle Rock fault is located approximately 2,000 feet south of the

San Rafael fault. The fault would not cross the LRT tunnel alignment based on interpretation discussed in the Preliminary Geotechnical Report (CH2M HILL, 2014). A horizontal zone of uncertainty approximately 85 feet wide is estimated for the Eagle Rock fault at Freeway tunnel depth. Additional geotechnical investigations would be required to further refine the locations and widths of the fault zones. Refer to the Fault Rupture Evaluation and preliminary fault investigation for the SR 710 North Study (Appendices E and G in CH2M HILL, 2014) for additional details and estimates of earthquake magnitude, and fault rupture displacement and width.

2.4 Groundwater and Rock Mass Permeability

The estimated groundwater levels along the freeway and LRT tunnels are shown in Figure 1 and Figure 2, respectively. The depth to groundwater ranges from less than 10 feet to approximately 175 feet below ground surface along the freeway tunnel alignment, resulting in groundwater levels up to approximately 150 feet above the Freeway tunnel crown. For the LRT alignment, the depth to groundwater ranges from less than 10 feet to approximately 160 feet below ground surface, resulting in groundwater levels up to approximately 70 feet above the tunnel crown. There appears to be a signigicant difference in groundwater levels on either side of the Raymond Fault, sugguesting it may be a groundwater barrier.

The unconsolidated alluvial sediments of the San Gabriel Valley and the Raymond Basins constitute important groundwater basins in Southern California. The groundwater basins mainly include the sand and gravel deposits and have been actively exploited by local communities for the last few decades as a source of groundwater. These deep aquifers are overlain locally by perched groundwater bodies. Based on the map of groundwater basins in the project area (CH2M HILL, 2014), both the freeway and LRT tunnel alignments traverse through the groundwater basins where tunnel excavations are expected to encounter alluvial soils below the groundwater table.

The sedimentary and basement complex rock masses contain water; however, for the purposes of water supply, these units are generally considered to be non-water bearing. The estimated ranges of permeability for each bedrock unit, based on results of the available packer test data, are summarized in Table 8.

For the tunnel reaches below the groundwater table, groundwater is expected to have a significant impact on inflows, tunnel stability, and ground movements during construction. The selected tunnel excavation and support methods need to address and mitigate the potential impact. Refer to *Tunnel Excavation Methods TM* (JA, 2014) for how the groundwater related issues can be addressed during construction.

2.5 Potentially Gassy Conditions

According to the Preliminary Geotechnical Report (CH2M HILL, 2014) there is a low to moderate potential of encountering naturally occurring oil and/or gas, most likely within that Puente Formation, along the subterranean portions of the Freeway Tunnel and LRT Alternatives. Naturally occurring oil and/or gas could also be found within any of the geologic formations within the study area. No oil wells are located in the immediate vicinity of the tunneled alternatives; there are nearby oil wells, but the number and density of the wells are such that they are not expected to have an effect on tunneled alternatives (CH2M HILL, 2014).

3 Characterization of Geologic Units

The characterization of geologic units involves two tasks: identifying and characterizing soil deposits (SDs) and rock mass types (RMTs) with similar mechanical characteristics, and estimating parameters associated with each SD and RMT for tunnel design evaluation. The information derived from the characterization of geologic units is used in Section 4 of this TM for identifying ground behaviors associated with each SD and RMT. Ground classes are then defined based on the anticipated ground behaviors associated with each SD and RMT. For cross passage design purposes, the information of anticipated ground classes will be used for selection of excavation and support methods.

The characterization of geologic units is based on an evaluation of the available geotechnical data as described in Section 2.1. The identification of SDs and RMTs depends on the geologic characteristics and relevant geotechnical parameters along the tunnel alignment, as observed in the boreholes and other investigations. SDs are characterized based on soil classification in terms of the grain-size distribution and Atterberg limits, as well as engineering properties including relative density or consistency, and undrained shear strength. In characterizing the RMTs, the Geological Strength Index (GSI) system (Hoek and Brown, 1974) and the modified Terzaghi's rock mass classification system (Proctor and White, 1968; Deere, et al., 1969) are employed to classify the rock mass conditions for each of the RMTs defined. In characterizing the rocks, these systems take into account of the rock's structural conditions such as discontinuity spacing and blockiness, and for the GSI system, surface conditions of the discontinuities including roughness and weathering condition are also considered. As the rock formations encountered are primarily stratified and tend to form slabs rather than blocks. Their behavior is expected to be controlled largely by the bedding plane weakness and partings. The terminology used in this memo such as "blocky" or "seamy" when referring to the modified Terzaghi's rock mass classification system should be associated with the rock structures of both slabs and blocks.

It is also noted that the rock mass quality assessments and ground characterization can also be carried out using the Rock Mass Rating (RMR) (Bieniawski, 1989) and Q (Barton et al., 1974) rock mass classification systems. These systems may be employed in the future phases of this study to enhance the understanding of rock mass characteristics and associated ground behavior during the tunnel excavations.

3.1 Soil Deposits

The alluvium, which overlies the bedrock, ranges from approximately 0 to 280 feet in thickness along the Freeway Tunnel alignment. Based on Figure 1, approximately 20 percent and 10 percent of the length of the tunnel excavation is expected to encounter alluvial soils and mixed-face conditions (i.e., alluvial soils over bedrock), respectively.

The alluvium ranges from approximately 0 to 300 feet in feet thickness along the LRT alignment. Based on Figure 2, approximately 45 percent and 25 percent of the length of the tunnel excavation is expected to encounter alluvial soil deposits and mixed-face conditions, respectively.

The alluvial soils generally increase in strength with depth. The consistency of the fine-grained soil encountered in the borings within the Freeway and LRT tunnel zones typically ranged from stiff to hard; while the relative density of the coarse-grained materials encountered ranged from very loose to very dense, but are typically dense to very dense. Although a detailed study of the characteristics of the boulders within the alluvium has not been performed specifically for this project at the current design stage, information from past local tunneling projects provide an indication on the size and strength of boulders that may be encountered during tunneling. The maximum dimension (i.e., size) of strong to extremely strong boulders (see Table 1) is probably about 3 to 5 feet based on descriptions in geotechnical baseline reports (GBRs) from local tunneling projects that include the Regional Connector Transit Corridor (The Connector Partnership, 2012), Eastside LRT (Eastside LRT Partners, 2002), and Northeast Interceptor Sewer (NEIS) projects (LADPW, 2001). On a volume basis, the amount of cobbles and boulders baselined in these past reports ranges from significantly less than one percent to a few percent of materials that make up the alluvium. The characteristics of the cobbles and boulders should be evaluated further in future design phases as additional data becomes available.

Based on the geotechnical information reviewed, lower-bound (LB) and mean engineering properties of the alluvium recommended for tunnel design are summarized in Table 9.

3.2 Rock Formations

As discussed in Section 2, four rock formations are expected to be encountered along the tunnel alignments. Due to large variations in rock mass quality, each of the rock formations exhibits quite different characteristics, which may result in significantly different ground behaviors during tunnel excavation. These different ground behaviors usually require different excavation and support measures to stabilize the rock mass, provide safe conditions, and achieve acceptable long-term performance during tunnel operations. The objectives of the ground characterization process are to identify and group together rock mass conditions with similar characteristics and expected behaviors. This facilitates both design and construction so similar excavation and support measures can be developed to handle the range of anticipated ground conditions associated with each group.

In characterizing the rock mass, each of the four rock formations was characterized and subdivided, when pertinent, into RMTs on the basis of rock mass quality in terms of discontinuity spacing and condition, strength, and weathering. Engineering properties including strength and deformation moduli were then developed for each of the RMTs for evaluation of anticipated ground behavior and design of tunnel excavation and support methods.

Potential ground behavior associated with the tunnel excavation in each RMT was evaluated using applicable geomechanical properties and ground and excavation conditions such as in situ stresses, groundwater conditions, excavation dimensions/orientations and sequence, and initial support measures. It should also be noted while the siltstone/claystone members of the Fernando, Puente and Topanga formations are indicated as potentially expansive (CH2M HILL, 2014), the tunnel alignments within these formations are generally anticipated to be below the groundwater level such that the ground may not exhibit significant swelling behavior during tunnel excavation. Additionally, based on past local tunnel experience including NEIS, the Puente Formation was not classified as swelling ground for the purpose of behavior during tunnel excavation. Additional testing from future geotechnical investigations should be performed to verify if the formation has any swelling potential and how it may affect tunnel excavation. A summary of the rock mass types and their associated key characteristics is presented below.

3.2.1 Rock Mass Types. Rock mass types (RMTs) represent rock mass conditions with similar lithology, physical characteristics, and/or mechanical properties. RMTs are the basis for the definition of rock mass characteristics for field identification during construction. They also provide the basis for developing estimates of materials properties for preliminary design evaluations, as well as for identifying ground classes along the tunnel alignment for determination of excavation and support requirements.

The definitions in terms of intact rock strength, fracture and bedding spacing, rock weathering, and slake durability are provided in Tables 1 to 5, respectively.

Fernando Formation (RMT Tf)

Fernando Formation is a fairly uniform, weak rock formation and it can be described by a single RMT, defined herein as RMT Tf. The rock mass is massive, slightly to very slightly fractured, moderately to slightly weathered, and extremely weak to very weak. This formation is considered to be on order between hard soil and very weak rock and is characterized herein as “Massive, Moderately Jointed” to “Very Blocky and Seamy” in terms of the modified Terzaghi’s rock mass classifications (Deere et al., 1969).

In terms of the overstress ratio (defined as the ratio of rock strength to in-situ stress), the Fernando Formation at the depth of the Freeway Tunnel Alternative has a value ranging from 0.3 to 1.6, indicating that the rock has “Minor” to “Small” squeezing potential (Hoek, 2000).

Puente Formation (RMTs Tp-1 and Tp-2)

For tunnel design purposes, the Puente Formation is characterized by two RMTs, Tp-1 and Tp-2. Tp-1 rock mass is very thickly bedded to massive, slightly to very slightly fractured, moderately weathered to fresh, and weak to strong. The GSI for Tp-1 is estimated to range from 45 to 55. RMT Tp-2 is very thinly to moderately bedded, moderately to intensely fractured, moderately weathered, and extremely weak to very weak. The GSI for Tp-2 is estimated to range from 35 to 45. Strongly cemented layers and/or concretions are expected to be locally within Tp-1 and Tp-2. In terms of the modified Terzaghi’s rock mass classifications, RMT Tp-1 is characterized herein as “Massive, Moderately Jointed” to “Moderately Blocky and Seamy”, and RMT Tp-2 is characterized herein as “Very Blocky and Seamy”. In terms of the GSI system RMT Tp-1 and RMT Tp-2 are characterized herein as “Massive” to “Blocky” and “Very Blocky” to “Blocky and Seamy”, respectively.

Topanga Formation (RMTs Tt-1 and Tt-2)

For tunnel design purposes, the Topanga Formation is characterized by two RMTs, Tt-1 and Tt-2. Tt-1 rock mass is thinly to thickly bedded, slightly to very slightly fractured, moderately weathered to fresh, and weak to medium strong. The GSI for Tt-1 is estimated to range from 50 to 60. Tt-2 rock mass is thinly to moderately bedded, moderately to intensely fractured, moderately weathered, and extremely weak to very weak. The GSI for Tt-2 is estimated to range from 40 to 50. Strongly cemented layers and/or concretions are expected to be locally within Tt-1 and Tt-2. In terms of the modified Terzaghi’s rock mass classifications, RMT Tt-1 is characterized herein as “Massive, Moderately Jointed” to “Moderately Blocky and Seamy”, and RMT Tt-2 is characterized herein as “Very Blocky and Seamy”. In terms of the GSI system, RMT Tt-1 and RMT Tt-2 are characterized herein as “Massive” to “Blocky” and “Very Block” to “Blocky and Seamy”, respectively.

Basement Complex Rocks (RMTs Wqd-1 and Wqd-2)

For tunnel design purposes, the Basement Complex Rocks are characterized by two RMTs, Wqd-1 and Wqd-2. Wqd-1 rock mass is of slightly to very slightly fractured, moderately weathered to fresh, and weak to medium strong. The GSI for Wqd-1 is estimated to range from 50 to 60. Wqd-2 rock mass is moderately to intensely fractured, moderately weathered, and extremely weak to very weak. The GSI for Wqd-2 is estimated to range from 30 to 45. In terms of the modified Terzaghi's rock mass classifications, RMT Wqd-1 is characterized herein as "Massive, Moderately Jointed" to "Moderately Blocky and Seamy", and RMT Wqd-2 is characterized herein as "Very Blocky and Seamy" to "Completely Crushed". In terms of the GSI system RMT Wqd-1 and RMT Wqd-2 are characterized herein as "Massive" to "Blocky" and "Very Blocky" to "Disintegrated", respectively.

3.2.2 Rock Mass Parameters. Overall rock mass parameters are controlled by the properties of the intact rock pieces, the presence of discontinuities (i.e. bedding, joints and shears) and the freedom of these pieces to slide and rotate under different stress conditions (Hoek et al., 1995). In addition to the effect due to discontinuities within the rock mass, behaviors of relatively weak rocks, such as those of the Fernando, Puente and Topanga Formations are controlled by the low strength of the rock materials. Rock mass properties also depend on scale; the volume of rock within one diameter of the tunnel excavation will largely control the behavior of the tunnel opening (Marinos et al., 2005). Parameters in terms of strength and stiffness properties associated with each of the RMTs are estimated and discussed below. These parameters will be used for the preliminary tunnel design evaluation and analysis and may be reassessed and updated in the future phases of the project.

Strength properties

Since it is not possible to test the rock mass with representative geologic features at the scale or size of the tunnel for the various RMTs, rock mass strength is estimated using a combination of geologic characterization, laboratory testing, and empirical methods. Geologic characterization is based on observation of cores, core logs, and core photographs. Test results used in the assessment of rock mass strength parameters include uniaxial and triaxial compression tests and field point load tests. Figures 13 and 14 show histograms of the UCS and Point Load Index (PLI) values, respectively, for the various rock formations.

The Geological Strength Index (GSI) system (Hoek et al., 2002), which empirically combines qualitative engineering geology assessments and laboratory test results, was used to estimate rock mass strength for the Puente Formation, Topanga Formation, and Basement Complex Rocks. The system involves the following parameters: GSI ratings; UCS of intact rock; Hoek-Brown parameter, mi (Hoek and Brown, 1997); and disturbance factor, D. The rock mass strength for the Fernando Formation was estimated based on laboratory test results and experiences from local relevant projects, such as the Northeast Interceptor Sewer (NEIS) Project.

Evaluation of the GSI classification parameters was performed on core intervals of about 20 to 60 feet, which represent the range of tunnel excavation sizes for bored running tunnels and cross passages for both Freeway tunnel and LRT alternatives. Rock mass characteristics at this scale are expected to control overall tunnel behavior and corresponding support requirements. GSI ratings are based on qualitative identification of the appropriate rock mass structure and discontinuity strength from GSI chart (see Figure 16). Evaluations of mi and D were based on recommended values by Hoek et al. (1995) and Hoek and Diederichs (2006), respectively. A disturbance factor (D) of zero was assigned globally for the evaluation of rock mass strength since the excavation-induced disturbance is likely to be minimal if the bored running tunnels are excavated using a TBM, while cross passages would be excavated using a roadheader or excavator. Mean and lower-bound values of the Hoek-Brown envelope parameters (GSI, UCS, mi) were estimated for each of the RMTs. Table 10 summarizes the preliminary rock mass strength estimated for each of the RMTs. Corresponding equivalent strength properties shown in the table, including cohesion and frictional angle for Mohr-Coulomb yield criterion, were estimated based on the anticipated range of confining stress levels at the depth (average about 50 and 150 feet) of tunnel excavations. The approach employed for this estimation is presented in Hoek (2007).

Deformation modulus

Rock mass deformability is also a scale-dependent property. Similar to the strength properties, the deformation modulus also needs to be determined at the scale of the tunnel. The most common measure of rock mass deformability is the deformation modulus. The deformation modulus is the unloading/reloading modulus of a virgin load curve measured in the field during a pressuremeter test or back-calculated from ground movements resulting from an excavation. The unloading/reloading deformation modulus represents the response of rock mass following the tunnel excavation and support installation (an unloading and reloading process). Therefore, this modulus is recommended for tunnel design evaluation.

Estimates of the deformation modulus in a highly fractured rock mass can have significant variations (Hoek and Diederichs, 2006), but the probable range of the deformation modulus of a rock mass can be estimated (Rafael and Goodman, 1979). For this study, several approaches were used in combination to estimate the range of deformability. The methods used include:

- Empirical equations
- Available downhole seismic velocity data
- Field testing results from pressuremeter tests
- The intact rock modulus from laboratory tests

Empirical equations used to estimate the rock mass modulus are based on field measurements of deformation and include the relationships proposed by Hoek et al. (2002) and Hoek and Diederichs (2006). Where available, results from downhole shear wave velocity measurements were used to estimate the static rock mass modulus. The static rock mass modulus was estimated from the shear wave velocity using a ratio between the static modulus and dynamic modulus of 5 for highly fractured rock (Rafael and Goodman, 1979). Where available, field testing results from pressuremeter testing were also used to estimate the deformation modulus. The intact rock modulus from laboratory tests is typically considered an upper bound limit which the deformation modulus of the rock mass should not exceed. Table 10 summarizes the mean and lower-bound moduli of deformation and Poisson's ratio for each RMT. These preliminary parameters are based on the evaluations of available geotechnical data using the methods listed above and experience with similar rock. Table 11 provides a comparison of estimated deformation moduli from different approaches for each of the RMTs.

3.3 In Situ Stresses

The ratios of in situ horizontal stress to vertical stress (K_0) in bedrock have been estimated based on the results of a number of pressuremeter tests that have been completed for the project. The in situ horizontal stresses are assumed to be the initial lateral stresses indicated in the pressuremeter test reports contained in the Preliminary Geotechnical Report (CH2M HILL, 2014). The effective horizontal stress was calculated by subtracting the estimated hydrostatic pressures based on the water level indicated on the boring logs. K_0 value was then estimated based on the calculated effective horizontal stress and the effective vertical stress at the depth of the pressuremeter test.

Based on the calculated K_0 value from pressuremeter tests and past experiences from local projects including the Regional Connector Transit Corridor project, the K_0 values for sedimentary rock formations are estimated to range from approximately 0.5 to 1.35 at tunnel depths. The K_0 value for the basement complex rocks is estimated to be about 0.5 based on limited number of pressuremeter tests. It should be noted that estimation of the K_0 values involved significant uncertainties such as the degree of disturbances to the borehole walls caused by drilling, amount of stress relief in the ground prior to pressuremeter testing, and the accuracy of the model used to calculate the initial lateral stress based on pressuremeter testing data.

Tables 9 and 10 provide the recommended K_0 values for the purposes of preliminary design evaluation for the soil units and the various rock formations, respectively. These values are for typical ground conditions. In fault zones, in situ stresses could be quite different because of the past tectonic movements and varying ground conditions over short distances. These values should be verified and updated in the future design phases as additional geotechnical data becomes available.

4 Anticipated Ground Conditions and Potential Ground Behaviors

This section describes anticipated ground conditions and potential ground behaviors along the proposed tunnel alignments. The tunnel alignments of these alternatives have been divided into a number of reaches (see Figures 1 and 2), based primarily on areas expected to exhibit similar ground conditions and/or potential behaviors, which generally correspond with the contacts between geologic formations. The locations of these geologic contacts are estimated (rounded to the nearest 50 feet) based on the SR 710 North Study Preliminary Geotechnical Report (CH2M HILL, 2014). The locations and extents of tunnel reaches are subject to change as additional geological data becomes available and the geologic profiles are updated. Descriptions of potential behaviors of the excavated ground along the tunnel alignments are based on definitions provided in Table 12.

The potential ground behaviors described do not account for the effect of any ground support or stabilization measures that may be implemented to improve ground conditions and ensure tunnel stability during excavations. Tunnel excavation methods are routinely used on excavations such as these to overcome the conditions expected along these tunneled alignments. Additional information on tunnel excavation methods can be found in *Tunnel Excavation Methods* (JA, 2014).

Ground conditions that are expected to be encountered along the proposed freeway and LRT tunnel alignments have been divided into four ground classes to assist in the selection of tunnel excavation and support methods. Ground classes are defined based on the physical characteristics of the ground and its potential behaviors during tunnel excavation. The ground assigned to a particular class is expected to behave similarly in the tunnel excavation, and, therefore, to require similar excavation and support methods. Ground Class 1 represents the better rock ground conditions over the anticipated range of ground conditions along each tunnel alignment, while Ground Class 2 represents the poorer rock ground conditions anticipated. Ground Class 3 represents the soft ground or mixed-face (alluvium over bedrock) ground conditions anticipated. Ground Class 4 represents the faulted or sheared ground conditions, including potential squeezing conditions.

The RMTs and alluvium described above are grouped into four categories corresponding to four ground classes based on similarity of predominant ground behaviors in the tunnel opening. The predominant RMTs and alluvium, key characteristics of ground conditions, and the predominant behaviors associated with each of four ground classes for the tunnel portions of the Freeway Tunnel and LRT Alternatives are defined in Table 13.

In future phases of this project and as more geotechnical information becomes available, the bored tunnel alignments as well as specific locations of the proposed cross passages should be optimized to avoid or minimize adverse conditions, such as mixed-face conditions, whereever possible.

4.1 Freeway Tunnel Alternative

4.1.1 Bored Running Tunnels. The running tunnel alignment for both the freeway single-bore and twin-bore variations has been divided into eight tunnel reaches as shown in Figure 1. The locations of reach boundaries, expected predominant geologic formation(s), anticipated ground conditions, and potential ground classes (behaviors) if unsupported upon tunnel excavation are summarized in Table 14 for each of the tunnel reaches.

Depending on the location of the alignment, the tunnel is expected to encounter variable ground behaviors ranging from stable to fast raveling, spalling, caving, flowing, to squeezing, with local wedge failures along joints, bedding and/or shear planes. A separate reach, Reach No. 2, was defined for the Fernando Formation since it generally has lower strength, slake durability index, and quartz content than the adjacent Puente Formation. Reach No. 4 was defined primarily for the Basement Complex Rocks and the mixed face with alluvium over this rock formation. It should be noted that ground conditions and behaviors may vary and change abruptly with location, particularly where fault zones are expected. There is limited geotechnical data on the materials within fault zones. It is assumed at this time that materials within fault zones would exhibit squeezing behavior upon excavation.

4.1.2 Cross Passages. There are a total of six pairs of cross passages to be constructed along the alignment of the freeway twin-bore variation. Each pair of cross passages consists of one upper level cross passage and one lower level cross passage. The locations of these cross passages and geologic profile along the alignment of bored running tunnels are shown in Figure 1. Based on the anticipated geologic/ground conditions, anticipated ground class for each of the six pairs of cross passages is summarized in Table 15. The stationing shown in Table 15 for each pair of cross passages corresponds to the location of the centerline between the upper level and lower level cross passages. The length of each cross passage is approximately 80 feet. It is noted that while one ground class is assigned for each pair of cross passage based on limited geotechnical information at this preliminary design stage, conditions may also vary along the cross passages and should be further evaluated as additional geotechnical data at the cross passage locations becomes available.

4.2 LRT Alternative

4.2.1 Bored Running Tunnels. The running tunnel alignment for the LRT Alternative has been divided into ten tunnel reaches (see Figure 2). The locations of reach boundaries, expected predominant geologic formation(s), anticipated ground conditions, and potential ground classes (behaviors) if unsupported after tunnel excavation are summarized in Table 16 for each of the tunnel reaches.

Due to the relatively low ground cover along the LRT tunnel alignment, the reaches are expected to encounter soils or mixed face conditions (soil over bedrock) (see Figure 2 and Table 16). Based on the geologic profile and tunnel vertical alignment shown in Figure 2, the mixed face conditions are expected to account for approximately 25 percent of the tunnel alignment, and may be able to be avoided as the alignment is optimized in subseguent stages of the project. Excavation of the bored running tunnels with a TBM in the mixed face conditions could be challenging; however, tunneling equipment can generally be designed to overcome these situations. Refer to *Tunnel Excavation Methods TM* (JA, 2014) for a discussion of potential issues that may be encountered in mixed face condition with TBM excavation. The predominant ground behaviors are expected to include fast raveling, caving, and flowing. Ground conditions and potential ground behaviors within each reach may vary and change abruptly , particularly where fault zones are expected.

4.2.2 Cross Passages. There are a total of 26 cross passages along the alignment of the LRT Alternative. The locations of these cross passages and geologic profile along the alignment of bored running tunnels are shown in Figure 2. Based on the anticipated ground conditions, the ground class expected for each of the 26 cross passages is summarized in Table 17. Each cross passage is approximately 20 feet long. It is noted that while one ground class is assigned for each cross passage based on limited geotechnical information at this preliminary design stage, conditions may also vary along the cross passages and should be further evaluated as additional geotechnical data at the cross passage locations becomes available.

5 Summary

The ground characterization and identification of RMTs, anticipated ground conditions and behaviors, and tunnel reaches described in this TM will be useful for developing the preliminary design concepts of the freeway and LRT tunnels. This TM will serve as a reference for several other TMs and preliminary cost estimate being prepared for this project. The assessments and results presented in this memorandum are considered preliminary and are subject to change as additional geotechnical data becomes available or if the alignments change.

6 Limitations

The information and recommendations presented in this TM are preliminary interpretations based on limited geotechnical data and the tunnel alignments that were available when the TM was prepared. A significant amount of geotechnical data from additional field explorations with in situ and laboratory testing would be required in order to advance design concepts to a complete preliminary design level. It is also assumed that the alignments of the tunneled portions of the alternatives will be optimized as the study progresses, and as such the ground characterization should be updated. The findings and recommendations presented in this TM should be reassessed when additional data becomes available.

7 References

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8 Revision Log

Revision 0	December 2, 2013	Internal Review
Revision 1	January 23, 2014	Metro/Caltrans Review
Revision 2	May 27, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Table 1: Definition of Rock Strength Descriptions[1]

Strength Category	Unconfined Compressive Strength (psi)
Extremely Weak	< 150
Very Weak	150 - 700
Weak	700 – 3,500
Medium Strong	3,500 – 7,000
Strong	7,000 – 14,500
Very Strong	14,500 – 35,000
Extremely Strong	> 35,000

[1] After ISRM, 1978.

Table 2: Definition of Bedding Descriptions[1]

Description	Bedding Spacing
Massive	> 10 feet
Very Thickly Bedded	3 to 10 feet
Thickly Bedded	1 to 3 feet
Moderately Bedded	4 inches to 1 foot
Thinly Bedded	1 inch to 4 inches
Very Thinly Bedded	1/4 inch to 1 inch
Laminated	< 1/4 inch

[1] CH2M HILL, 2014

Table 3: Definition of Fracture Spacing Descriptions[1]

Description	Fracture Spacing
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 feet
Slightly Fractured	Core lengths mostly from 1 to 3 feet
Moderately Fractured	Core lengths mostly from 4 inches to 1 foot
Intensely Fractured	Core lengths mostly from 1 inch to 4 inches
Very Intensely Fractured	Mostly chips and fragments

[1] CH2M HILL, 2014.

Table 4: Definition of Rock Weathering Descriptions[1]

WEATHERING DESCRIPTORS FOR INTACT ROCK						
Description	Diagnostic Features					General Characteristics
	Chemical Weathering-Discoloration and/or Oxidation		Mechanical Weathering-Grain Boundary Conditions (Disaggregation) Primarily for Granitics and Some Coarse-Grained Sediments	Texture and Leaching		
	Body of Rock	Fracture Surfaces		Texture	Leaching	
Fresh	No discoloration, not oxidized.	No discoloration or oxidation.	No separation, intact (tight).	No change	No leaching	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull.	Minor to complete discoloration or oxidation of most surfaces.	No visible separation, intact (tight).	Preserved	Minor leaching of some soluble minerals.	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty," feldspar crystals are "cloudy."	All fracture surfaces are discolored or oxidized.	Partial separation of boundaries visible.	Generally preserved	Soluble minerals may be mostly leached.	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in-situ disaggregation, see grain boundary conditions.	All fracture surfaces are discolored or oxidized, surfaces friable.	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated.	Texture altered by chemical disintegration (hydration, argillation).	Leaching of soluble minerals may be complete.	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated).	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete.		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes."

[1] CH2M HILL, 2014.

Table 5: Slake Durability Classification[1]

Slake Durability Class	Slake Durability Index
Very high	>98
High	95–98
Medium-high	85–95
Medium	60–85
Low	30–60
Very Low	<30

[1] Gamble, 1971

Table 6: Summary of Laboratory Test Results for Shear Strength and Atterberg Limit Tests on Fine-Grained Soils

Soil Unit	Undrained Shear Strength* (ksf)		Plasticity Index		Liquid Limit	
	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests
Artificial Fill	2.1 – 4.1	2	24 – 33	4	37 – 50	4
Alluvium	1.2 – 6.7	9	3 – 39	55	17 – 58	55

* Undrained shear strength from unconsolidated undrained (UU) triaxial tests.

Table 7: Summary of Test Results on Rock Samples

Geologic Formation	Total Unit Weight (pcf)		Plasticity Index		Liquid Limit		UCS (psi)		PLI (psi)		RQD [1]		Cerchar Abrasivity Index [2]		Quartz Content [2] (%)		Slaking Durability Index [3]	
	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests	Range	No. of Tests
Fernando Formation	106 – 137	72	22 – 42	18	44 – 63	18	48 – 533	24	1 – 34	3	0 – 100	72	N/A	0	7 – 10	2	0.6 – 87	14
Puente Formation	107 – 161	33	19 – 38	9	47 – 60	9	36 – 519	19	23 – 511	3	0 – 100	52	N/A	0	15 – 20	2	0.0 – 96	17
Topanga Formation	108 – 157	9	19 – 35	6	47 – 59	6	13 – 4,898	14	4 – 290	14	7 – 100	29	N/A	0	N/A	0	14.1 – 98.0	7
Basement Complex Rocks	91 – 164	21	11	1	25	1	35 – 1,593	14	6 – 187	13	0 – 76	45	0.0 – 19.0	3	13 – 21	3	N/A	0

Notes: N/A=Not available; UCS=Unconfined compressive strength; PLI=Point load index (axial corrected index)

[1] A significant portion of the cores included in the RQD calculation do not necessarily meet the "sound core" definition provided in the standard test method for RQD (ASTM D6032), so care should be taken when using these RQD values to evaluate the overall rock mass quality.

[2] Refer to *Tunnel Excavation Methods* (JA, 2014) for discussion concerning rock abrasivity and quartz content on tunnel construction and costs.

[3] Refer to Table 5 for slake durability classification.

Table 8: Summary of Packer Test Results (CH2M HILL, 2010)

Geologic Unit	Estimated Permeability (cm/s)	No. of Tests
Fernando Formation	8.2×10^{-7} to 2.8×10^{-6}	7
Puente Formation	2.3×10^{-6} to 2.1×10^{-5}	27
Topanga Formation	4.6×10^{-7} to 4.0×10^{-4}	36
Basement Complex Rocks	9.8×10^{-7} to 1.3×10^{-5}	10

Table 9: Recommended Soil Parameters for Preliminary Design Evaluation

Soil Type	Range	Total Unit Weight (pcf)	Deformation Modulus (ksi)	Poisson's Ratio	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Horizontal-to-vertical Stress Ratio (K_0) [Range]
Fill	Mean	120	2.0	0.30	0	32	0.5
Alluvium	LB	125	6.9	0.35	0	32	0.6 [0.4 – 1.2]
	Mean		13.9		500	36	

Note: LB=Lower Bound

Table 10: Recommended Rock Mass Parameters for Preliminary Design Evaluations

Geologic Formation	Range	Total Unit Weight (pcf)	Average Applicable Depth (ft)	GSI Classification Parameters *				Hoek-Brown Model Parameters *			Equivalent Mohr-Coulomb Model Parameters		Deformation Modulus (ksi)	Poisson's Ratio	Horizontal-to-vertical Stress Ratio (K_0)
				Intact Rock UCS (psi)	GSI	mi	D	m_b	s	a	Cohesion (psf)	Friction Angle (degrees)			
Fernando Formation, Tf	LB	136	150	50	N/A						1,050	20	10	0.35	0.65 (0.5 – 0.8)
	Mean			300							1,950	29	25		
Puente Formation, Tp-2	LB	134	50	30	35	6	0	0.589	0.0007	0.516	250	17	10	0.30	0.7 (0.5 – 1.35)
	Mean			50	45	6	0	0.842	0.0022	0.508	400	22	15		
Puente Formation, Tp-1	LB	134	150	150	45	10	0	1.403	0.0022	0.508	1,300	26	35	0.30	0.7 (0.5 – 1.35)
	Mean			400	55	10	0	2.005	0.0067	0.504	2,300	36	70		
Topanga Formation, Tt-2	LB	134	50	30	40	6	0	0.704	0.0013	0.511	300	17	10	0.30	0.7 (0.5 – 1.35)
	Mean			60	50	6	0	1.006	0.0039	0.506	450	24	15		
Topanga Formation, Tt-1	LB	134	150	230	50	12	0	2.012	0.0039	0.506	1,800	32	40	0.30	0.7 (0.5 – 1.35)
	Mean			500	60	12	0	2.876	0.0117	0.503	2,950	41	100		
Basement Complex Rock, Wqd-2	LB	158	50	35	30	25	0	2.052	0.0004	0.522	450	25	15	0.25	0.5 (0.4 – 0.6)
	Mean			80	45	25	0	3.506	0.0022	0.508	800	35	20		
Basement Complex Rocks, Wqd-1	LB	158	150	250	50	25	0	4.192	0.0039	0.506	2,600	37	50	0.25	0.5 (0.4 – 0.6)
	Mean			680	60	25	0	5.991	0.0117	0.503	4,350	48	130		

Notes: * UCS=Uniaxial Compressive Strength; GSI=Geological Strength Index; mi=Hoek-Brown constant related to rock type and lithology; D=Disturbance Factor which depends upon the degree of disturbance caused by excavation; mb, s, and a=Hoek-Brown constants related to rock mass strength and characteristics.

Table 11: Summary of Rock Mass Deformation Modulus Estimates

Rock Mass Type		Intact Rock Modulus (ksi)[1]	Detailed Hoek-Diederichs (2006) (ksi)	Pressure-meter Modulus (ksi)[2]	Shear Wave Velocity Method (ksi)[3]	Proposed Deformation Modulus (ksi)
Tf	LB	10	N/A	10	12	10
	Mean	30		39	19	25
Tp-2	LB	10	2	N/A	11	10
	Mean	25	6		37	15
Tp-1	LB	80	33	52	27	35
	Mean	120	49	280	75	70
Tt-2	LB	30	9	N/A	7	10
	Mean	60	18		24	15
Tt-1	LB	100	52	36	60	40
	Mean	350	182	296	129	100
Wqd-2	LB	10	5	N/A	17	15
	Mean	40	21		29	20
Wqd-1	LB	65	52	118	44	50
	Mean	600	312	615	217	130

[1] Estimated unloading/reloading modulus based on UCS test data, except for the Fernando Formation. For the Fernando Formation, the initial secant modulus is indicated. Number of tests: Tf – 15 tests; Tp – 13 tests; Tt – 11 tests; and Wqd – 11 tests.
[2] Estimated unloading/reloading modulus. Number of measurements: Tf – 4; Tp – 10; Tt – 4; and Wqd – 6.
[3] A reduction factor of 0.2 was assumed for estimating the static rock mass modulus from the shear wave velocity (Raphael and Goodman, 1979).

Table 12: Definitions of Ground Behaviors[1]

Classification	Behavior
Firm/Stable	Excavated tunnel stands unsupported for several days or longer. The term includes a great variety of materials: sands and sand-gravels with clay binder, stiff unfissured clays at moderate depths, and massive rock.
Flowing	A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and wall, and can flow for great distances, completely filling the tunnel in some cases.
Raveling/Caving	Chunks or flakes of material begin to drop out of the crown or sidewalls sometime after the ground has been exposed, due to loosening or to overstress and “brittle” fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.
Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (±30°-35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose. Can become flowing ground in the presence of water.
Slaking/Softening	Slaking is the deterioration and breakdown of intact rock upon exposure by excavation and manifests as slabbing of material from the crown and sidewalls. The severity of this behavior is assessed based on the slake durability index. Softening, which is dependent on wetting and exposure by excavation, is the reduction of intact rock strength at the invert or elsewhere and manifests as the development of a muddy or unstable invert or sloughing along segments of the tunnel perimeter.
Spalling	Spalling occurs when spalls or rock fragments break or separate from the crown or sidewalls as a result of tensile failure caused by blasting or due to high in-situ stress. Spalling usually occurs in hard, massive to moderately jointed rock.
Squeezing	Ground slowly advances into the tunnel, without visible fracturing or loss of continuity, and without a perceptible increase in water content. Ductile, plastic yield, and time-dependent deformation due to overstress. The rate of squeeze depends on the degree of overstressing.
Structurally Controlled Block Instability	Structurally (discontinuity) controlled, gravity-induced failure of rock blocks that manifest as falling and sliding of blocks.
Swelling	Ground absorbs water, increases in volume, and expands slowly into the tunnel.

[1] Modified Tunnelman’s Ground Classification after Heuer (1974) and Proctor and White (1977).

Table 13: Definitions of Ground Classes

Ground Class	Soil or Rock Mass Formation and Type	Soil or Rock Characteristics and Ground Condition[1]	Potential Ground Behavior[2]
1	Puente Formation (RMT Tp-1), Topanga Formation (RMT Tt-1), and Basement Complex Rocks (Wqd-1)	Sandstone, siltstone, and shale with interbedded siltstone/sandstone; igneous and metamorphic rocks; predominantly moderately to very slightly fractured; slightly weathered to fresh; weak to very strong rock; Massive, moderately jointed to moderately blocky and seamy[3]; Massive to blocky[4]	Structurally-controlled block instability or slabbing; stable to slow raveling
2	Fernando Formation (RMT Tf), Puente Formation (RMT Tp-2), Topanga Formation (RMT Tt-2), and Basement Complex Rocks (Wqd-2)	Sandstone, siltstone, interbedded siltstone/sandstone, shale, and claystone/mudstone; igneous and metamorphic rocks; predominantly intensely to moderately fractured, highly to moderately weathered; extremely weak to very weak rock; Massive, moderately jointed to completely crushed[3]; Blocky to disintegrated[4]	Unstable conditions exhibiting slow to fast raveling/caving; structurally-controlled instability or slabbing; and slaking/softening
3	Soil (primarily alluvium) or mixed-face conditions (alluvium over bedrock)	Loose to very dense sand and gravel deposits; soft to hard silt and clay deposits	Fast raveling/caving; flowing if groundwater inflows are not controlled; slaking/softening
4	Fault or shear zones (in Fernando Formation, Puente Formation, Topanga Formation, or Basement Complex Rocks)	Heavily sheared or faulted rock including clay gouge/infilling materials, shattered rock, poorly laminated rock; extremely weak to very weak rock; moderately to completely weathered; Completely Crushed to Squeezing at Moderate Depth[3]; Disintegrated to Laminated/Sheared[4]	Squeezing; swelling; fast raveling/caving; slaking/softening

[1] Discontinuity, weathering, and rock strength characteristics are defined in Tables 1 to 4.
[2] Potential ground behavior represents behavior of ground if no stabilization measures are implemented for ground control.
[3] Per Terzaghi's rock mass classification system.
[4] Per GSI system.

Table 14: Summary of Tunnel Reaches for Freeway Tunnel Alternative

Reach No.	Stationing		Length (ft)	Geologic Formation	Ground Cover (ft)	GW Head above Crown (ft)	Anticipated Fault Zones	Cross Passage (CP) Number [c]	Anticipated Ground Classes
	From	To							
1	1500+00	1524+50	2,450	Puente Formation (Tp-1, Tp-2)	55 - 140	30 - 110	Highland Park Fault	No CP	Ground Class 1 Ground Class 2 Ground Class 4
2	1524+50	1560+00	3,550	Fernando Formation (Tf)	120 - 170	100 - 110	None	1	Ground Class 2
3	1560+00	1624+50	6,450	Topanga Formation [a] (Tt-1, Tt-2) Puente Formation (Tp-1, Tp-2)	125 - 280	65 - 150 [b]	Unnamed Fault B Unnamed Fault C	2, 3, 4	Ground Class 1 Ground Class 2 Ground Class 4
4	1624+50	1658+00	3,350	Alluvium (Qal) Basement Complex Rocks (Wqd-1, Wqd-2) Topanga Formation (Tt-1, Tt-2)	145 - 155	0 - 65	None	5	Ground Class 1 Ground Class 2 Ground Class 3
5	1658+00	1667+00	900	Topanga Formation (Tt-1, Tt-2)	155 - 165	0 - 140	Raymond Fault Eagle Rock Fault	No CP	Ground Class 1 Ground Class 2 Ground Class 4
6	1667+00	1690+50	2,350	Topanga Formation (Tt-1, Tt-2)	105 - 165	20 - 120	None	6	Ground Class 1 Ground Class 2
7	1690+50	1695+00	450	Topanga Formation [a] (Tt-1, Tt-2) Alluvium (Qal)	95 - 105	0 - 20	San Rafael Fault	No CP	Ground Class 1 Ground Class 2 Ground Class 3 Ground Class 4
8	1695+00	1723+40	2,840	Alluvium (Qal)	20 - 95	0	None	No CP	Ground Class 3

a) Prominent geologic formation
b) Approximated groundwater elevation was interpolated
c) Refer to Figure 1 for locations of cross passages.

Table 15: Locations and Characteristics of Cross Passages for Freeway Tunnel Alternative

CP No.	Stationing	Ground Cover (ft)	Soil Overburden Depth (ft)	Ground Surface Elevation (XP Crown Elevation) (ft)	Ground Class/Ground Condition	Groundwater Table Elevation (Head at Crown Elevation) (ft)
1	1533+75	140 - 160	85	480 (310 – 330)	Ground Class 2: In Rock; Fernando Formation (Tf)	440 (110 - 130)
2	1563+75	130 - 150	20	510 (360 – 380)	Ground Class 2: In Rock; Puente Formation (Tp-2)	490 (110 - 130)
3	1593+75	280 - 300	0	720 (420 – 440)	Ground Class 1: In Rock; Topanga Formation (Tt-1)	720* (280 - 300)
4	1623+75	150 - 170	0	640 (470 – 490)	Ground Class 1: In Rock; Topanga Formation (Tt-1)	640* (150 - 170)
5	1653+75	150 - 170	250	700 (530 – 550)	Ground Class 3: In Soil; Alluvium (S)	540 (0 - 10)
6	1683+75	125 - 145	100	780 (630 – 650)	Ground Class 2: In Rock; Topanga Formation (Tt-2)	710 (60 - 80)

*Approximated groundwater elevation was interpolated

Table 16: Summary of Tunnel Reaches for LRT Alternative

Reach No.	Stationing		Length (ft)	Geologic Formation	Ground Cover (ft)	GW Head above Crown (ft)	Anticipated Fault Zones	Cross Passage (CP) Number [b]	Anticipated Ground Classes
	From	To							
1	170+40	178+50	810	Alluvium (Qal)	10 - 50	0 - 15	Unnamed Fault A [c]	No CP	Ground Class 3
2	178+50	193+00	1,450	Alluvium (Qal) Puente Formation (Tp-2)	50 - 65	15 - 30	None	1, 2	Ground Class 2 Ground Class 3
3	193+00	220+50	2,750	Alluvium (Qal)	55	5 - 15	Highland Park Fault [c]	3, 4, 5, 6	Ground Class 3
4	220+50	276+00	5,550	Alluvium (Qal) Puente Formation (Tp-2)	50 - 65	0 – 10 [a]	Unnamed Fault B	7, 8, 9, 10, 11, 12	Ground Class 2 Ground Class 3 Ground Class 4
5	276+00	299+50	2,350	Topanga Formation (Tt-1, Tt-2)	50 - 85	0 (?)	None	13, 14, 15	Ground Class 1 Ground Class 2
6	299+50	352+50	5,300	Alluvium (Qal)	55 - 75	0	None	16, 17, 18, 19, 20	Ground Class 3
7	352+50	354+50	200	Alluvium (Qal) Topanga Formation (Tt-2)	75	65	Raymond Fault	No CP	Ground Class 2 Ground Class 3 Ground Class 4
8	354+50	378+50	2,400	Topanga Formation (Tt-1, Tt-2)	60 - 90	0 - 65	San Rafael Fault	21, 22, 23	Ground Class 1 Ground Class 2 Ground Class 4
9	378+50	384+50	600	Alluvium (Qal) Topanga Formation (Tt-2)	60	0	San Rafael Fault	24	Ground Class 3
10	384+50	402+20	1,770	Alluvium (Qal)	55 - 60	0	None	25, 26	Ground Class 3

Notes: a) Approximated groundwater elevation was interpolated
b) Refer to Figure 2 for locations of cross passages.
c) These fault zones are anticipated to be located below the tunnel invert elevation.

Table 17: Locations and Characteristics of Cross Passages for LRT Alternative

CP No.	Stationing	Ground Cover (ft)	Soil Overburden Depth (ft)	Ground Surface/CP Crown Elevation (ft)	Ground Class/Ground Condition	Groundwater Table Elevation/ Head at Crown Elevation (ft)
1	181+45.00	55	55	425/370	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	390/20
2	188+90.43	60	60	440/380	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	400/20
3	196+15.43	55	85	445/390	Ground Class 3: In Soil; Alluvium	400/10
4	203+40.72	60	90	460/400	Ground Class 3: In Soil; Alluvium	410/10
5	210+65.70	60	80	460/400	Ground Class 3: In Soil; Alluvium	410/10
6	217+90.68	60	80	470/410	Ground Class 3: In Soil; Alluvium	420/10
7	232+53.99	55	55	475/420	Ground Class 3: In Soil; Alluvium or Mixed Face; Alluvium over Fernando Formation	430/10
8	240+04.04	60	50	490/430	Ground Class 2: In Rock; Puente Formation (Tp-2)	440*
9	247+54.08	60	60	500/440	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	440*
10	255+04.13	55	65	515/460	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Puente Formation (Tp-2)	450*
11	262+54.17	55	60	525/470	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Puente Formation (Tp-2)	470*
12	270+04.21	60	65	540/480	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Topanga Formation (Tt-2)	480*
13	277+54.25	60	60	560/500	Ground Class 2: In Rock; Topanga Formation (Tt-2)	500*
14	285+03.84	50	0	560/510	Ground Class 1: In Rock; Topanga Formation (Tt-1)	560*
15	292+53.83	75	0	600/525	Ground Class 1: In Rock; Topanga Formation (Tt-1)	600*
16	308+53.99	60	220	600/540	Ground Class 3: In Soil; Alluvium	490
17	316+04.03	60	300	610/550	Ground Class 3: In Soil; Alluvium	500
18	323+54.06	60	220	620/560	Ground Class 3: In Soil; Alluvium	520
19	331+04.09	55	110	635/580	Ground Class 3: In Soil; Alluvium	550
20	349+04.06	80	230	690/610	Ground Class 3: In Soil; Alluvium	540
21	356+54.09	80	50	720/640	Ground Class 2: In Rock; Topanga Formation (Tt-2)	690/50
22	364+04.37	80	0	750/670	Ground Class 1: In Rock; Topanga Formation (Tt-1)	750*
23	371+54.40	80	0	760/680	Ground Class 1: In Rock; Topanga Formation (Tt-1)	760*
24	379+04.94	60	70	750/690	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Topanga Formation (Tt-2)	690*
25	386+54.44	60	180	760/700	Ground Class 3: In Soil; Alluvium	650
26	394+04.44	55	180	760/705	Ground Class 3: In Soil; Alluvium	650

*Approximated groundwater elevation was interpolated

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

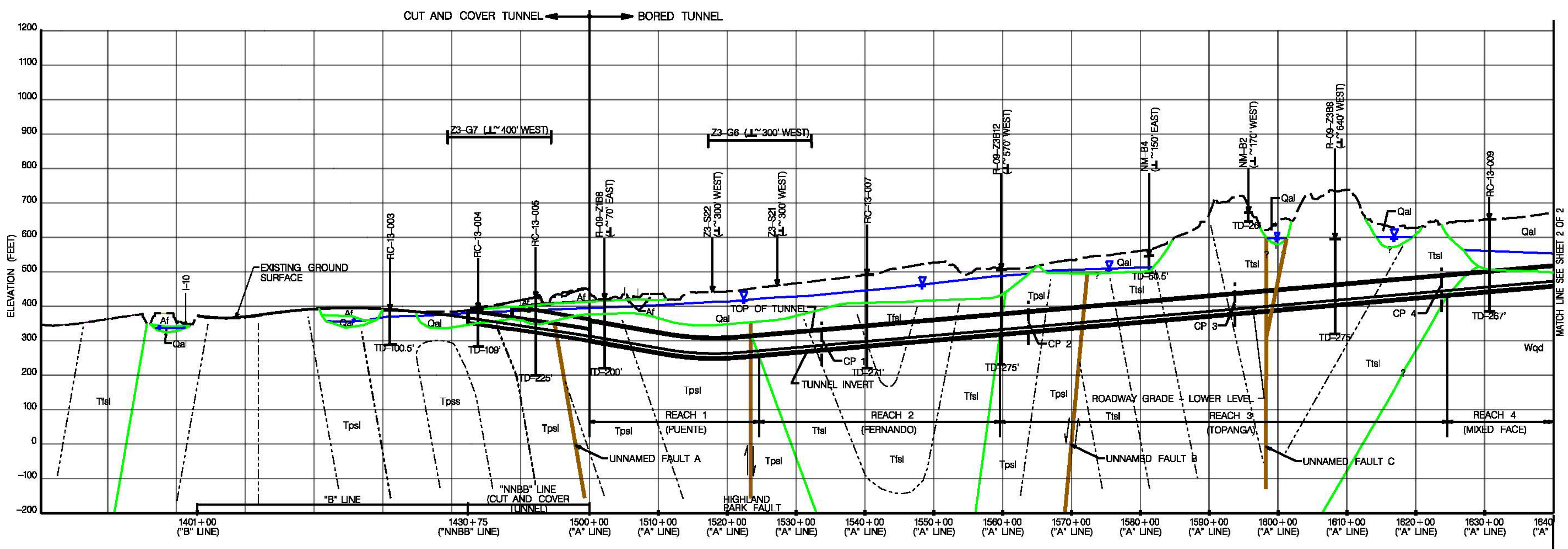
2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/1/2013.

5) PREDOMINATE GEOLOGIC FORMATION WITHIN EACH REACH IS SHOWN IN PARENTHESIS UNDER THE REACH NUMBER. "MIXED FACE" IS DEFINED AS SOIL OVER BEDROCK WITHIN THE FACE OF TUNNEL EXCAVATION.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
- INTRAFORMATIONAL CONTACT
- GENERALIZED BEDDING
- ESTIMATED TOP OF GROUNDWATER TABLE
- Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION
- A-13-020 TD-149.8' GEOTECHNICAL BOREHOLE WITH TOTAL DEPTH AND PROJECTION:
  A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
  R-09-Z1B8 – CH2M HILL, 2010
  NM-B3 – NINYO AND MOORE, 1999
  EMI-3 – EARTH MECHANICS INC, 2006
  ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FIGURE 1
FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

Figure 1a: Freeway Tunnel Geologic Profile

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

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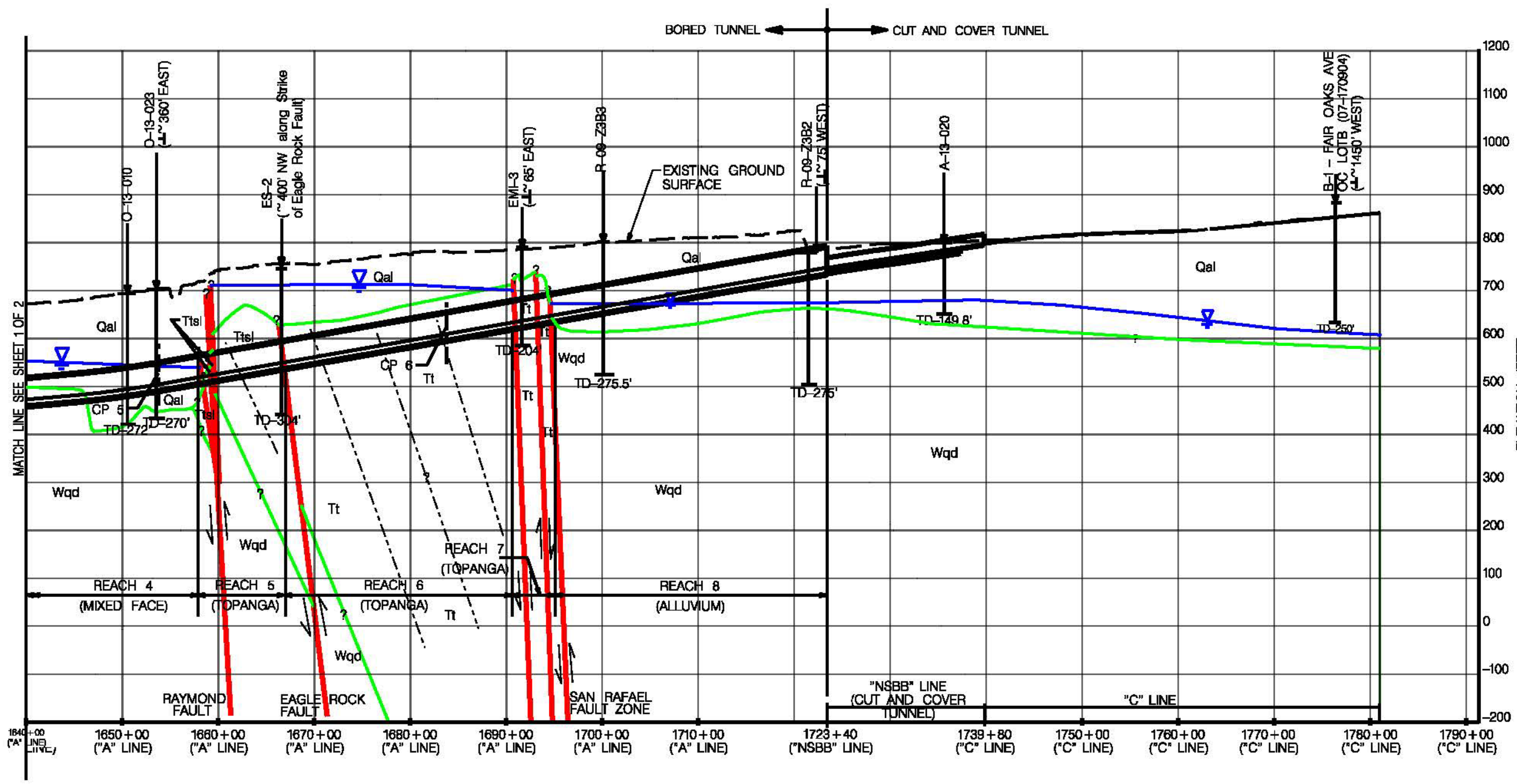
4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

5) PREDOMINATE GEOLOGIC FORMATION WITHIN EACH REACH IS SHOWN IN PARENTHESIS UNDER THE REACH NUMBER. "MIXED FACE" IS DEFINED AS SOIL OVER BEDROCK WITHIN THE FACE OF TUNNEL EXCAVATION.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

- Af ARTIFICIAL FILL
- Qal ALLUVIAL SOIL
- Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER
- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
- Tpsl PUENTE FORMATION, SILTSTONE MEMBER
- Tpss PUENTE FORMATION, SANDSTONE MEMBER
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  - NM-B3 – NINYO AND MOORE, 1999
  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FIGURE 1
FREEWAY GEOLOGIC PROFILE
SHEET 2 OF 2

Figure 1b: Freeway Tunnel Geologic Profile

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

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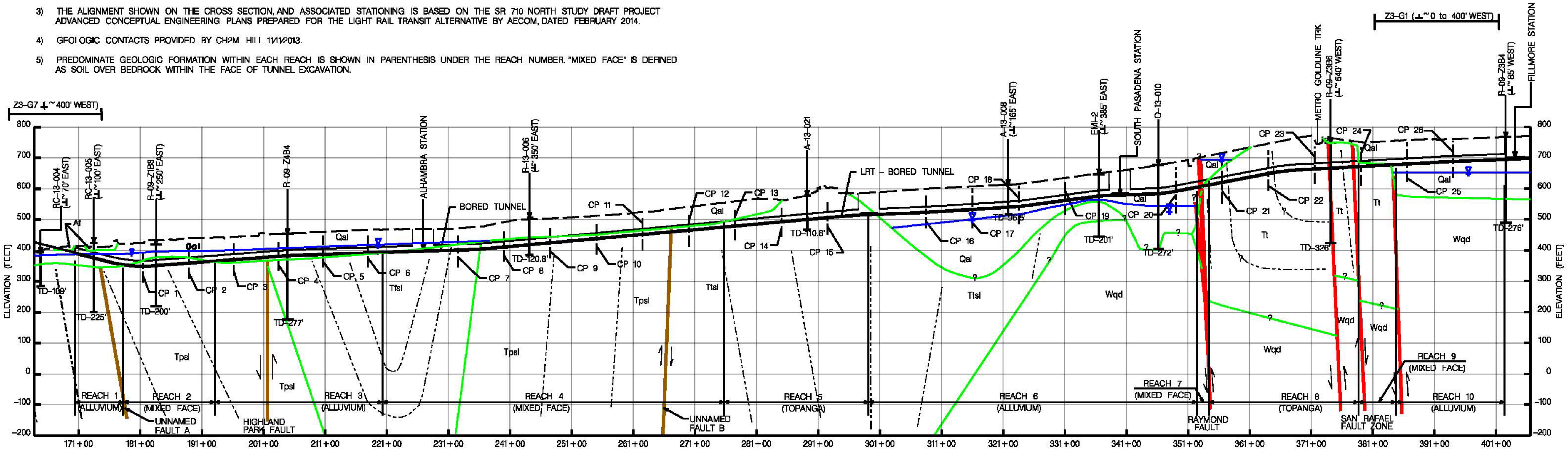
Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative



LEGEND

UNITS

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- Qal ALLUVIAL SOIL
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- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
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  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FIGURE 2
LRT GEOLOGIC PROFILE

Figure 2: LRT Tunnel Geologic Profile

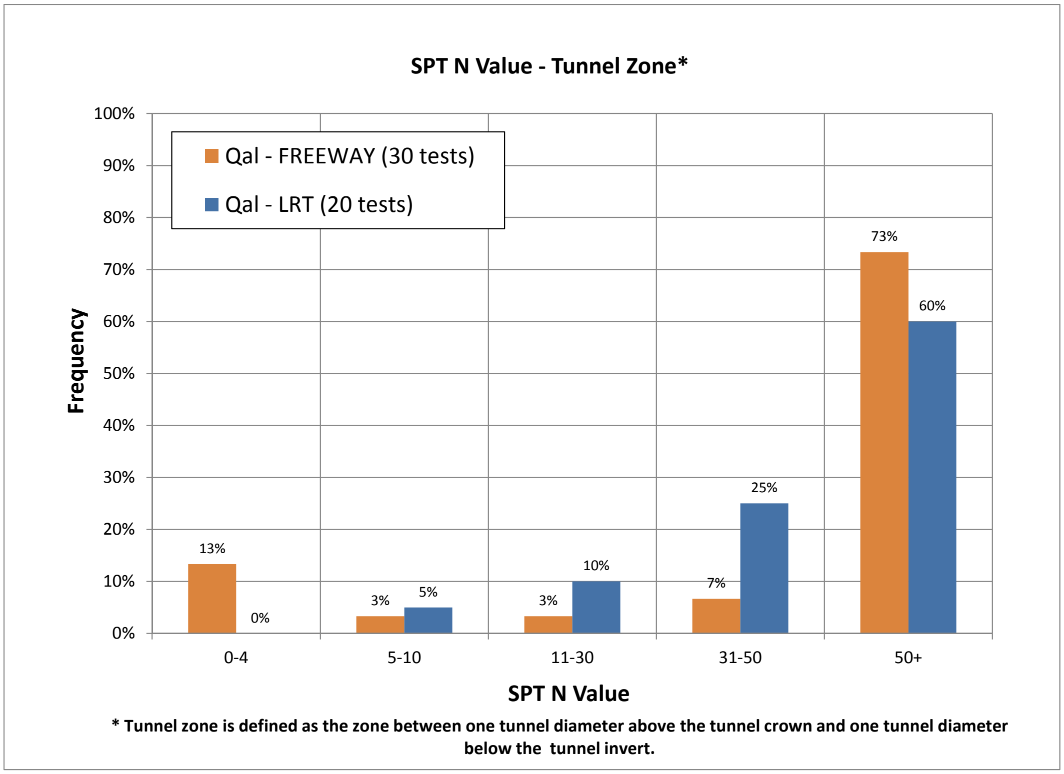
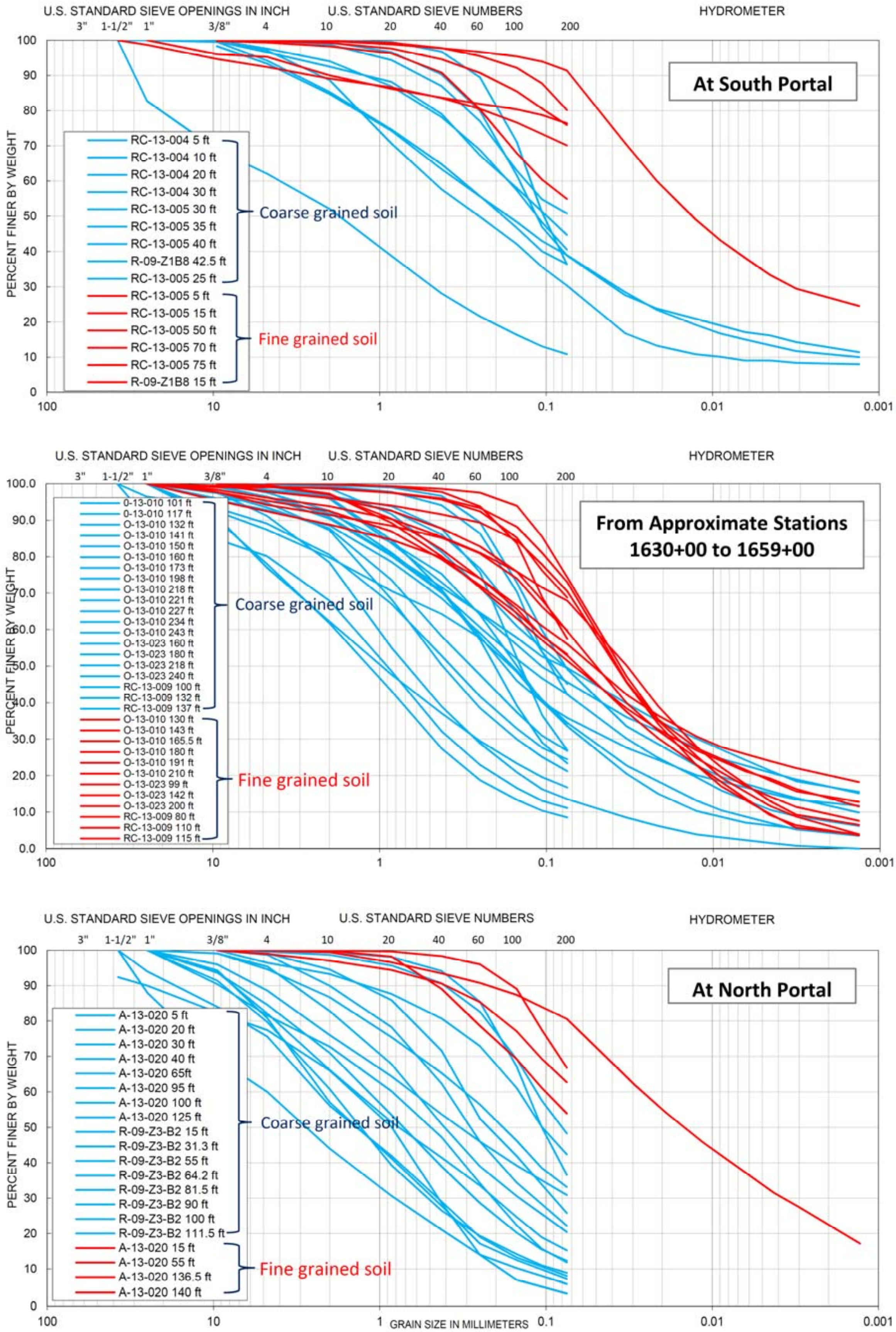
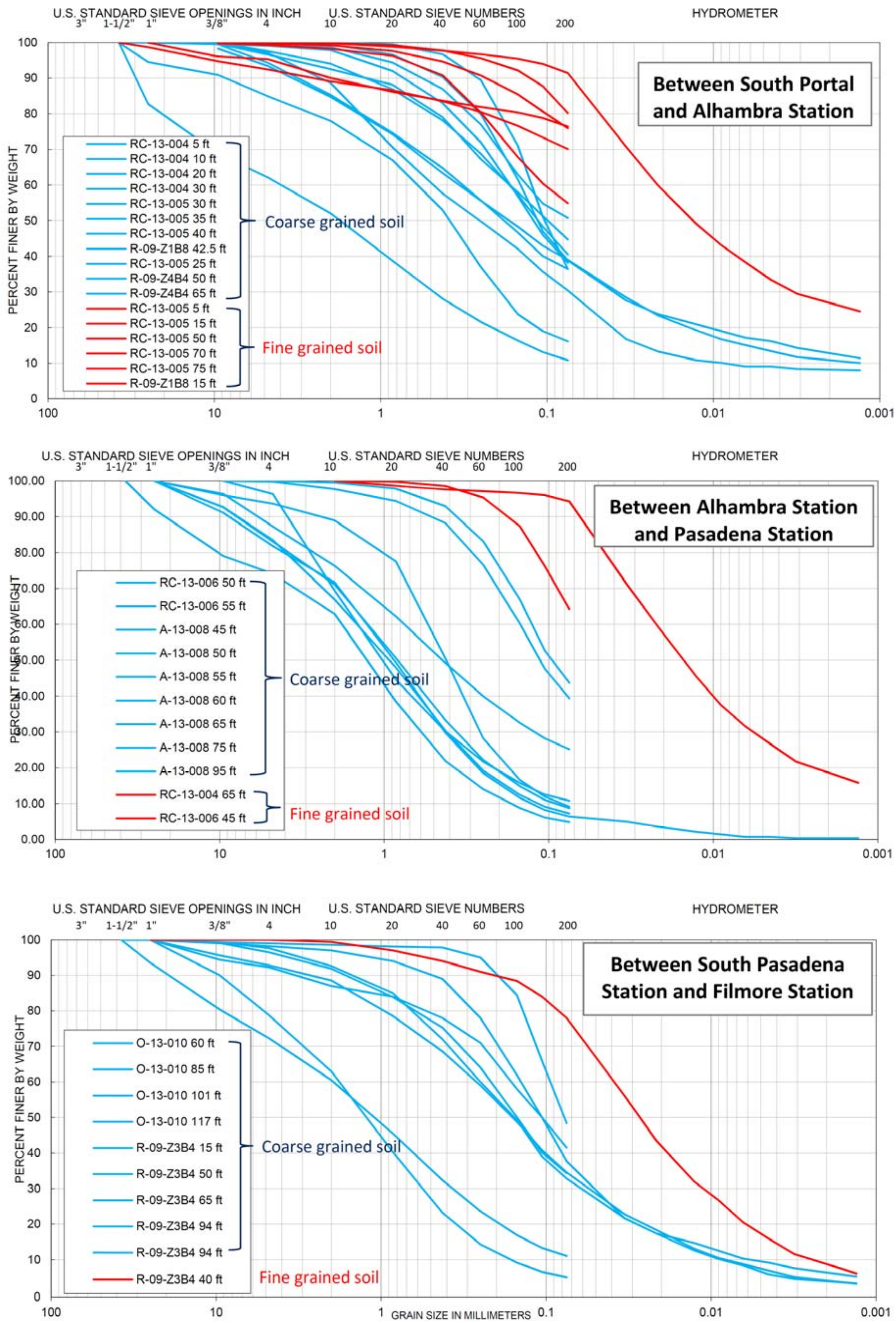


Figure 3: SPT N Values in Alluvium within Tunnel Zone



(a) along Freeway Tunnel Alternative



(b) along LRT Tunnel Alternative

Figure 4: Grain Size Distribution Curves from Alluvial Soil Samples (Grain size curves from CH2M HILL, 2014)

Figure 5: Typical Fernando Formation Core Samples (RMT Tf) (Cores from Boring RC-13-007)

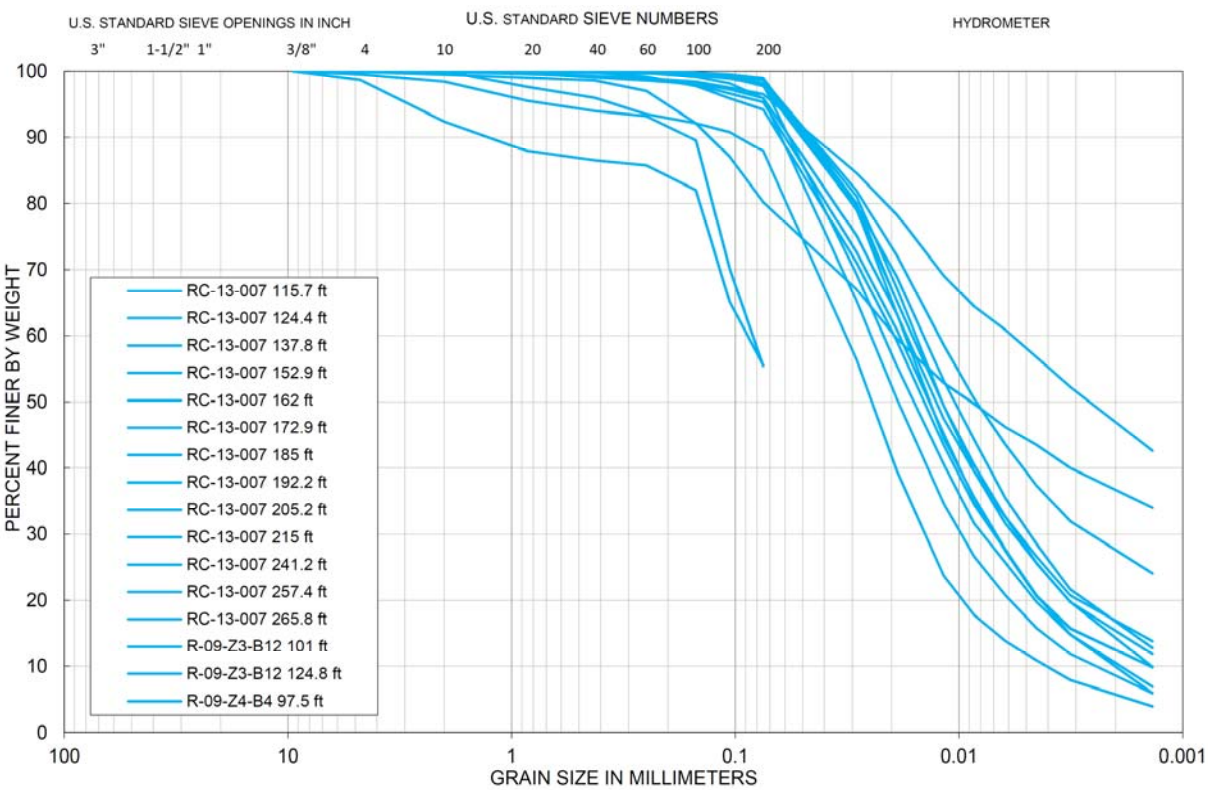


Figure 6: Grain Size Distribution Curves from Samples of Fernando Formation Siltstone Member (Tfsl)
(Grain size curves from CH2M HILL, 2014)

(a) RMT Tp-2

(b) RMT Tp-1

Figure 7: Examples of Puente Formation (Cores from Boring RC-13-005)

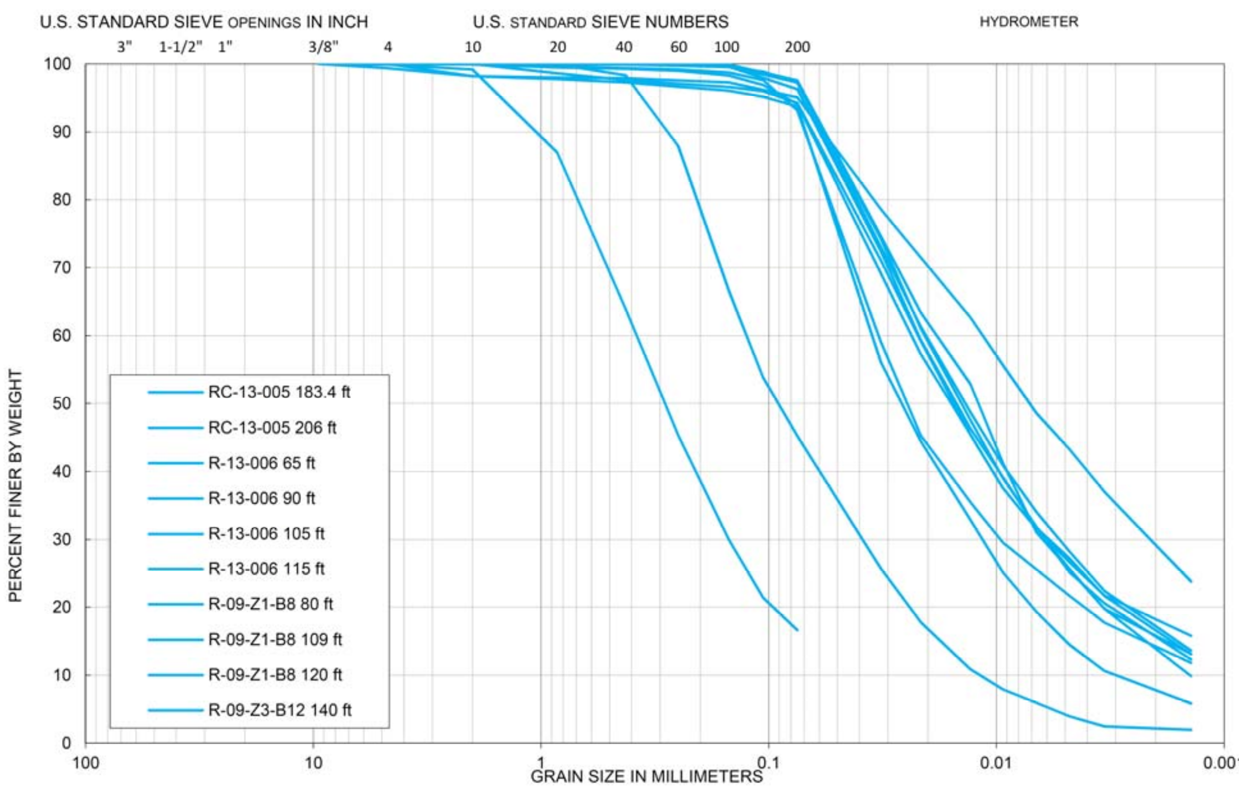
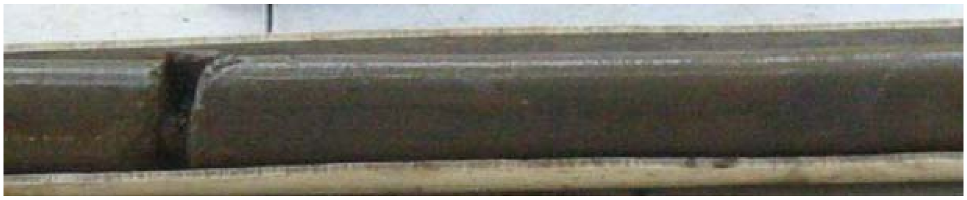


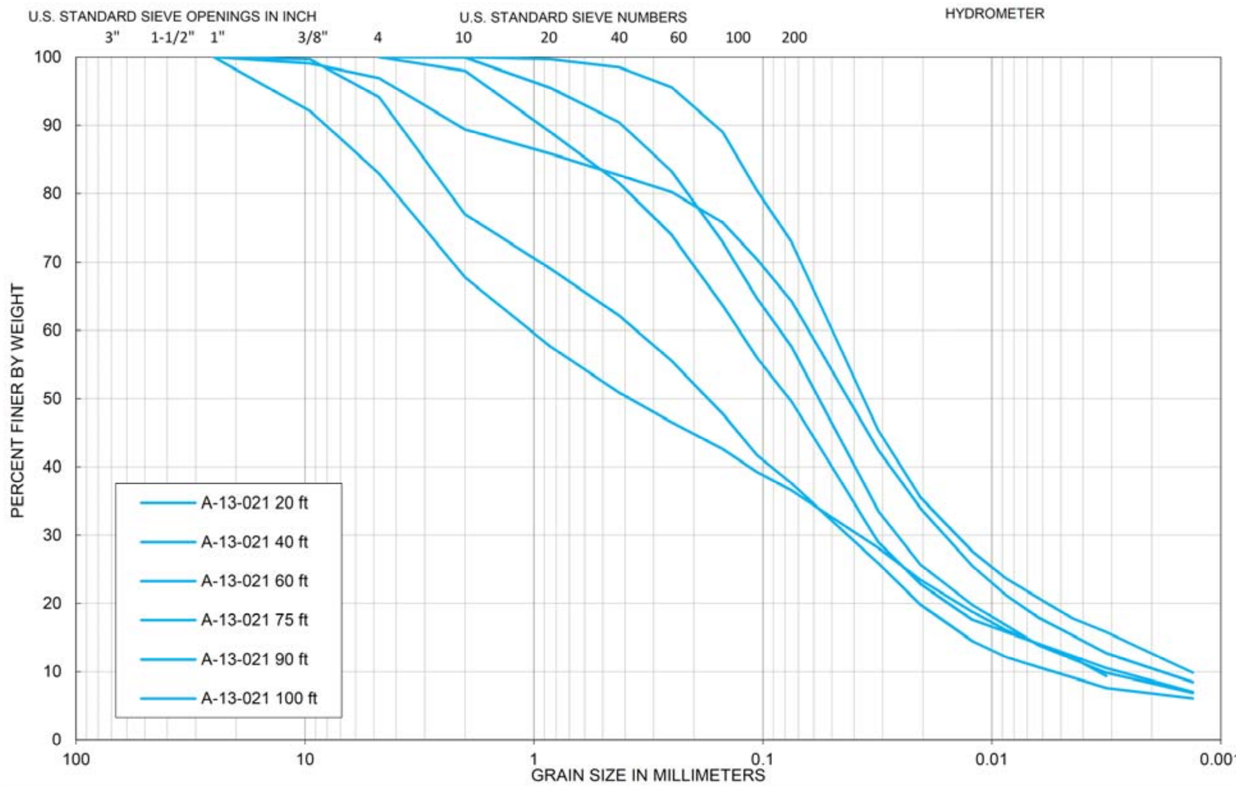
Figure 8: Grain Size Distribution Curves from Samples of Puente Formation Siltstone Member (Tpsl) (Grain size curves from CH2M HILL, 2014)

(a) RMT Tt-2

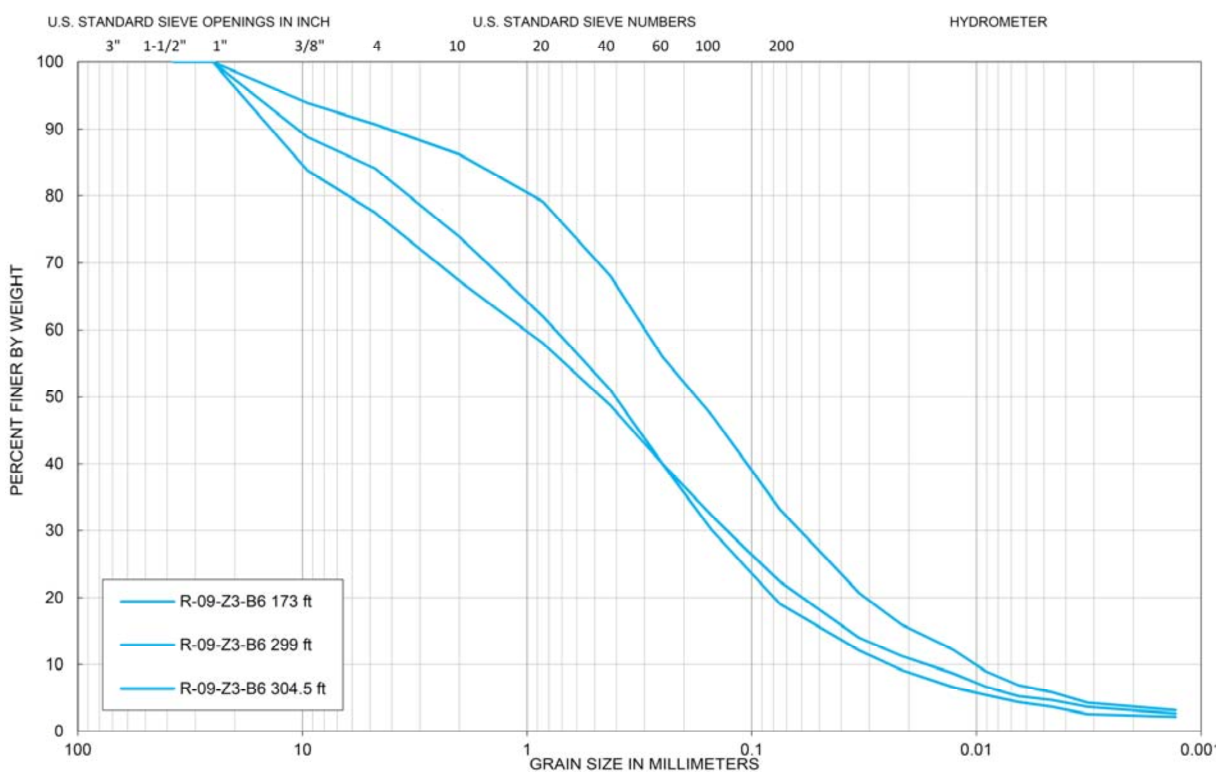


(b) RMT Tt-1

Figure 9: Examples of Topanga Formation (Cores from Boring R-09-Z3B8)



(a) Siltstone Member (Ttsl)



(b) Conglomerate Member (Ttcg)

Figure 10: Grain Size Distribution Curves from Samples of Topanga Formation (Grain size curves from CH2M HILL, 2014)

(a) RMT Wqd-2 (Cores from Boring R-09-Z3B4)

(b) RMT Wqd-1 (Cores from Boring RC-13-009)

Figure 11: Examples of Basement Complex Rocks

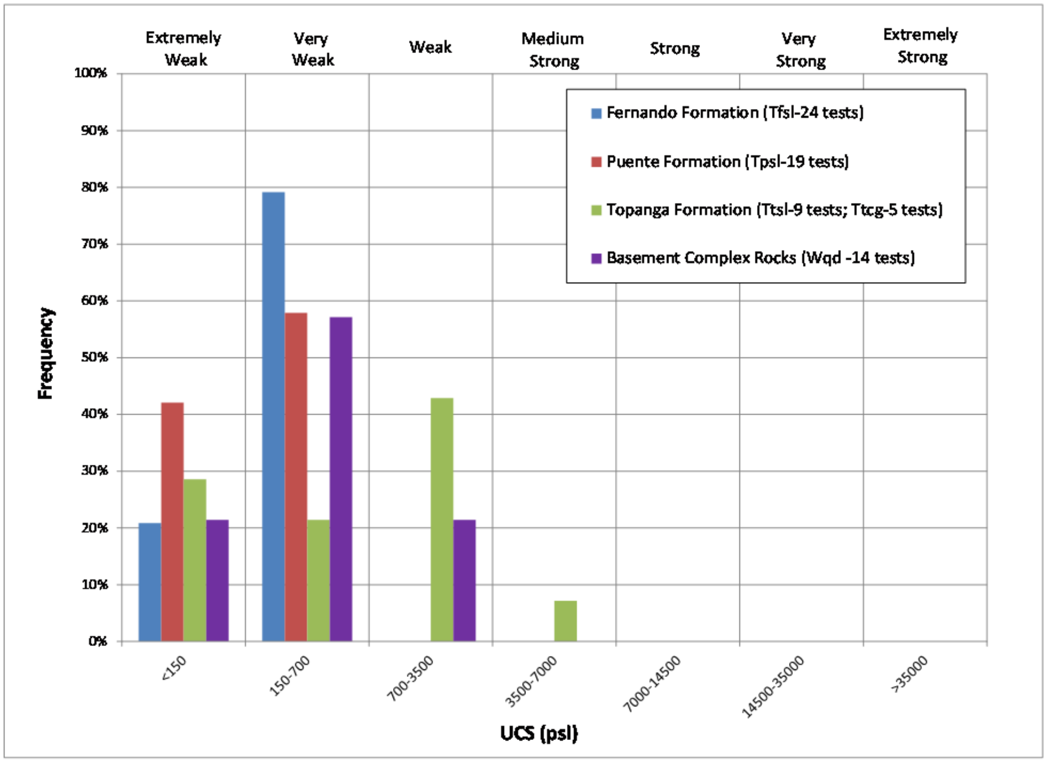


Figure 12: Histogram of UCS for Various Rock Formations

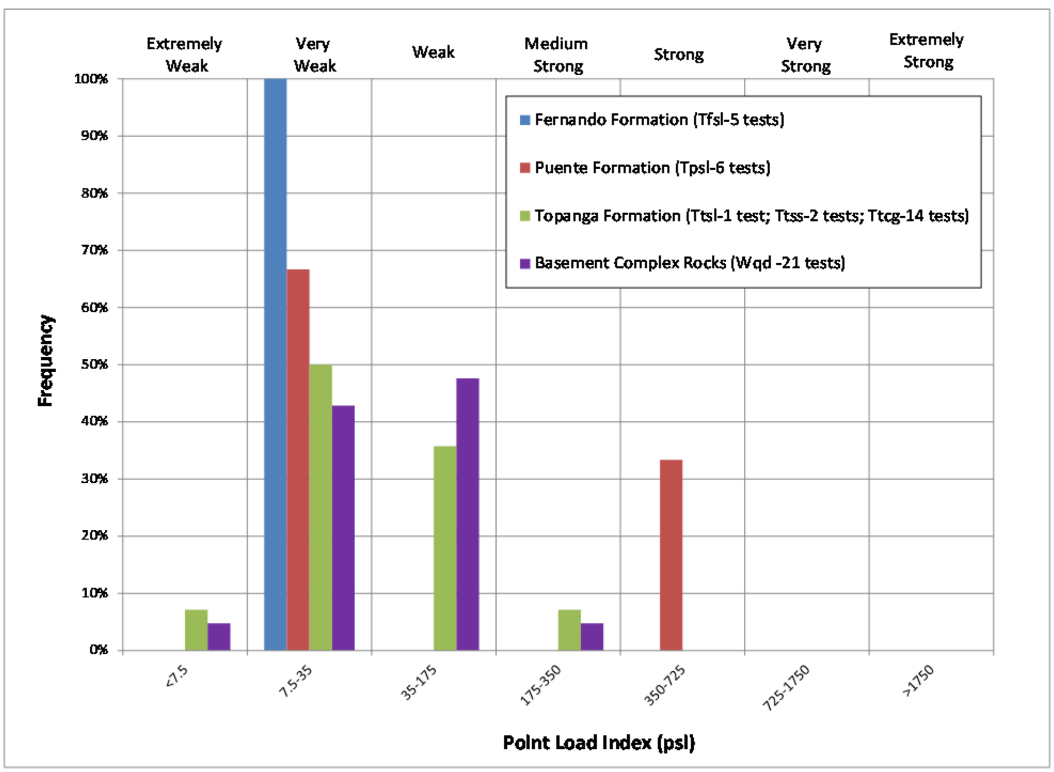
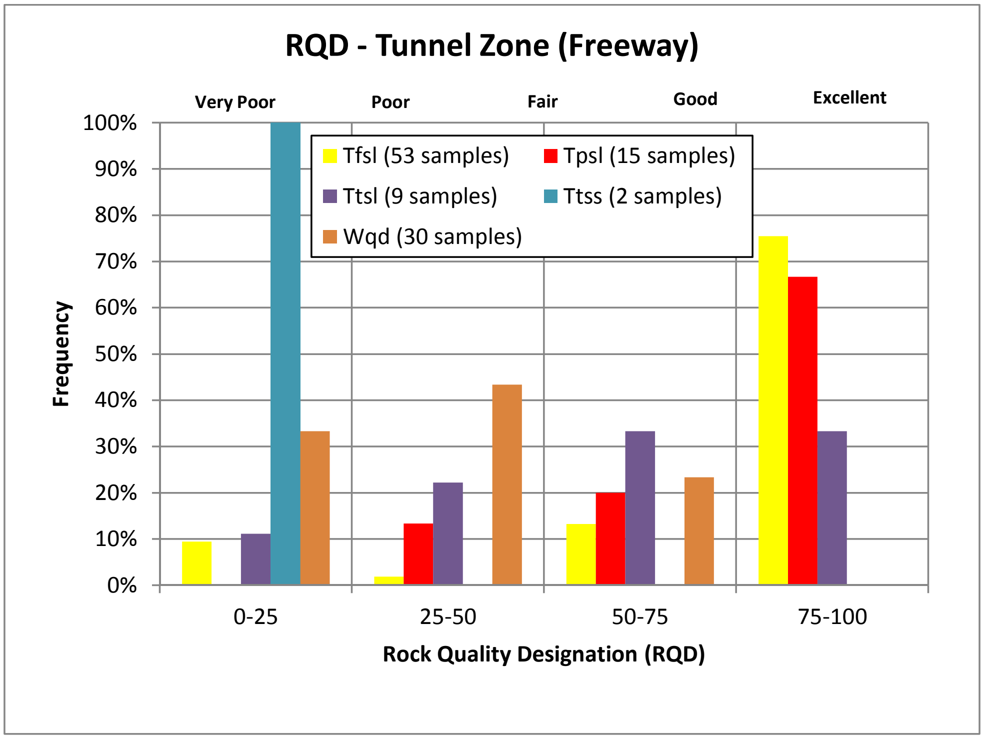
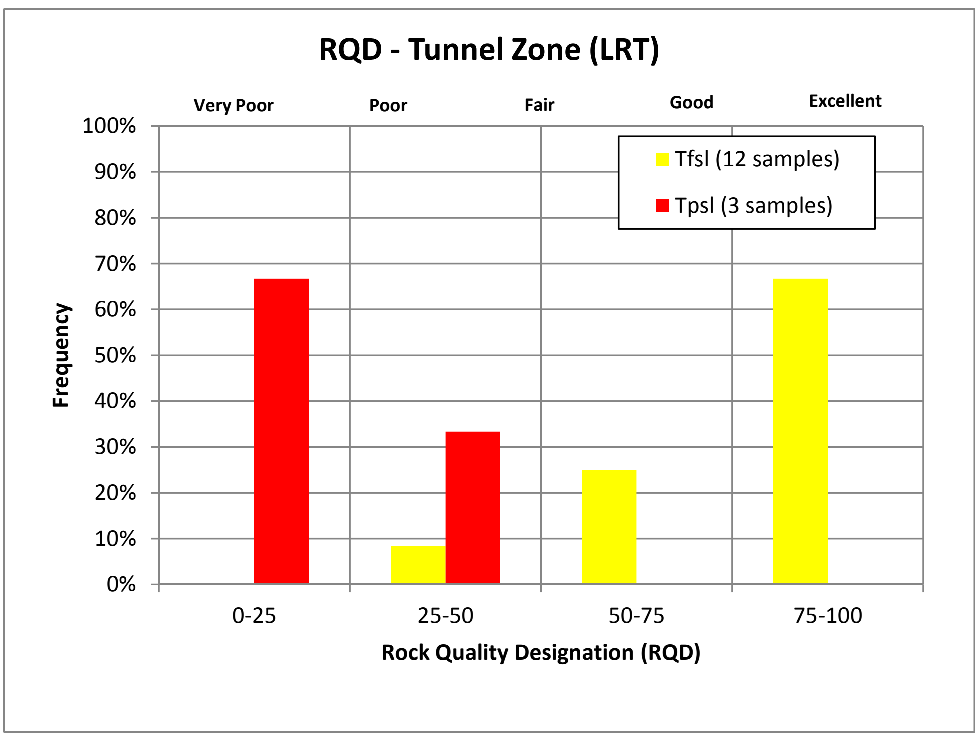


Figure 13: Histogram of PLI for Various Rock Formations



Note: A significant portion of the cores included in the RQD calculation do not necessarily meet the “sound core” definition provided in the standard test method for RQD (ASTM D6032), so care should be taken when using these RQD values to evaluate the overall rock mass quality.

Figure 14: Histogram of RQD for Various Rock Formations within the Tunnel Zone of the Freeway Alternative



Note: A significant portion of the cores included in the RQD calculation do not necessarily meet the “sound core” definition provided in the standard test method for RQD (ASTM D6032), so care should be taken when using these RQD values to evaluate the overall rock mass quality.

Figure 15: Histogram of RQD for Various Rock Formations within the Tunnel Zone of the LRT Alternative

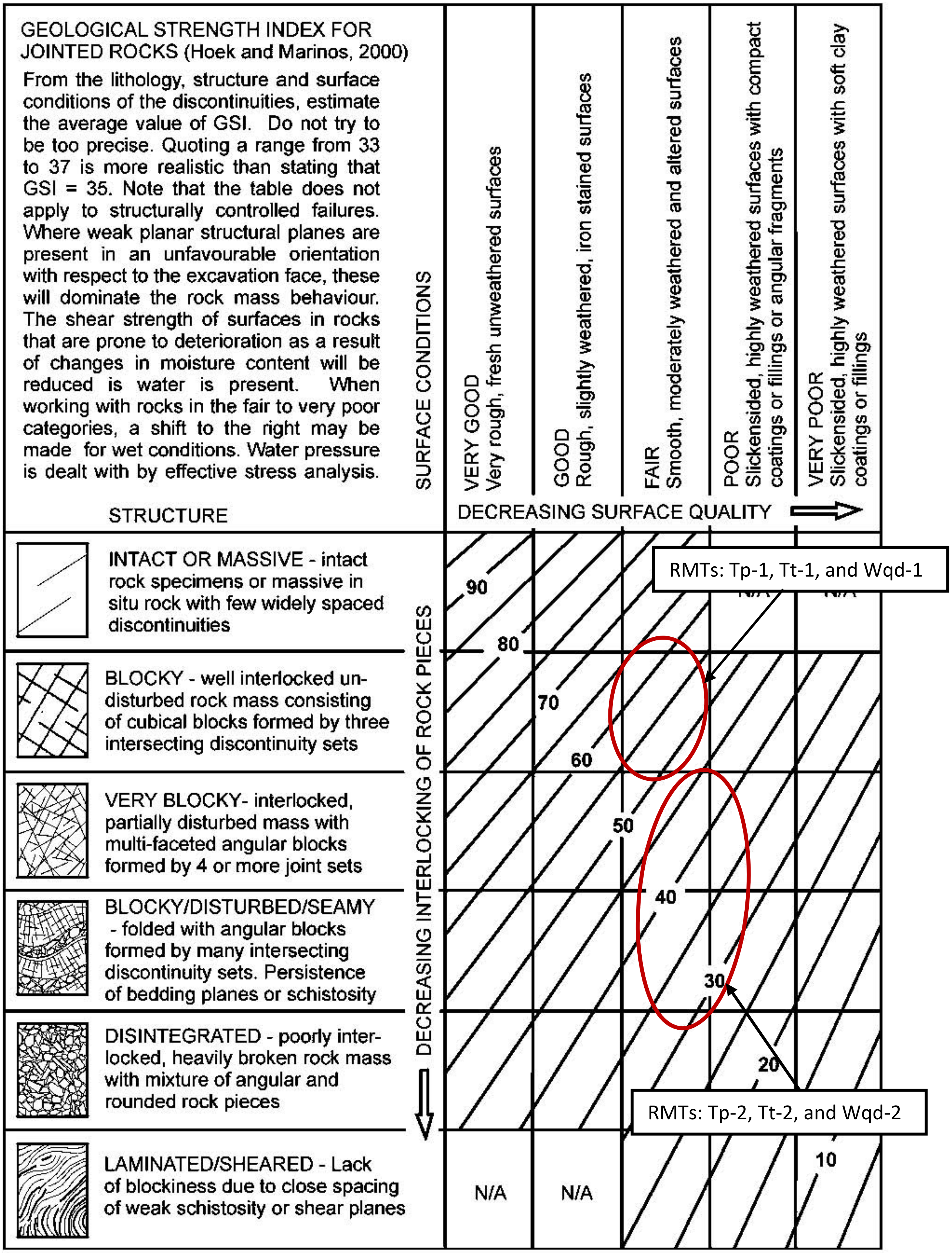
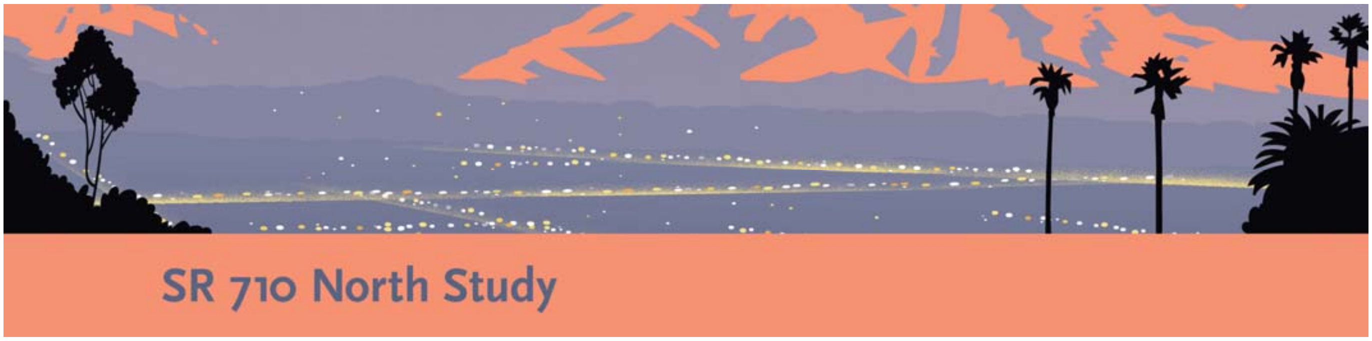


Figure 16: GSI System Explanation with Ranges for Various Rock Mass Types (Modified after Marinos et al., 2005) (Refer to Table 10 for definitions of the Rock Mass Types, RMTs)

Appendix C
TM-3 Tunnel Excavation Methods



TECHNICAL MEMORANDUM 3

Tunnel Excavation Methods

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) discusses excavation methods for the running tunnels and cross passages associated with the Freeway Tunnel and LRT Alternatives that are being evaluated. Excavation of the freeway and LRT running tunnels are expected to be mined with tunnel boring machines (TBMs) and the cross passages would be excavated using the sequential excavation method (SEM). This TM details these excavation methods and explains why they are suitable for these alternatives.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Tunnel Alternative Description

Three project alternatives are evaluated in this TM: Freeway Tunnel Alternative (single- and twin-bore variations) and the bored tunnel portions of the LRT Alternative. These alternatives are briefly described below.

2.1 Freeway Tunnel Alternative

The single-bore and twin-bore variations for the Freeway Tunnel Alternative have the same vertical and horizontal alignment. The bores are expected to have an excavated diameter in the range of approximately 60 feet (JA, 2014i). Each tunnel bore would accommodate four traffic lanes, two on the top deck and two on the bottom deck. Accordingly the single-bore alternative would provide a total of four traffic lanes (two in each direction), while the twin-bore alternative would provide eight traffic lanes (four in each direction). It is anticipated that the Freeway Tunnel Alternatives would be mined from portals at either end of the alignment, so that two TBMs would be used to mine each bore. Each tunnel drive is approximately 11,170 feet long with no intermediate shafts. The cover along the current alignment ranges from approximately 40 to 280 feet, measured from the ground surface to the crown of the tunnel.

In addition to the running tunnels, the Freeway Tunnel Alternative also includes six pairs of emergency vehicle cross passages (twelve total) along the twin-bore variation to connect the two tunnels, which are located approximately one tunnel diameter apart. These cross passages, which are approximately circular in shape and about 29 feet in diameter, would be excavated using Sequential Excavation Methods (SEM)[1].

Each pair of cross passages consists of one cross passage for the upper level roadway and one for the lower level. To accommodate the turning radius of emergency vehicles, the cross passages would intersect with both the north- and southbound running tunnels at an angle of about 50 degrees; however this should be revisited in future phases of this study (JA, 2014i). Cross sections of the running tunnels and cross passages, as well as a plan showing the interface of the running tunnels and cross passages are shown in Figures 1, 2, and 3, respectively.

2.2 LRT Alternative

The LRT Alternative includes approximately 23,000 feet of twin-bore tunnel that is expected to be excavated with two TBMs from a portal at the south end of the alignment. The excavated diameter of the bored tunnels is expected to be approximately 22 feet (JA, 2014i). The LRT Alternative also includes four underground stations that would be excavated using cut-and-cover techniques, and it is assumed that these would be excavated in advance of the TBM arrival at each location. Along the alignment, the TBMs would break into the south end of each station, be walked through the station excavation, and recommence tunneling at the north end of the station.

[1] In the SEM, tunnel excavation and support is typically performed in a series of drifts, depending on the anticipated ground conditions, which are sequenced to develop successively larger openings until the design profile is achieved.

In addition to the bored tunnels of the LRT Alternative, twenty-six pedestrian cross passages would be excavated along the alignment to connect the two tunnels, which are located approximately one tunnel diameter apart. These cross passages, which are oval-shaped and approximately 12 feet wide by 14 feet high, would be excavated using SEM and would be used for emergency egress only. Cross sections of the running tunnels and the cross passages, as well as a plan showing the interface of the running tunnels and cross passages, are shown in Figures 4, 5, and 6, respectively.

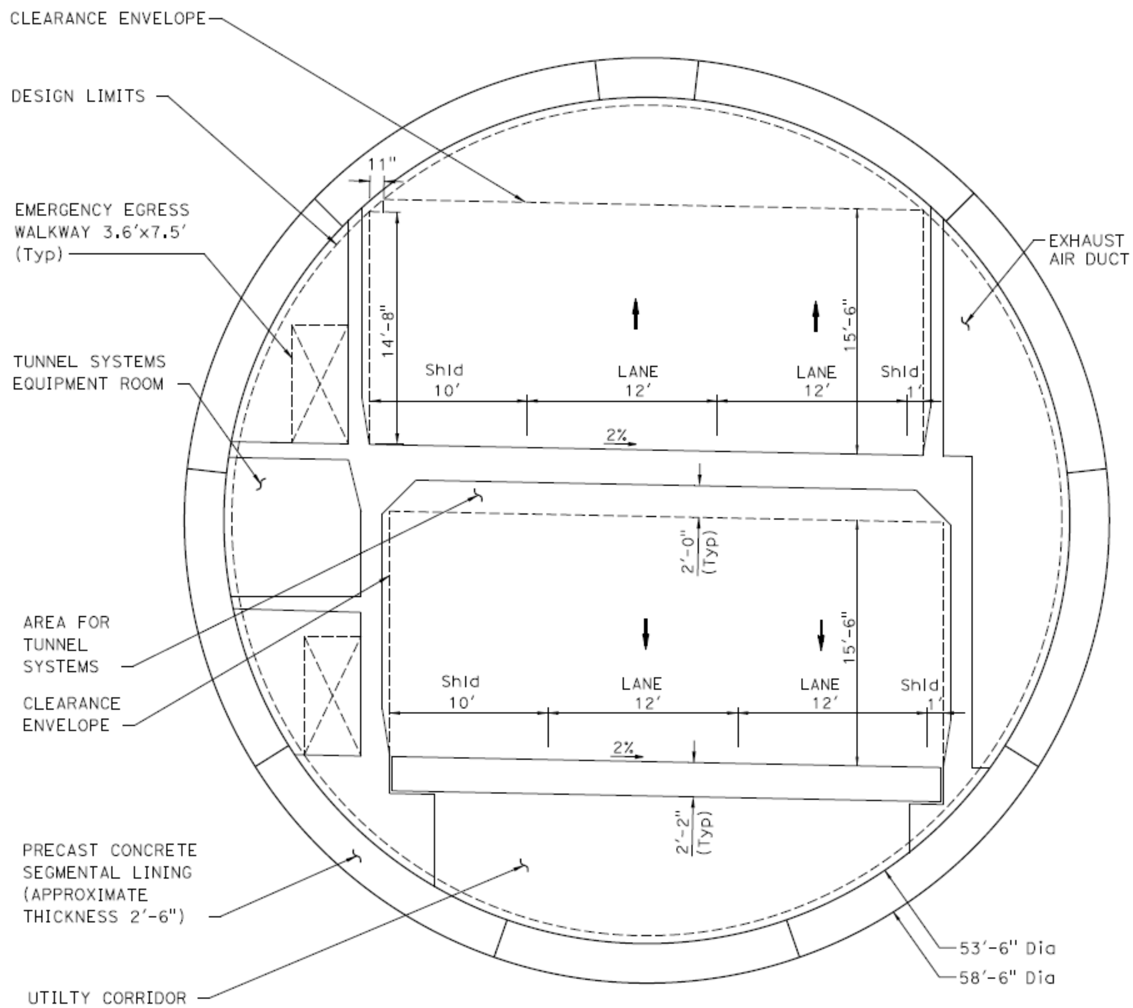


Figure 1: Cross Section of Freeway Bored Tunnel

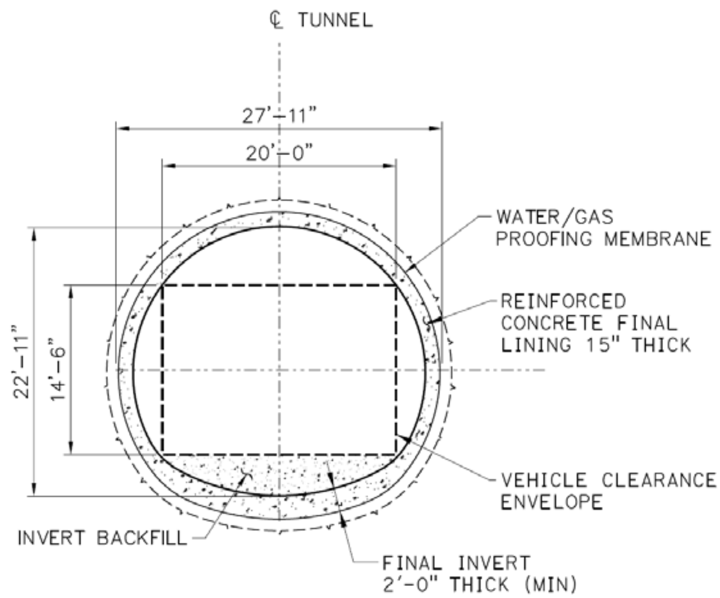


Figure 2: Cross Section of Cross Passage for Freeway Twin-Bore Tunnel

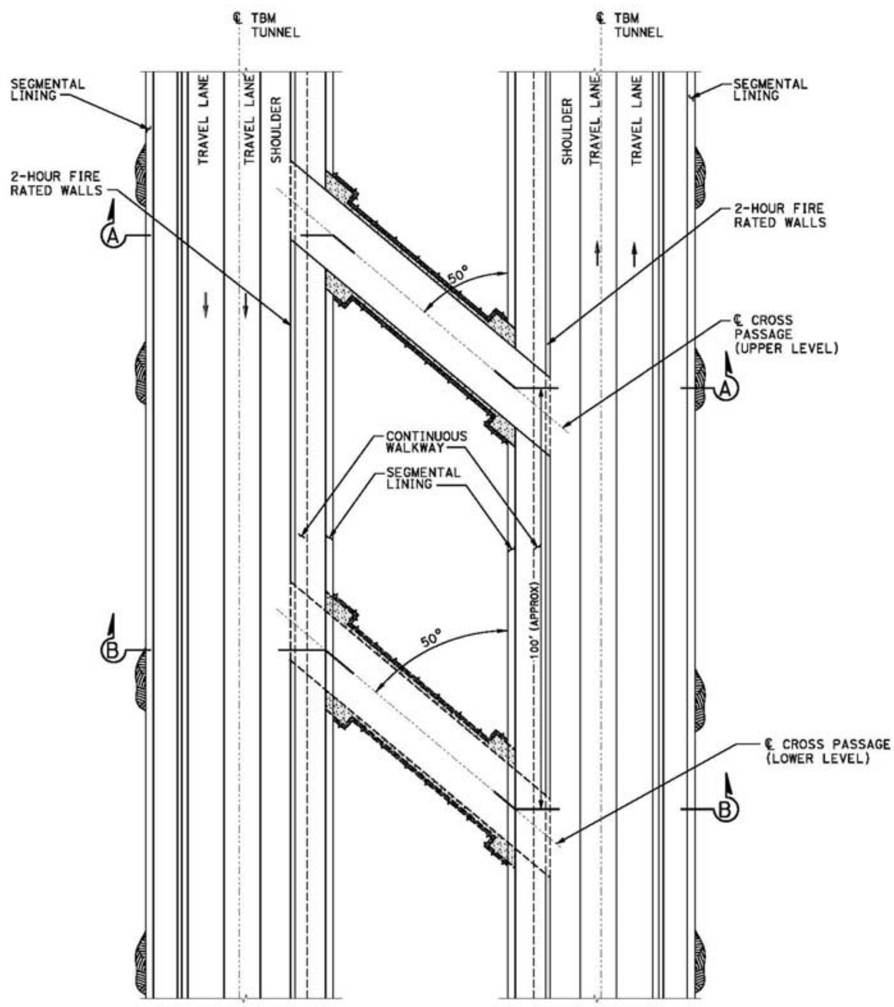


Figure 3. Typical Plan of Cross Passages for Freeway Twin-Bore Tunnel

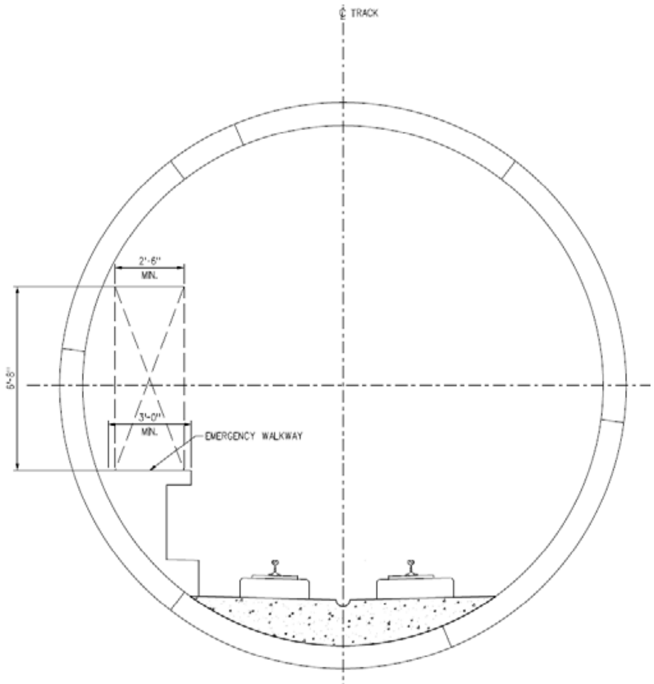


Figure 4. Cross Section of LRT Bored Tunnel

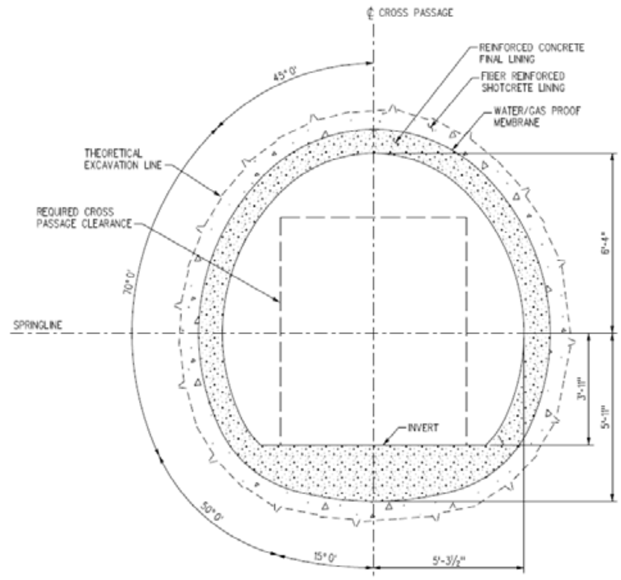


Figure 5. Cross Section of Cross Passage for LRT Alternative

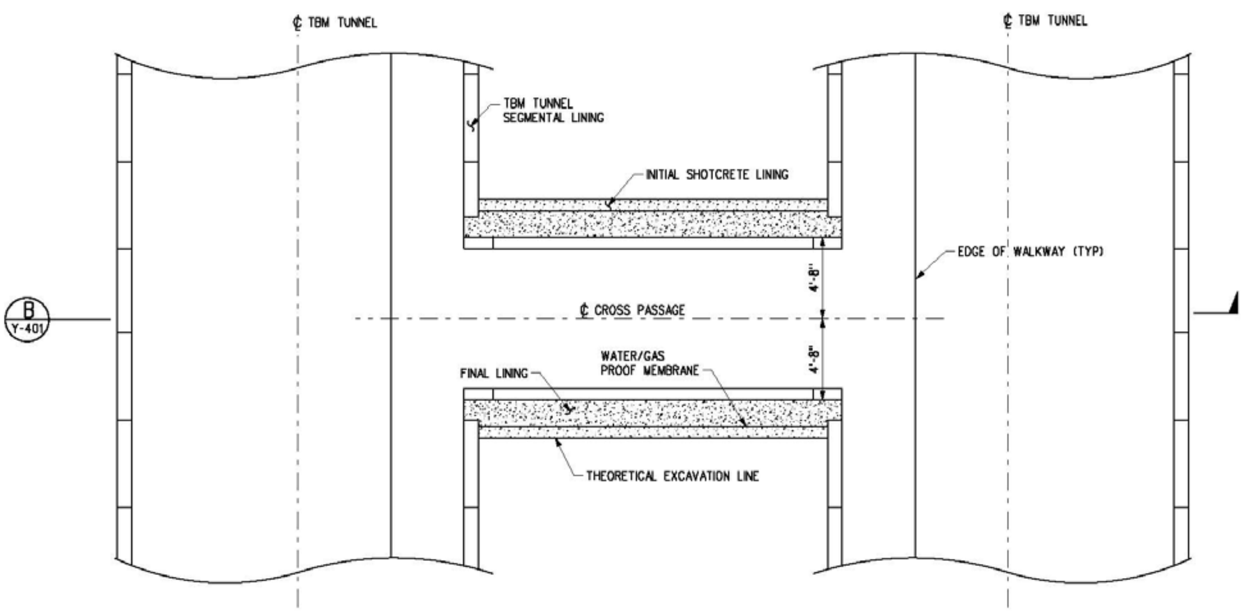


Figure 6. Typical Plan of Cross Passage for LRT Alternative

3 Anticipated Geologic Conditions

Anticipated geotechnical conditions were evaluated based on geologic data contained in the geotechnical report prepared by CH2M HILL (2014). A generalized geologic profile along the freeway and LRT tunnel alignments are shown in Attachment A. Detailed discussions of the geotechnical conditions along the tunnel alignments are summarized in the memorandum titled *Tunnel Ground Characterization* (JA, 2014a).

3.1 Geology

The geologic conditions along the tunnel alignments generally consist of Quaternary-age alluvium, weak Tertiary-age sedimentary rock formations (Fernando, Puente, and Topanga Formations), and stronger crystalline basement complex rocks.

As seen in the profiles in Attachment A, the freeway tunnels wouldbe excavated in the sedimentary rock formations, the alluvium and basement rock. The LRT tunnel alignment is closer to the ground surface and expected to vary between alluvium and the weak sedimentary rock formations. Given the current vertical profile of the LRT alternative, it is also possible that mixed-face conditions (i.e., both rock and soil in the excavation face) could occur along the LRT alternative alignment as the vertical tunnel profile is, at many locations, near the depth of the estimated contact between the alluvium and sedimentary formations. The alignments of all tunnel alternatives should be optimized when additional geotechnical information becomes available, especially to avoid long periods of mixed-face conditions.

3.2 Faulting

One active and two potentially active faults cross the tunnel alternatives: the Raymond fault is considered an active fault, while the Eagle Rock and San Rafael faults are designated as potentially active. For planning purposes these latter two faults are also being treated as active faults (CH2M HILL2014) during this study. The Freeway alternative crosses all three faults, and the LRT alternative crosses the Raymond and the San Rafael faults. The approximate locations of these fault zones are shown in the profiles in Attachment A. Where the tunnels cross the faults, special measures are planned, as discussed in *Preliminary Design Concepts for Fault Crossings* (JA, 2014g), to accommodate the potential fault movement. Additionally, it is possible that squeezing or swelling ground may be encountered in the fault zones.

3.3 Groundwater

Portions of the tunnels for both the Freeway and LRT alternatives are expected to be excavated below the groundwater table. Groundwater depths along the alignments range from as shallow as 10 feet to as deep as 175 feet below the ground surface. Water inflows into the tunnels could occur while excavating below the groundwater table in the saturated alluvium. The rock formations along the alignment are generally considered non-water-bearing; however, seepage may occur within sandstone beds and faulted and fractured zones. The Raymond Fault is a known groundwater barrier in the location of the alignments. At the Raymond Fault, groundwater levels on the north side are significantly higher than the levels on the south side.

In addition to the potential for tunneling effects on groundwater levels, it is well known that groundwater conditions can adversely affect tunnel construction. Excavation in saturated materials increases the potential for instability of the ground at the face of the excavation, resulting in loss of ground and surface settlement. Specialized TBMs have been developed to control groundwater inflows, balance groundwater pressures, and prevent instability of the tunnel face. The design of specialized machines and their operations becomes more complex as the groundwater head increases.

4 Overview of Tunnel Excavation Methods

The method of excavation for tunnels is largely governed by the size and length of the tunnel and ground and groundwater conditions. Typically for long tunnels (more than about 5,000 feet or so), a TBM in conjunction with a precast concrete lining system is the most cost effective approach. Shorter tunnel excavations such as cross passages or utility chambers, are typically constructed using conventional hand-mining methods, sometimes utilizing a SEM approach. This TM addresses the tunneling methods for both the running tunnels and the cross passages of the Freeway Tunnel and LRT Alternatives; this TM does not address cut-and-cover portions of the tunnel. Additionally, excavations associated with the portals and LRT stations are not discussed in this TM; refer to *Preliminary Design Concepts for* the *Freeway Portal Excavation and Support Systems* (JA, 2014b) and *Preliminary Design Concepts for* the *LRT Portal and Station Excavation and Support Systems* (JA, 2014c) for details.

4.1 Running Tunnels

Where ground conditions are appropriate, TBM excavation methods are generally more attractive for long tunnel drives because of the higher advance rates, as compared to conventional methods or the SEM. Because of the length and size of the running tunnels for the Freeway Tunnel and LRT Alternatives, a TBM is considered most likely approach for these tunnels.

A pressurized-face TBM is ideally suited for this project due to the potential of high groundwater pressures combined with the varying permeability and strength of the soil units, including mixed-face conditions, along the proposed alignments. The two most common pressurized-face excavation methods are slurry and earth pressure balance (EPB) TBMs. A large factor that helps differentiate which type of TBM to select is the geotechnical conditions expected along the alignment. EPB and slurry TBM methods are discussed further below.

4.2 Cross Passages

Based on the expected ground conditions and cross passage lengths and configurations, the SEM is considered the most appropriate approach for excavation and support of the cross passages for both the twin-bore variation of the Freeway Tunnel Alternative and the LRT Alternative. This method, also known as the New Austrian Tunneling Method (NATM), offers flexibility in geometry such that it can accommodate almost any size of opening. The method is employed in hard rock using drill-and-blast excavation techniques, medium hard and soft rock using roadheaders, and soft ground using backhoe excavation. The typical tunnel cross sections for SEM include elliptical or modified horseshoe-shaped configurations, or circular to promote smooth stress redistribution in the ground around the excavation. This method would require ground treatment in weaker ground and where groundwater is present.

5 Tunnel Boring Machine Excavation Methods

5.1 TBM Selection

There are two basic types of pressurized-face TBMs that could be employed for the running tunnels; earth pressure balance (EPB) and slurry machines. A third type of machine is a dual-mode EPB/slurry machine which can be converted to operate in either mode, depending on the ground conditions. The choice between slurry or EPB excavation methods is influenced by several factors, including grain size distribution; strength; ground permeability; occurrence of boulders or other obstructions, hazardous gases, and contaminants; and muck disposal considerations.

5.1.1 Earth Pressure Balance TBM. In the EPB TBM method, earth pressures at the face are a result of hydrostatic and active earth pressures. A screw conveyor is connected to a cutting chamber (earth plenum), and by synchronizing the TBM advance rate and the screw conveyor extraction rate, a pressure is built up in the chamber and maintained to counterbalance the external earth and hydrostatic pressures at tunnel face. A schematic diagram of an EPB TBM is shown in Figure 7. Excavated spoils are discharged onto a conveyor belt through a slide gate at the rear of the screw auger and then into muck cars. Slide gates may be located along the screw conveyor to remove obstructions such as rock clasts or cobbles. EPB TBMs were used successfully with the Metro Gold Line Eastside Extension project, and an EPB TBM is currently being used to excavate the Alaskan Way Viaduct in Seattle, Washington, which has a diameter of similar magnitude to that of the proposed Freeway alternative.

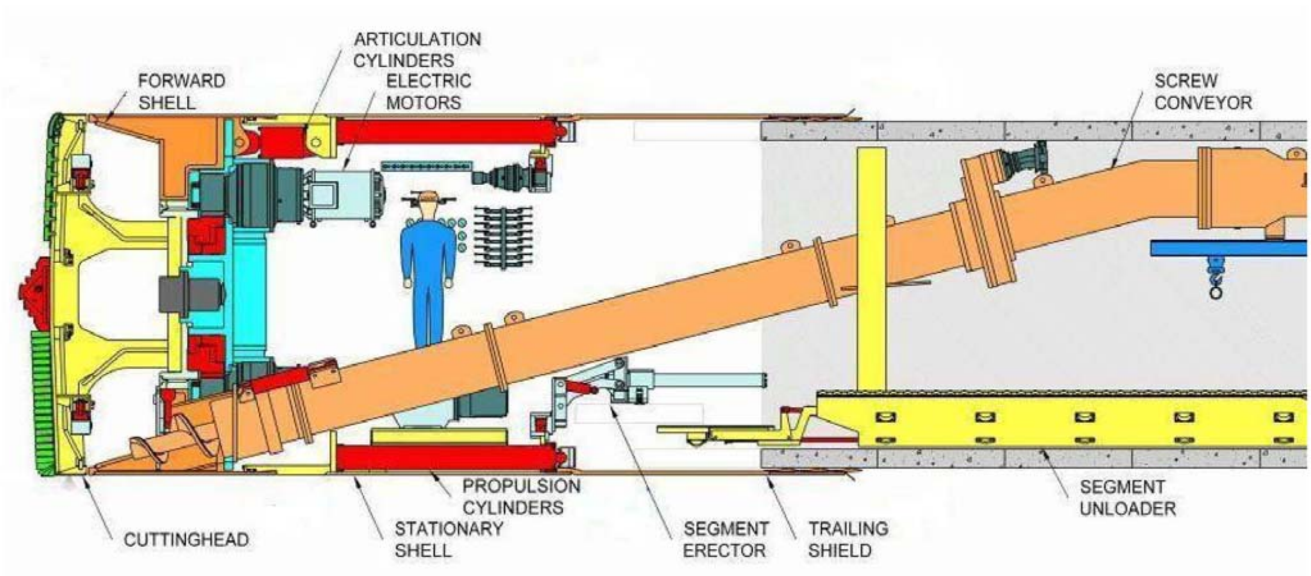


Figure 7. EPB TBM Layout

5.1.2 Slurry Pressure Balance TBM. Slurry TBMs rely on an hydraulic pressure created by a recirculating bentonite slurry that applies a positive pressure to the tunnel face, counterbalancing the external earth and groundwater pressures at the tunnel face. This is achieved by a filter cake, or "impermeable" membrane, that forms on the tunnel face as excavation proceeds. In slurry tunneling, the use of bentonite can be minimized or omitted if the ground contains adequate clay-sized particles. The excavated material is suspended in the slurry and pumped through a closed piping system to a slurry separation plant at the ground surface, where the suspended material is removed from the slurry. The muck removed at the separation plant is disposed of, while the slurry is reconditioned and pumped back to the tunnel face. In addition to counterbalancing the external pressures at tunnel face, the slurry also helps lubricate the cutterhead, reduce cutting tool abrasion, and make spoils inert for ease of solid removal. Slurry TBMs are not as common in the U.S. as EPB machines, although they have been used to excavate tunnels close to the size of the LRT alternative tunnels for the CSO program in Portland, Oregon and for the Brightwater Conveyance Project in King County, Washington. A longitudinal section of a slurry TBM is shown in Figure 8.

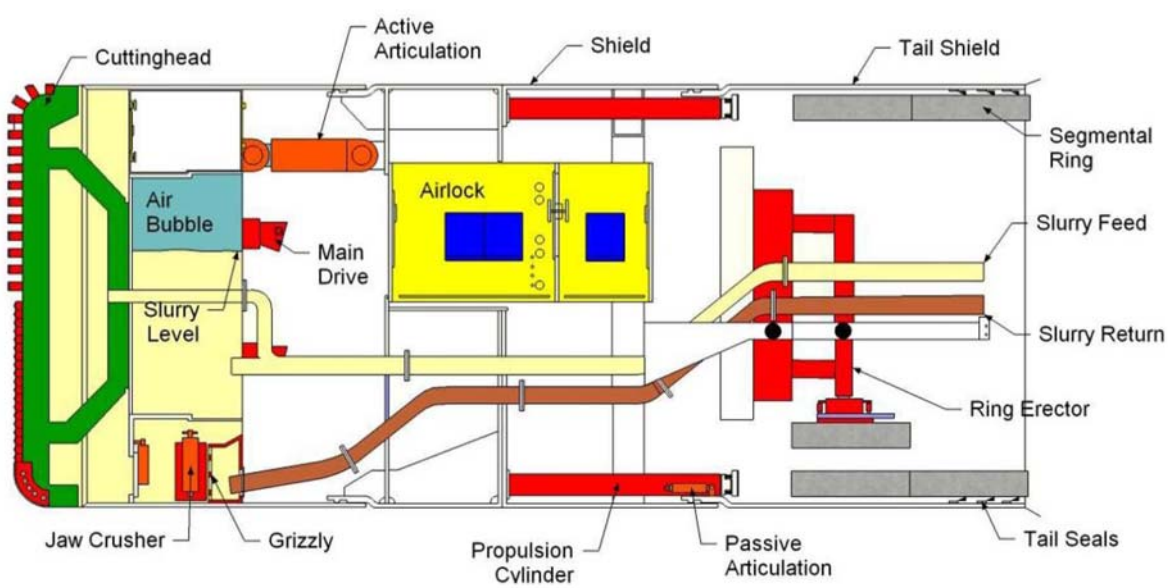


Figure 8. Slurry TBM Layout

5.1.3 Dual-Mode Machines. A third type of TBM, which is seeing increased use in the tunneling industry is a hybrid TBM. This method incorporates the best features of both the EPB and the Slurry TBMs for varying and difficult ground conditions. The hybrid TBM uses the advantages of a screw auger to remove muck from the earth plenum, incorporates the conveyance of a slurry pipe and pumping system and does this without the necessary slurry separation plant. The Port of Miami Tunnel was excavated with a dual-mode pressurized-face TBM which had an excavated diameter greater than 40 feet.

5.1.4 Applicability Based on Soil Types. The grain size distribution of the formations along the tunnel alignment is a key factor in determining which TBM type is best suited to a given project. Applicable soil conditions for EPB and slurry TBMs are presented in the shaded regions of Figure 9, which have been adapted from Langmaack (2001) and Maidl, et. al. (1996). The figure shows three generalized areas, as described below, and can be used to conceptually illustrate how grain size may affect TBM selection. In future phases of this study, more detail on the ground conditions, current ground conditioners commercially available, and other factors should be reviewed, in part, to aid in TBM selection.

The right hand side of Figure 9 (gray area) represents fine-grained and generally cohesive materials, which are best suited for EPB TBM technology. Within this zone are areas that would require conditioning due to adhesion or “sticky” materials, and other areas that would require only a minimum of ground conditioning. Fine-grained soils such as clayey silt and silty sand are ideally suited for EPB TBM excavation. Inadequate fines content can lead to face instability since a plug cannot be formed within the screw conveyor due to the increased permeability. When the excavated material contains few fines, bentonite, ground limestone, and/or hydrophilic polymers can be injected into to the excavation chamber to improve the consistency and lower the material permeability.

The central portion (brown area) of the figure identifies an area that is a common operating area for both technologies in terms of grain size. It should be noted that more conditioning is required for the EPB technology as the curves approach the slurry area due to the lack of fines, requiring greater volumes of ground conditioners to provide a plastic enough muck consistency to form a suitable plug that can balance the external groundwater pressure. Conversely, when using a slurry TBM close to the limit of the EPB area, the slurry will require conditioning agents such as anti-plugging dispersant agents due to the presence of fine clay and silt materials.

Finally, the area on the left hand side of the figure (tan area) would be suited for slurry TBMs. In the slurry area, polymer conditioners may be introduced to prevent the slurry migration into more permeable coarser-grained material. In practice, slurry TBMs are best suited for the excavation of cohesionless sands and gravels below the groundwater table. The slurry TBM works most effectively in these types of soils because the hydrostatic pressure is balanced by the formation of a “cake” to help form a hydraulic gradient between the hydrostatic pressure in the ground and the slurry pressure in the cutterhead chamber. Additionally, less complex separation methods can be used in the slurry separation plant, as coarse-grained soils are easily separated from the slurry using mechanical shakers, screens, and cyclones.

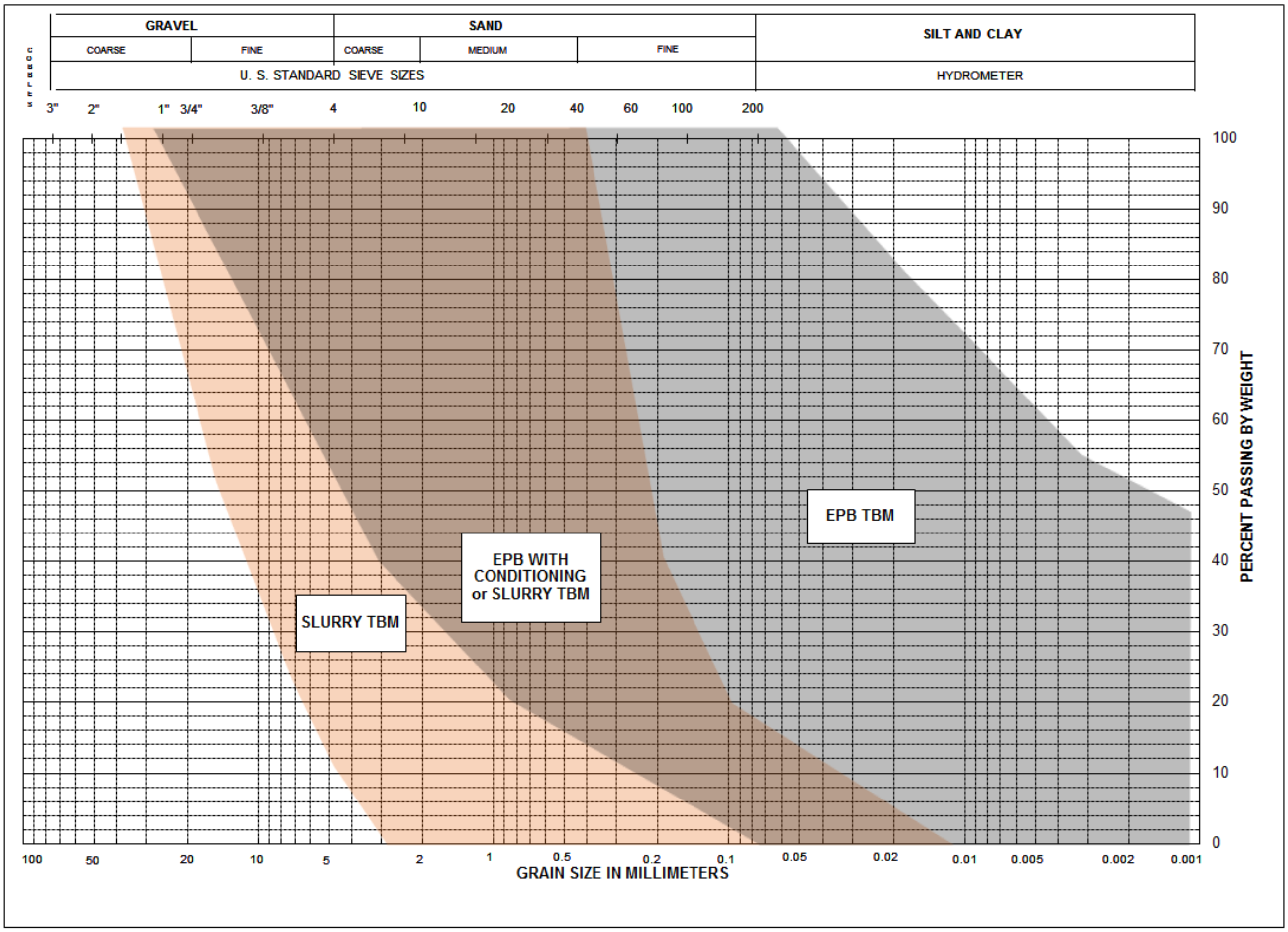


Figure 9. Example of Applicability of TBM Type Based on Soil Conditions

5.2 Cutterhead Configuration

For both and EPB and slurry TBM methods, the tunnel is excavated with a bi-rotational cutterhead equipped with cutting tools to remove the ground at the tunnel face and draw the loosened material into the cutterhead. Typically, disc cutters are used in TBMs to excavate hard rock, and ripper-style tools are used to cut through soil. The disc cutters score the rock on the excavation face as the cutterhead turns, cracks eventually form, and the rock chips away.

Additionally, scraper and bucket tools (i.e., drag teeth) are configured in either case to gather the cuttings and direct them toward the openings in the cutterhead for removal. Figure 10 shows a TBM from the Brightwater Tunnels in King County, Washington, which has ripper tools on the cutterhead, and Figure 11 shows the TBMs from the Arrowhead Tunnels in San Bernardino County, California, which were outfitted with disc cutters.

A cutterhead can be designed to handle both soft ground and hard rock and is known as a “mixed ground” cutterhead. A mixed ground cutterhead is a compromise, with tooling and opening configurations that can be modified for the majority of ground expected while permitting mining through all of the ground types on a particular tunnel alignment. This is done by equipping the cutterhead with flexible back loading saddles or cutter boxes, which permit the use of both disc cutters and ripper-style tools. Additionally, TBMs can be designed to have both disc cutters and ripper-style tools simultaneously.

Generally, disc cutters are used when the unconfined compressive strength (UCS) of the anticipated material exceeds approximately 5 to 7 ksi; otherwise, rippers should be sufficient. Based on preliminary data presented in *Tunnel Ground Characterization* (JA, 2014a), it is anticipated that while the maximum UCS values for some of the rock formations may exceed 7 ksi, the average UCS values are well below this range (see Figure 13). For this project, due to the mixed-face and rock conditions that may be encountered in both the Freeway and LRT alternatives, the cutterhead would need to be designed for soil, sedimentary rock with inclusions of harder and stronger material, and also, for the Freeway tunnels only, harder bedrock.

When discussing maintenance of the TBM cutterhead it is useful to discriminate between primary and secondary wear. Primary wear is the wear expected on the replaceable cutting tools, while secondary wear is the wear on the supporting structures of the cutterhead, which can lead to major overhauls underground. When soil abrasivity is identified as a risk during the design phase, the TBM specifications should include a provision for wear protection on the cutterhead surfaces. Before tunneling operations begin, thorough ground conditioning tests should be carried out on samples of the expected soil types to optimize conditioner application with the goal of reducing cutterhead torque and abrasion and thereby extending cutter tool life. During tunnel excavation, control of primary wear and minimization of secondary wear should be addressed by regular inspection of the cutterhead and especially the primary wear elements like the cutting tools.

Principal factors affecting primary and secondary wear are:

- The nature of the soil and rock, including its abrasiveness and stickiness;
- Face confinement or support pressure;
- The type and style of cutting tools chosen;
- The opening area and the geometry of the openings on the cutterhead;
- The wear plates and hard facing fitted to the cutterhead; and
- Ground conditioning and the volume and type of ground conditioning agent used.

Typically, when the cutter tools need to be changed, an intervention is required to perform the repair or maintenance to the cutterhead. Hyperbaric interventions are typically required for pressurized-face TBMs because they are carried out by workers directly behind the cutterhead in the excavation chamber which is subjected to the pressurized environment, which could be as high as 6 bar in the Freeway alternative depending on the location of the groundwater table. These interventions slow down the excavation process. Depending on the ground and groundwater conditions, these interventions could be performed in free air.

Additionally, because of the expected TBM size for the Freeway Tunnel Alternative, it could be possible to perform cutting tool changes from within the spokes of the TBM cutterhead under free air. The TBM currently mining the SR-99 Tunnel in Seattle, Washington, has been designed so that the majority of the cutting equipment can be changed in this fashion. Another way to perform an intervention under free air would be to reduce the pressure at the face of the excavation by dewatering or ground improvement. It’s possible that this can be performed in conjunction with the ground improvement necessary for cross passage excavation depending on the location of the intervention along the alignment.

Figure 10. Brightwater TBM with Ripper-Style Cutting Tools for Soil or Soft Ground Conditions

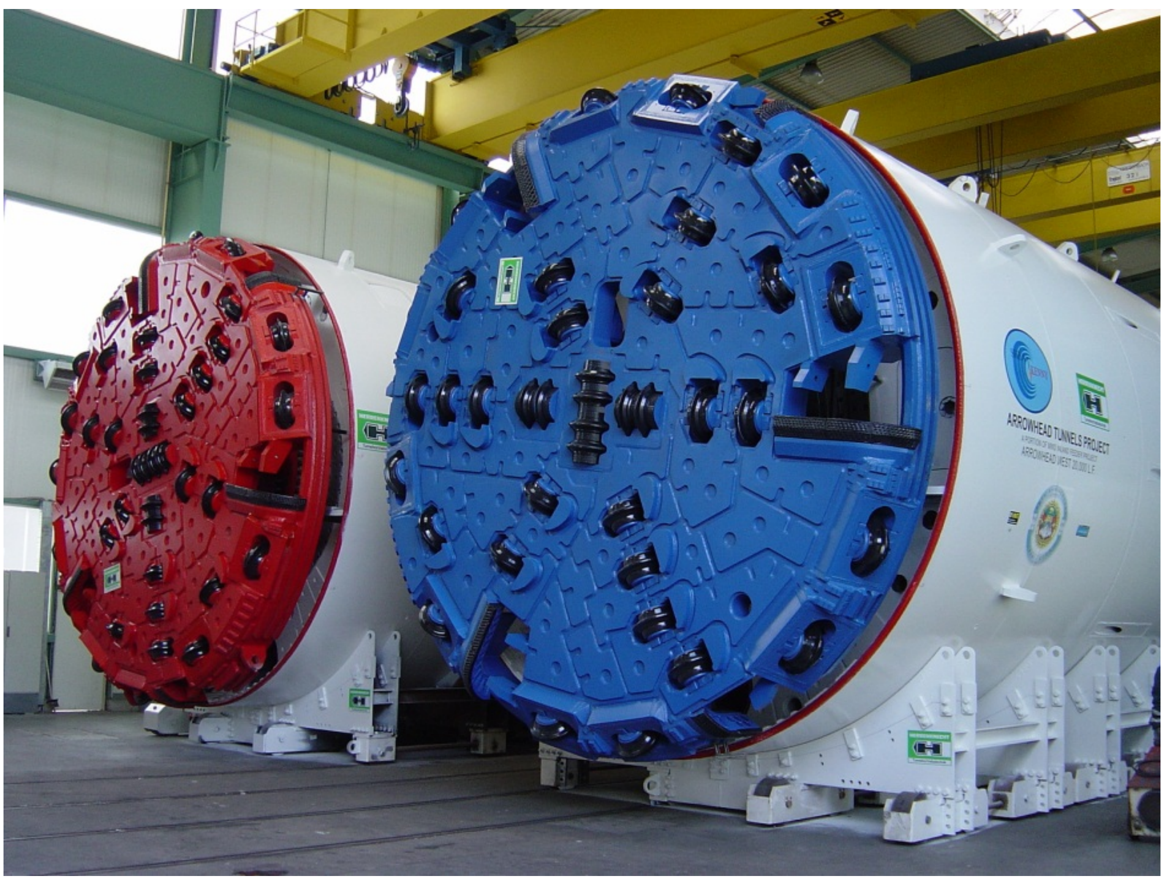


Figure 11. Arrowhead Tunnels TBMs with Disc Cutters for Hard Rock

5.3 Groundwater Control Measures

Excavation of the tunnels along the majority of the Freeway Tunnel Alternative is expected to be below the groundwater table, while only some portions of the LRT Alternative would be excavated below the groundwater table, as can be seen in the geologic profiles in Attachment A. Groundwater inflows are not limited to the saturated alluvium; most of the rock along the alignment has relatively low permeability, but there are zones of higher permeability including sandstone beds and fractured zones. In addition, groundwater can accumulate adjacent to a fault which can act as a groundwater barrier (CH2M HILL, 2014).

For pressurized-face TBMs, groundwater pressures can be balanced by face pressures, significantly limiting or completely eliminating groundwater inflows. Installed watertight segmental linings can be designed so that inflows into the completed tunnel are near zero.

5.4 Ground Loss and Settlement Control Measures

Excavation of the tunnels in soft or weak ground is a potential source of ground loss and surface ground movements, but current tunneling methods are able to limit ground loss and resulting settlement to very small values. One of the key requirements for the TBM tunnel excavation is to limit the ground loss. Use of a TBM with pressurized face can meet these requirements by controlling face pressure and contact grouting pressure as well as injecting bentonite along the shield so that the ground loss and resulting surface settlements are minimized. Further discussion of ground control during tunneling and ground movements resulting from tunneling can be found in *Evaluation and Control of Ground Movements* (JA, 2014d).

5.5 Squeezing Ground

Squeezing ground behavior occurs when ground slowly converges into the tunnel excavation with time. The pressurized-face TBM handles squeezing ground conditions better than other methods since face pressures are balanced, preventing/minimizing the soil at the face and around the shield from squeezing. The main area of concern is along the body of the TBM where high friction forces caused by the convergence of the ground can result in the TBM becoming trapped. In order to overcome the large frictional forces that could be imposed on the TBM equipment, requirements and tunneling procedures must be developed and implemented.

Included amongst these are to provide for:

- a copy cutter on the cutterhead to allow for sufficient overcut,
- the capacity to inject bentonite lubricants into the annulus around the TBM shield,
- a tapered TBM shield
- the measurement of pressure along the shield, and
- continuous mining through areas identified as high risk for trapping of the TBM.

In the next phases of the project, the full extent of the risk of squeezing ground should be evaluated.

5.6 Ground Conditioning

Ground conditioning is an important aspect of soft ground tunnel construction, and the main objective is to improve TBM performance and to modify the ground to provide better control of the tunneling operation. The addition of suitable conditioning agents may be introduced at various points in the tunneling process, including: at the face of the cutterhead, within the cutterhead chamber, in the muck removal system (screw conveyor), and around the outside of the tunneling shield. Conditioning agents improve TBM performance in several ways, as summarized below:

- Improved stability of tunnel face and better control of surface settlement.
- Improved flow of excavated material through the cutterhead.
- Reduced wear of cutterhead face plate and tools, and all parts of the muck removal system.
- Reduced torque and cutterhead power requirements.

- Reduced friction and heat buildup along the TBM shield.
- Improved handling of excavated material because it is formed into a suitably plastic-like mass.

It is far more cost effective to treat the soil being excavated with appropriate conditioners than to resort to time-consuming changes to equipment during tunnel excavation. In general, soil conditioning is required for EPB TBMs in all ground types.

For a Slurry TBM, bentonite is mixed with water to create a slurry that is used to support the tunnel face and also to transport the excavated material to the ground surface. The excavated material is mixed with the slurry in the chamber and suspended in the slurry at a consistency that allows removal from the chamber by pumping. The pressurized slurry exerts a hydraulic pressure at the tunnel face and forms a mud cake to stabilize and seal the tunnel face. This is similar to the use of drilling muds to stabilize deep boreholes. The use of bentonite may not be required in formations with enough natural clay to develop a slurry with the proper specific gravity.

In tunnel construction, soils are typically treated with foams, polymers, and/or bentonite. Depending on the soil and groundwater conditions, these agents can be used either alone or in combination with one another. Foams and polymers are generally used with EPB TBMs, and bentonite, sometimes with polymers, is used with Slurry TBMs. The type of polymer or foam should be selected based on functional requirements and would be a required submittal from the contractor. The additives used for soil conditioning in EPB and Slurry TBM operations are normally specified to be non-toxic and biodegradable. When used in the concentrations recommended by the manufacturer they have been tested and shown in practice to have little impact on the customary routine of excavated soil disposal. There are natural product lines that are widely accepted and used in the tunneling industry, and therefore the soil conditioning process is not expected to introduce contamination into the muck.

5.7 Tunnel Support Requirements

For pressurized-face TBMs, segmental linings are the typical form of initial excavation support and would also serve as the final lining. This form of support requires an additional step in the excavation cycle, but provides full and continuous support of the ground. Segmental linings with gasketed joints are essentially watertight to minimize water inflows into the tunnel and allow backfill grouting of the segments.

A key issue for segmental lining installation is backfilling of the void between the lining and the ground as the TBM advances. This backfilling both provides stability to the ring by bedding it, and bridges the gap between the lining and ground to provide intimate contact for limiting ground convergence and settlement. There are a number of methods and materials used for this backfilling, but most often a cement-based grout is injected, either through ports in the tail shields or through ports in the lining as the TBM advances, providing continuous filling of the tail void.

5.8 Preliminary Assessment of Tunnel Excavation Methods for Running Tunnels

There are several factors that can influence the decision of what type of pressurized-face TBM to use for the excavation of the running tunnels considered in this study. In Figure 12, the average grain size curves for each of the geologic units expected to be encountered along the alignments have been plotted on the shaded regions from Figure 9. While this data is preliminary and only displays the average curves, it suggests that the majority of the grain sizes collected to date plot in the EPB TBM section of the chart due to the fines content in several of the units; however, this is subject to change as more geotechnical data becomes available and because the study of the range of grain sizes for each type of TBM is a developing field. For the diameters being evaluated for both tunneled alternatives, it’s expected that it will be feasible for TBMs to excavated the running tunnels; however, further study as to the type of TBM to be used for each of the alignments will be evaluated in future phases of this study.

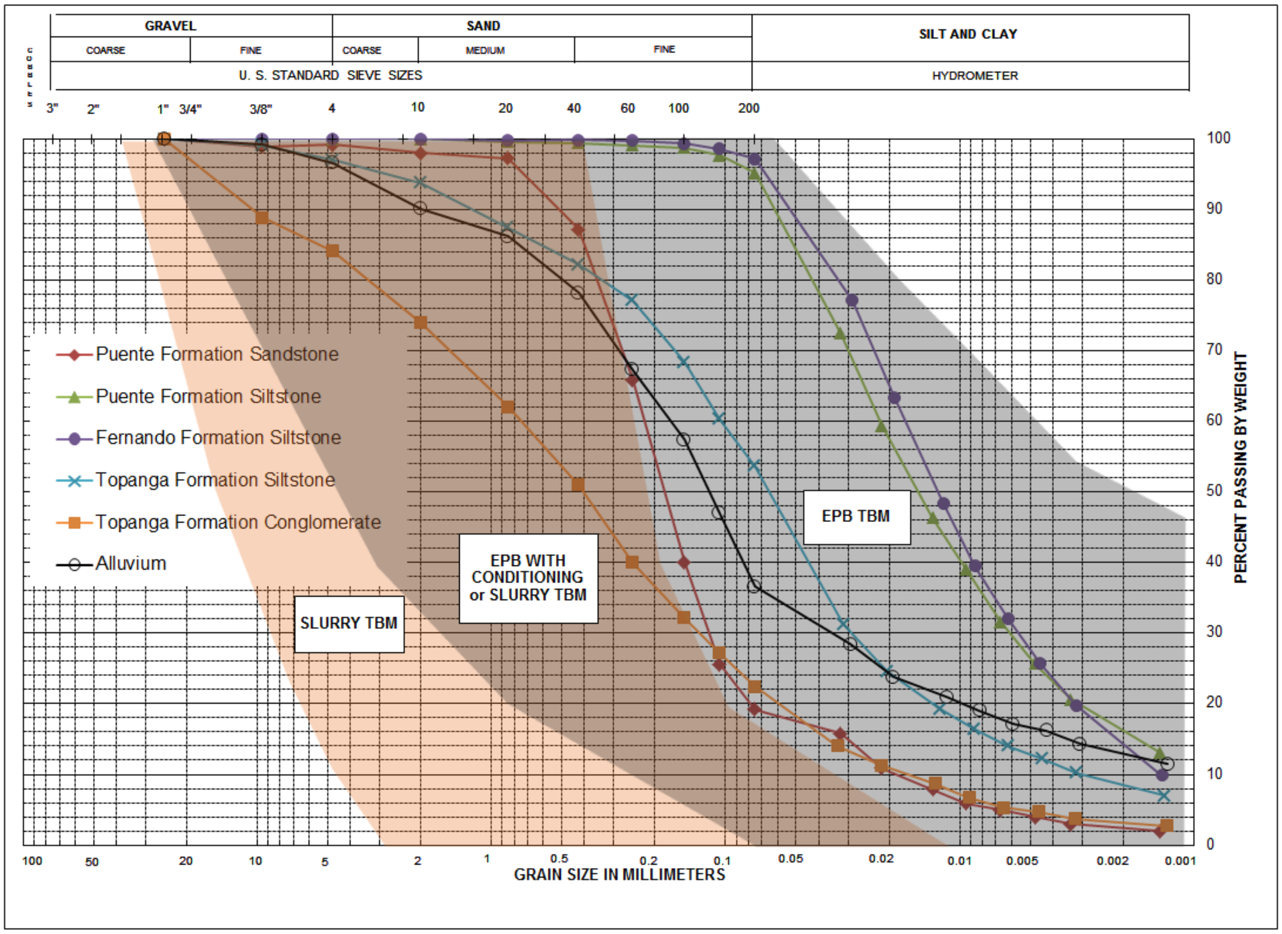


Figure 12. Preliminary Grain Size Analysis Curves, per Geologic Unit

6 Sequential Excavation Method

Cross passages along the running tunnels would be constructed using the Sequential Excavation Method (SEM), also known as the New Austrian Tunneling Method (NATM) or the Sprayed Concrete Lining (SCL) Method. With the SEM, tunnel excavation and support is typically performed in a series of drifts, depending on the anticipated ground conditions, which are sequenced to develop successively larger openings until the design profile is achieved. The SEM is flexible for tunnels in weak and variable ground conditions and considered to be the most suitable and economical method of constructing the cross passages for both the Freeway twin-bore and LRT alternatives.

6.1 Mining Methods

Depending on the rock strength, fracturing, and type, several tunnel excavation methods are applicable for an SEM construction approach. These methods include a hydraulic excavator, similar to a backhoe, with a digger paddle or cutterhead attachment; a roadheader; hydraulic impact hammers; and controlled drill-and-blast methods. For tunnel excavation in soil or soft ground, ground improvement prior to mining may also be required in order to ensure stability of the tunnel, minimize groundwater inflows, and control ground loss.

Based on the available geotechnical information (CH2M Hill, 2014 and JA, 2014a), moderately hard to weak rock and soil are the predominant ground conditions anticipated in excavation of the cross passages. Hence, it appears that conventional excavation methods using mechanical equipment would be feasible, and drill-and-blast techniques do not appear to be required for cross passage excavation and will not be discussed further in this memorandum. Mechanical excavation methods, consisting of a roadheader and hydraulic excavator or backhoe in conjunction with ground improvement measures (if required) are discussed herein and is the assumed method for cross passage excavation for the purposes of this study. Other methods, such as drill-and-blast, may also be feasible and may be explored in future phases of this study pending updated geologic studies.

6.1.1 Mechanical Excavation Methods. The effectiveness of a mechanical excavation method using either a roadheader or a backhoe depends on the rock properties (strength and fracture spacing), machine characteristics (power, weight, and size), and size of the excavation.

As detailed in *Tunnel Ground Characterization* (JA, 2014a), the unconfined compressive strength (UCS) for rock formations to be encountered during cross passage excavation ranges from extremely weak (less than 150 psi) to very strong (greater than 14,500 psi), as shown in Figure 13 and Table 1. The majority of rocks are expected to be extremely weak to weak (refer to Figure 13). Fracture spacing of these rocks varies from intensely to very slightly fractured (1 inch to over 3 feet), with the majority having a spacing of less than 3 feet. With the range of strength and fracture spacing anticipated, the excavatability and effectiveness of a machine would depend largely on rock strength and less on fracture spacing.

Roadheaders that could be used to excavate the cross passages within all rock formations anticipated include a mid-sized machine in the 50-ton class, such as the Sandvik MT300 (formerly Alpine Tunnel Miner ATM 75. See the clearance requirements discussed below for use of a 50-ton-class roadheader), shown in Figure 14. The excavation performance using a 50-ton-class roadheader varies primarily by intact rock strength. Figure 15 shows the expected cutting rates for a 50-ton-class roadheader for the Caldecott 4th Bore Tunnel Project on State Route 24 through the Berkeley Hills (Sandvik, 2009). The rocks anticipated in cross passages for the SR-710 alignment are expected to be generally weaker than the sedimentary rocks present along the Caldecott Tunnel alignment. Therefore, it is anticipated that the net cutting rate of a 50-ton-class roadheader would be greater than about 25 bank cubic yards per hour based on Figure 15, if this type of roadheader were employed.

Furthermore, rock mass defects such as fracture condition, orientation, and spacing would weaken the rock mass and thereby improve roadheader performance by either promoting “ripping” in addition to “cutting” action in intermediate quality rock masses, such as Ground Class 1, or rendering “ripping” action as the primary excavation process in poor quality rock masses such as Ground Class 2 or 3 with mixed-face conditions (see JA, 2014a for the definition of ground classes). Based on the Rock Mass Cuttability Rating (Sandvik, 2008), for rock mass with a fracture spacing of less than 8 inches, “ripping” is the dominating excavation process. As the cutting rates shown in Figure 15 can increase by a factor ranging from about 3 to 10 in rippable rock masses, it is assumed that roadheader excavation would be effective in the majority of cross passage excavation. Even in portions of the rock with fracture spacings greater than 8 inches, use of a roadheader is expected to be effective due to their relatively low UCS.

The performance of a roadheader is also affected by the wear of cutting bits, which depends on rock characteristics such as quartz content, UCS, and Cerchar Abrasivity Index (CAI). The amount of tool wear to a roadheader can be correlated to the quartz content and UCS of a rock. The CAI is a measurement of rock abrasivity and can also be used to estimate cutter wear and costs. Based on the very limited test data available, the CAI values of the rocks to be encountered in the cross passages is shown in Table 1, indicating a moderate abrasivity and suggesting that the excavation would be within the economical range (Pichler, 2010).

The size of a roadheader versus the excavation size must also be considered when assessing whether or not a roadheader is feasible. A roadheader of the 50-ton class is approximately 10.5 feet high and 11.5 feet wide. Based on manufacturer recommendations, the clearance requirements for a 50-ton weight class would be satisfied for the excavation of the freeway cross passages. However, any roadheader within this class may be too large to permit efficient excavation sequencing for excavation of the LRT cross passages, where use of an excavator or backhoe with a cutting attachment may be a more feasible and economical approach.

TABLE 1
Rock Properties Relevant to Cross Passage Excavation

Geologic Formation	UCS (psi)	Cerchar Abrasivity Index	Quartz Content (%)	Slaking Durability Index
Fernando	48 – 624	N/A	7 - 10	0.6 – 61.1
Puente	47 – 15,397	0.2 – 3.3	15 - 20	0.0 – 95.3
Topanga	11 – 16,080	N/A	N/A	0.0 – 98.0

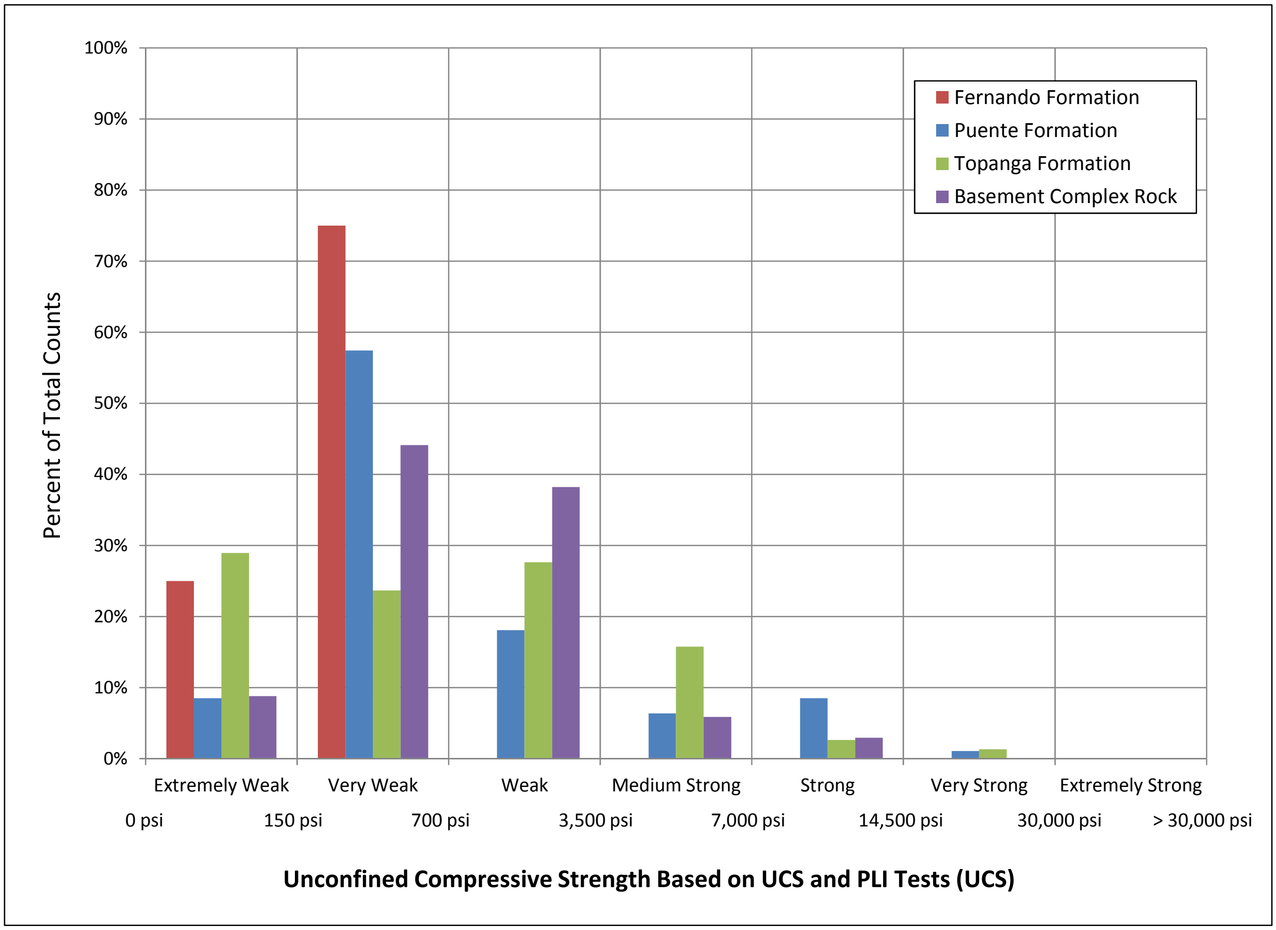


Figure 13. Unconfined Compressive Strength for Various Rock Formations

Figure 14. A 50-ton Class Roadheader Layout

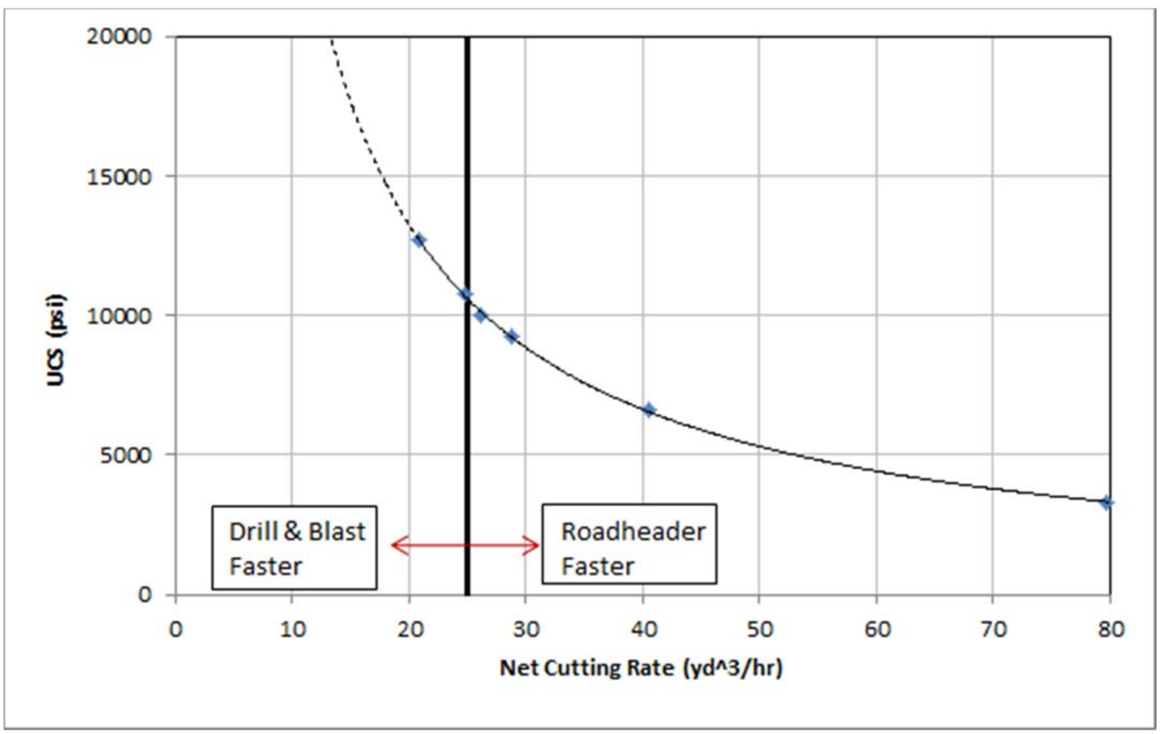


Figure 15. Net Cutting Rate of an MT-300 Roadheader (50-ton class) (Modified after Sandvik, 2009)

6.1.2 Ground Improvement. The excavation of cross passages for both alternatives is expected to encounter soil or mixed-face conditions. In these areas, low stand-up time and unstable conditions are anticipated due to the pervasiveness of raveling and running and flowing conditions, especially with the presence of groundwater. Use of systematic ground improvement measures would be required to stabilize the openings, control ground movements and groundwater inflows, and minimize ground loss and the potential for surface settlement. Feasible ground improvement measures include dewatering, permeation grouting, chemical grouting, jet grouting, or ground freezing, individually or in combination with each other.

Among six pairs of cross passages proposed for the Freeway twin-bore alternative, one pair of cross passages (CP #5) is expected to be excavated entirely within soil (see the geologic profile and vertical alignment for the

Freeway Tunnel Alternative in Attachment A). The ground cover for these two cross passages ranges from about 150 to 170 feet, with a groundwater head at the crown of about 0 to 10 feet above the excavation. Except for jet grouting due to the depths of these cross passages, all ground improvement measures mentioned above are expected to be feasible.

A total of sixteen out of twenty-six cross passages for the LRT alternative are expected to be either entirely within soil or to encounter mixed-face conditions (see the geologic profile and vertical alignment for the LRT Alternative in Attachment A). Due to relatively shallow ground cover of less than 80 feet, use of any or a combination of the ground improvement measures mentioned above is expected to be feasible.

Ground improvement for excavation of cross passages in rock may also be required in order to effectively control groundwater inflows and maintain stable openings. Any feasible ground improvement measures mentioned above for cross passages in soil or mixed face conditions are also applicable to those in rock and can be carried out within tunnels, or from the ground surface if access permits. Selection of the specific ground improvement measures to be employed should be based on the evaluation of their effectiveness for ground stabilization purposes, using criteria such as grout take, surface settlement limits, and inflow criteria as used for groundwater inflow control.

6.2 Excavation Sequence and Initial Support Requirements

Unlike the bored tunnels, cross passages would require installation of an initial support system followed by a final lining. Initial support systems for the cross passages would be required to maintain stability of the tunnel, control ground movements, and provide for safe working conditions. Excavation sequence and initial support requirements should be developed based on the anticipated ground conditions and support requirements and excavation sequences required to maintain heading stability and control ground movements.

The initial support systems are designed and selected by considering the following interrelated factors:

- Size and shape of the opening;
- Strength, physical characteristics, and behavior of the ground;
- Variability of the ground conditions at each of cross passage locations;
- Orientation, spacing, and characteristics of discontinuities in the rock mass;
- Groundwater conditions; and
- Construction factors such as the selected tunneling equipment, excavation methods, and the interface with other components of the project.

In selecting an appropriate initial support system for the cross passage, longer-term ground behaviors such as slaking and softening of the rock and potential for overbreak must also be considered, in addition to the predominant ground behaviors, as described in *Tunnel Ground Characterization* (JA, 2014a). Slaking and softening will occur in shale, siltstone, crushed sandstone, and sheared rock where slake durability is expected to be very low to low. Slaking and softening would cause instability in the tunnel invert and could be aggravated by the presence of groundwater and construction traffic.

An overview of the excavation sequence and initial support requirements for the Freeway Tunnel and LRT Alternatives are described below. Details of the evaluation of these requirements are given in *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* (JA, 2014e) and *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014f).

6.2.1 Freeway Tunnel Alternative. Ground conditions that would be encountered in various cross passages along the freeway alignment have been divided into three ground classes to assist in the selection of tunnel excavation and support methods (JA, 2014a). The ground assigned to a particular class is expected to behave similarly in the tunnel excavation, and to require similar support methods. Ground Class 1 represents the overall best ground conditions, while Ground Class 3 represents the poorest ground conditions anticipated. More information about the ground classes can be found in *Tunnel Ground Characterization* (JA, 2014a).

All cross passages for the Freeway Tunnel Alternative are expected to be excavated using a sequence consisting of a top heading and a bench/invert due to their height. This sequence is applicable to all three support types corresponding to three ground classes. The primary initial support and presupport elements considered for the cross passage excavations include, but are not limited to, cement-grouted rock dowels, lattice girders, fiber-reinforced shotcrete lining, spiles, and fiberglass face dowels.

The approximate locations of these cross passages are indicated on the geologic profile and vertical alignment for the twin-bore variation of the Freeway Tunnel Alternative in Attachment A and the types of ground classes for each are detailed in *Tunnel Ground Characterization* (JA, 2014a). The preliminary design concepts for the support types for each ground class are detailed in *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* (JA, 2014e).

6.2.2 LRT Alternative. Ground conditions that would be encountered in various cross passages along the LRT alignment have been divided into three ground classes to assist in the selection of tunnel excavation and support methods (JA, 2014a). All cross passages for the LRT Alternative are expected to be excavated using a sequence consisting of either a full face or a top heading and a bench/invert, depending on the ground class encountered.

The primary initial support and presupport elements considered for the cross passage excavations include, but are not limited to, cement-grouted rock dowels, lattice girders, fiber-reinforced shotcrete lining, spiles, and fiberglass face dowels.

The approximate locations of these cross passages are indicated on the geologic profile and vertical alignment for the LRT Alternative in Attachment A and the types of ground classes for each are detailed in *Tunnel Ground Characterization* (JA, 2014a). The preliminary design concepts for the support types for each ground class are detailed in *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014f).

6.3 Groundwater Control Measures

As indicated in Section 3.3, the groundwater levels are expected to be above the base of the excavation for most of the Freeway Tunnel Alternative cross passages and for a good number of LRT Alternative cross passages (see the geologic profiles for both the Freeway Tunnel and LRT Alternatives in Attachment A). Therefore, some level of groundwater inflows are anticipated during excavation of cross passages, especially those for the Freeway Tunnel Alternative, unless systematic ground improvement measures, as discussed above, are implemented to treat the ground prior to excavation. The highest flows are expected to occur where the ground is highly fractured, where shear zones are encountered, or where untreated saturated alluvium is encountered.

The groundwater inflows to be encountered during the excavation are expected to occur as either transient (also referred to as flush flows), or sustained (steady-state) flows. Both types of flows are anticipated to occur in the cross passage excavation and would need to be managed using feasible groundwater control measures. Flush flows in rock occur primarily as flow through open fractures in recently excavated zones. Steady-state flows occur after flush flows have decreased and reached a sustained level. Estimates of the maximum potential groundwater flush flows and sustained flows are not available and will be carried out in future design phases.

Where ground improvement measures are not implemented, the following groundwater control measures are considered feasible for use during the cross passage excavation:

- Probing ahead of excavation zones to identify zones of high flush flows;
- Dewatering/pre-drainage from the ground surface;
- Ground modification measures (i.e., pre-excavation grouting or ground freezing) to reduce the rock mass hydraulic conductivity of zones of disturbed or faulted rock;
- Reduction of groundwater head by passive drainage via weep holes and/or strip drains, and advance probing.

Additional inflow control measures would be required where needed to protect the regional groundwater resources. More detailed information in terms of the characteristics of groundwater resources is required in order to develop specific criteria, plans, and procedures for employing the effective groundwater control measures and meeting the project-specific requirements.

6.4 Construction Monitoring

An integral part of the design and construction of cross passages is the construction monitoring of ground performance/support behavior during construction. Standard practices for tunneling using the SEM include observation of encountered ground conditions and monitoring of the ground deformations and groundwater conditions by geotechnical instrumentation.

The primary purpose of the monitoring program is to provide information on the construction process, including its effect on the ground surface and nearby surface and subsurface structures and the adequacy of the ground support. A well-designed program would provide the contractor and designers with information that can be used to verify design assumptions (structure deformations, geology, ground movements, etc.), which, in turn, helps to ensure that a stable excavation and final structure are achieved during and after construction.

The construction monitoring program should be in place at the start of each excavation and continue until completion of the permanent structure. In the cases where the cross passages are to be drained during excavation, and the natural groundwater level has to be depressed during construction, postconstruction monitoring is recommended. Postconstruction monitoring should be performed until the groundwater level has been restored to its preconstruction level. Excavations should be monitored for greater than anticipated deflections. In-tunnel instruments are also necessary as they provide immediate feedback of the ground and initial support behavior.

The following instrumentation monitoring activities are typical for excavations using the SEM:

- Monitoring ground displacements at the surface and near surface
- Monitoring of convergence/deformations of the excavation support systems
- Monitoring regional and local groundwater levels

These monitoring activities are used for verifying excavation support design assumptions in terms of ground conditions, effectiveness of selected ground support measures, needs for contingency support measures, and effectiveness of groundwater control measures.

6.5 Tunnel Interfaces

The cross passages would be excavated after the bored running tunnels have passed the location of cross passages to be constructed. Details of the excavation sequencing can be found in *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* (JA, 2014e) and *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014f). To facilitate the construction of cross passages, the following activities are required:

- Installation of a temporary bracing system within the running tunnels adjacent to the cross passage to maintain support of the ground during the initiation of cross passage excavation (i.e., breakout from the running tunnels). These temporary support systems would ensure the stability of the running tunnels during the cross passage construction. They would remain in place until the final lining for cross passages is installed. The design and installation of these temporary support systems should minimize potential impact to the construction traffic within the completed running tunnels.
- Installation of equipment for ground freezing, if used. This equipment would remain in place until the final lining for cross passages is installed.
- Removal of installed segments within the cross passage breakout openings. These segments would have to be predesigned to accommodate removal.

Locations of the cross passages would need some minor flexibility to allow for adjustment of locations to coincide with circumferential segment joints, which would ease breakout. The final lining system of the cross passages would also have to be specially detailed to provide a relatively watertight connection to the segmental lining gasket system.

Construction activities related to the cross passage construction, such as roadheader or excavator excavation, muck removal, shotcreting, etc., may also interfere with those for the bored running tunnel construction. Coordination of activities related to construction of different underground components is critical in achieving the project objectives.

6.6 Preliminary Assessment of Tunnel Excavation Methods for Cross Passages

Use of a 50-ton weight class roadheader is anticipated to be a feasible option for the excavations of the freeway cross passages. This class of roadheader is expected to be able to effectively excavate the anticipated rock with known range of rock strengths in the cross passages. For the excavations of the LRT cross passages, use of an excavator or backhoe with a cutting attachment is expected to be a more feasible and economical approach.

Systematic ground improvement measures are expected to be required, especially for excavation of the cross passages in soil or mixed face conditions and in areas where high groundwater inflows are anticipated. These measures can be employed to stabilize the openings, control ground movements and groundwater inflows, and minimize ground loss and the potential for surface settlement. Feasible ground improvement measures may include dewatering, permeation grouting, chemical grouting, jet grouting, or ground freezing, individually or in combination with each other.

7 Other Considerations

7.1 Fault-Induced Offset

As shown in Attachment A, the running tunnels are expected to cross active and potentially active faults, where large fault-induced offsets up to several feet could occur. These fault-induced offsets could potentially cause significant damage to the tunnel segmental linings and internal structures, so mitigation measures that would limit the adverse impact to the tunnel structures should be employed in the tunnel design and construction. The potential offsets have been estimated by CH2M HILL (2014).

To accommodate the expected fault-induced offsets, a tunnel section consisting of an enlarged internal cross section at the fault crossings for each tunnel bore is proposed. The proposed seismic reach is sized so that it has a length long enough to extend beyond the anticipated boundaries of the fault and an enlarged tunnel cross section large enough to accommodate the anticipated fault offset displacements. The objective would be to design the structure to avoid collapse in an earthquake and at the same time have a system that could be repaired without major reconstruction to restore functionality after an event. Details of the proposed methods for the tunnel seismic sections for each alternative are provided in *Preliminary Design Concepts for Fault Crossings* (JA, 2014g).

7.2 Construction Power

TBMs and the other supporting equipment necessary to excavate the tunnels require a significant amount of power. Primary power is usually supplied by utilities via high-voltage transmission to a substation at the tunnel’s construction portal area, and backup emergency power is supplied by generators. Backup power for use in emergencies is only needed to support critical activities such as tunnel ventilation and lighting. The site distribution system is generally designed by the contractor based on equipment requirements. Because of the potentially long lead time for the required power to be supplied to the portal areas, a preliminary estimate of the power necessary for construction of both the Freeway Tunnel and LRT Alternatives has been performed earlier in the study phase before all of the project assumptions, such as the tunnel diameter being used for this study, were determined. The results are summarized in a brief memorandum which is included as Attachment B. The detail provided in the power needs memorandum was shown so that local agencies would understand the needs of a typical TBM boring operation, however the actual power needs are subject to change based on the means and methods of the contractor who performs the work.

7.3 Excavated Material

A significant amount of excavated material would be generated from the excavation of the running tunnels, cross passages, TBM launch portals, and underground LRT stations. The material would have to be handled, stockpiled at the portal areas, and disposed of. Details of the estimated volumes of excavated material which could be generated for each of the alternatives are presented in *Handling and Disposalof Excavated Material* (JA, 2014h).

7.4 Manmade Obstructions

When tunneling with a TBM it is generally necessary to determine the risk of encountering manmade objects along the alignment, including but not limited to oil wells, foundation piles, borehole casings, and storage tanks. Attempts should be made to identify the risk of potential obstructions during the site reconnaissance phase. Existing records and historical documents such as maps, surveys, photographs, as well as visual surface indicators along alignment (graded areas and foundations), should be consulted and analyzed to determine the risk of potential obstructions.

The East Central Interceptor Sewer is an example of a Los Angeles project that successfully tunneled through Inglewood Oil Fields (Keller and Crow, 2004). If any zones of the SR-710 alignment were determined to be high-risk, a similar procedure could be used as that used to predict the presence of abandoned oil wells. First, a surface magnetometer study was performed in the area of interest to determine any anomalies. Then, during tunneling, forward probe drilling was performed from within the TBM; after probing, magnetometers were inserted ahead of the TBM. If anomalies were detected, the TBM speed was reduced. If deemed necessary, surface explorations may be performed. If an obstruction is determined to exist, it may be necessary to excavate and remove or treat it according to present-day abandonment standards.

7.5 Construction Portal Laydown Areas

The layout of temporary facilities at construction portal sites is ultimately the contractor’s responsibility. However, for planning purposes, both the north and south portal sites should be checked to ensure that there is enough space for the contractor to perform the required construction operations associated with the tunnel excavation.

Examples of typical construction portal site layouts for the twin-bore variation of the Freeway Tunnel Alternative (north and south portals) are shown in Attachment C. Due to the fact that several details of the project are not yet determined and that it is ultimately at the contractor’s discretion, these plans are of a conceptual nature only; they serve to show that there is sufficient space available at the portals for construction activities and excavation-related equipment. Layouts were not provided for the single-bore variation of the Freeway Tunnel Alternative or the LRT Alternative, as the twin-bore variation of the Freeway Tunnel Alternative would require the most area.

7.6 TBM Abandonment

For the LRT Alternative, the two TBMs are expected to each be launched at the south portal and mine north to the Fillmore Station, where the TBMs would be retrieved. For the Freeway Tunnel Alternative, it is anticipated that two TBMs would be used to excavate each tunnel bore, and each TBM would excavate roughly half of the alignment; however this should be re-evaluated in future phases of the study. If two TBMs were used for each bore, the two TBMs excavating each tunnel bore would meet underground at the end of their drives. Because the TBMs meet underground, they cannot be retrieved like in the LRT Alternative.

Typically, when a TBM is abandoned the TBM shield is left in place providing temporary ground support while the remaining TBM components including the trailing gear and cutterhead would be removed from the tunnel. The cutterhead would be removed in pieces, with the contractor supporting the ground around it as needed. Then a reinforced concrete cast in place final lining would be installed inside each TBM shield (longitudinally between the segmental lining already installed by each TBM). Abandoning the TBM shield is a practice that is commonly performed if a TBM cannot be retrieved at a shaft or portal location at the end of its drive.

8 References

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9 Revision Log

Revision 0	November 8, 2013	Internal Review
Revision 1	December 18, 2013	Metro/Caltrans Review
Revision 2	May 27, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

Geologic Profiles

NOTES:

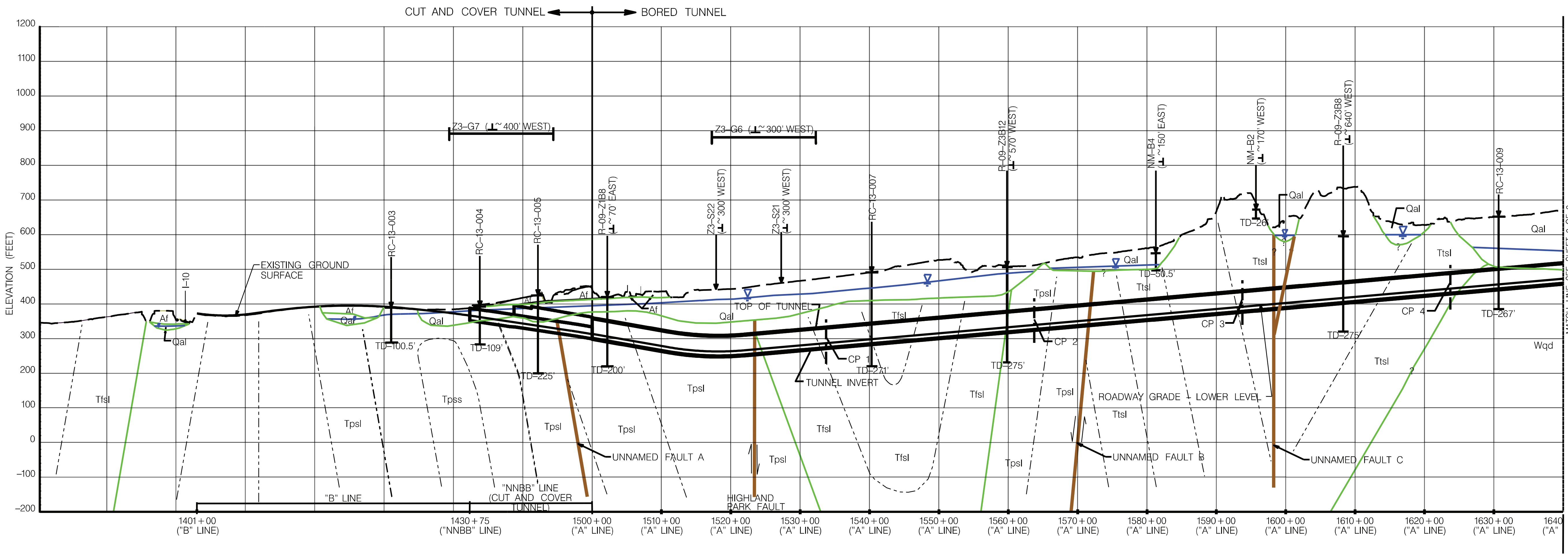
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Tss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
- INTRAFORMATIONAL CONTACT
- GENERALIZED BEDDING
- ESTIMATED TOP OF GROUNDWATER TABLE
- Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION
- A-13-020 TD-149.8' GEOTECHNICAL BOREHOLE WITH TOTAL DEPTH AND PROJECTION:
  - A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
  - R-09-Z1B8 – CH2M HILL, 2010
  - NM-B3 – NINYO AND MOORE, 1999
  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

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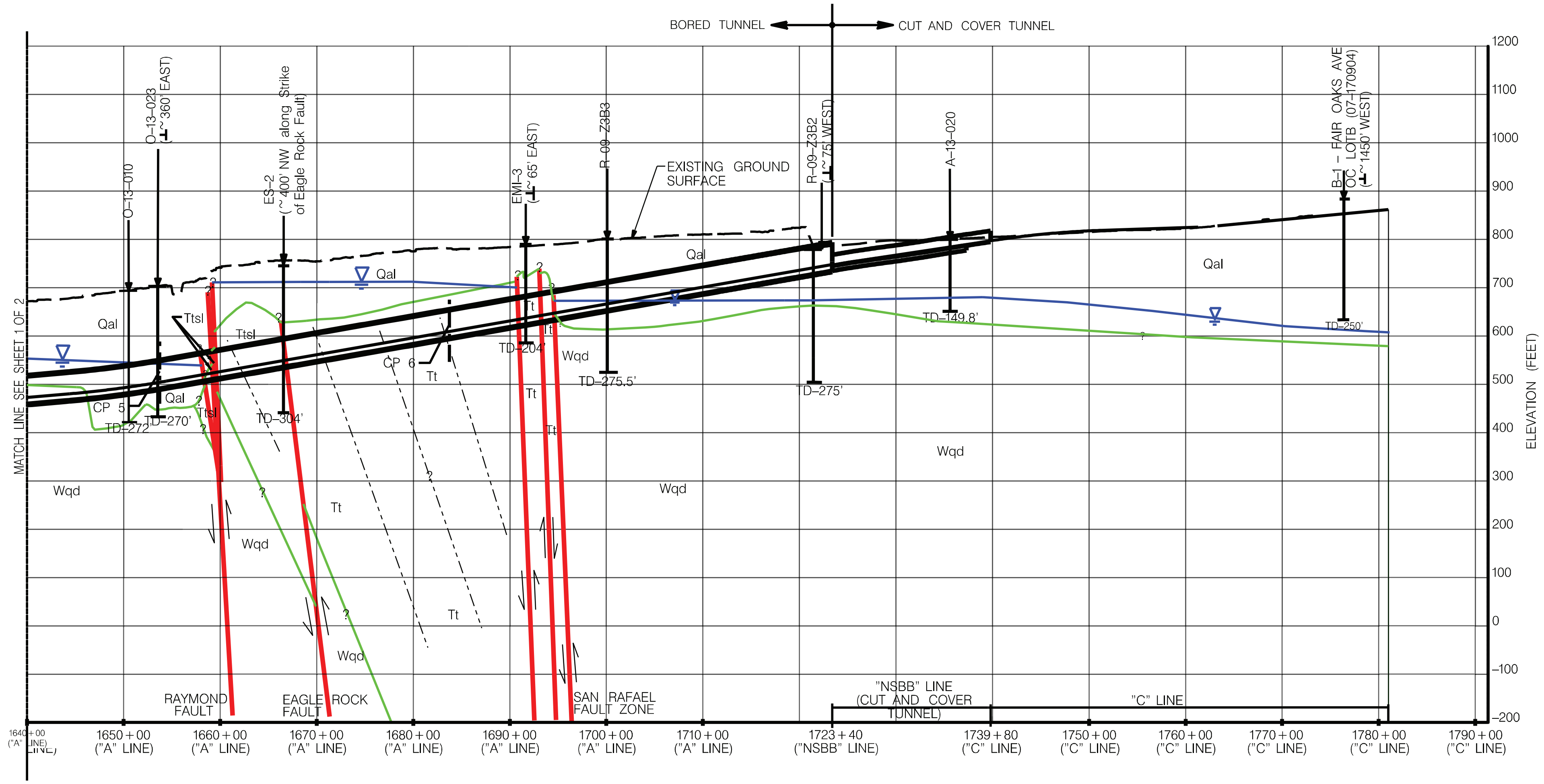
Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af ARTIFICIAL FILL

Qal ALLUVIAL SOIL

Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER

Tfsl FERNANDO FORMATION, SILTSTONE MEMBER

Tpsl PUENTE FORMATION, SILTSTONE MEMBER

Tpss PUENTE FORMATION, SANDSTONE MEMBER

Tt TOPANGA FORMATION, UNDIFFERENTIATED

Ttss TOPANGA FORMATION, SANDSTONE MEMBER

Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER

Ttsl TOPANGA FORMATION, SILTSTONE MEMBER

Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

A-13-020 TD-149.8' GEOTECHNICAL BORHOLE WITH TOTAL DEPTH AND PROJECTION:
A, R, RC, O-13-001 - CH2M HILL, THIS STUDY
R-09-Z1B8 - CH2M HILL, 2010
NM-B3 - NINYO AND MOORE, 1999
EMI-3 - EARTH MECHANICS INC, 2006
ES-2 - CALTRANS, 1974

CP - CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 2 OF 2

NOTES:

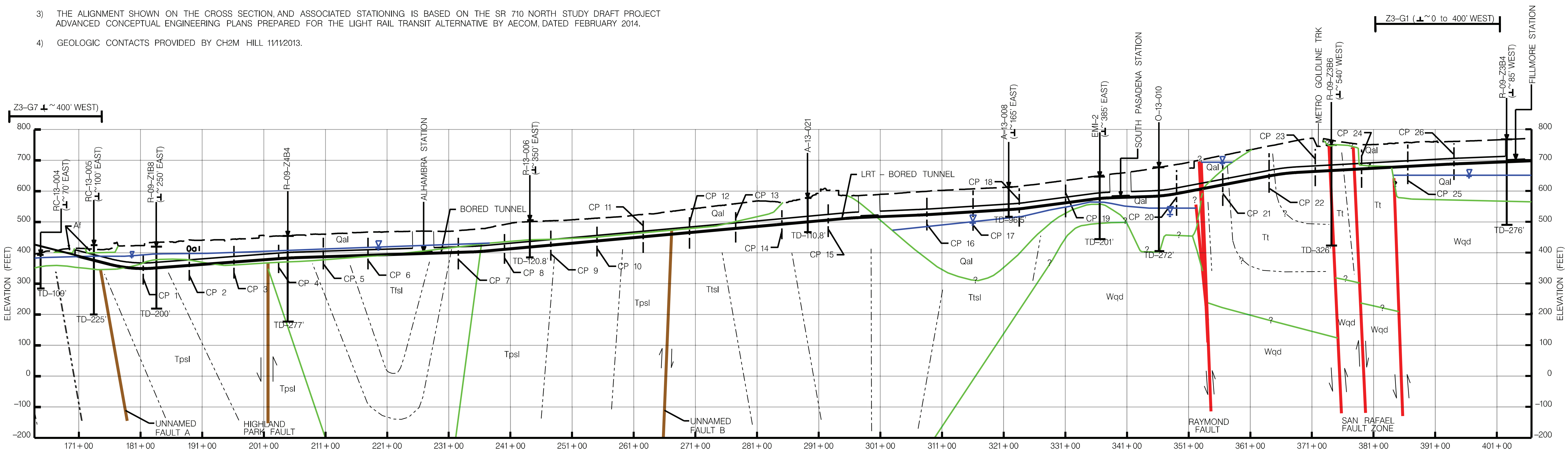
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT ADVANCED CONCEPTUAL ENGINEERING PLANS PREPARED FOR THE LIGHT RAIL TRANSIT ALTERNATIVE BY AECOM, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative



LEGEND

UNITS

Af ARTIFICIAL FILL

Qal ALLUVIAL SOIL

Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER

Tfsl FERNANDO FORMATION, SILTSTONE MEMBER

Tpsl PUENTE FORMATION, SILTSTONE MEMBER

Tpss PUENTE FORMATION, SANDSTONE MEMBER

Tt TOPANGA FORMATION, UNDIFFERENTIATED

Ttss TOPANGA FORMATION, SANDSTONE MEMBER

Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER

Ttsl TOPANGA FORMATION, SILTSTONE MEMBER

Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

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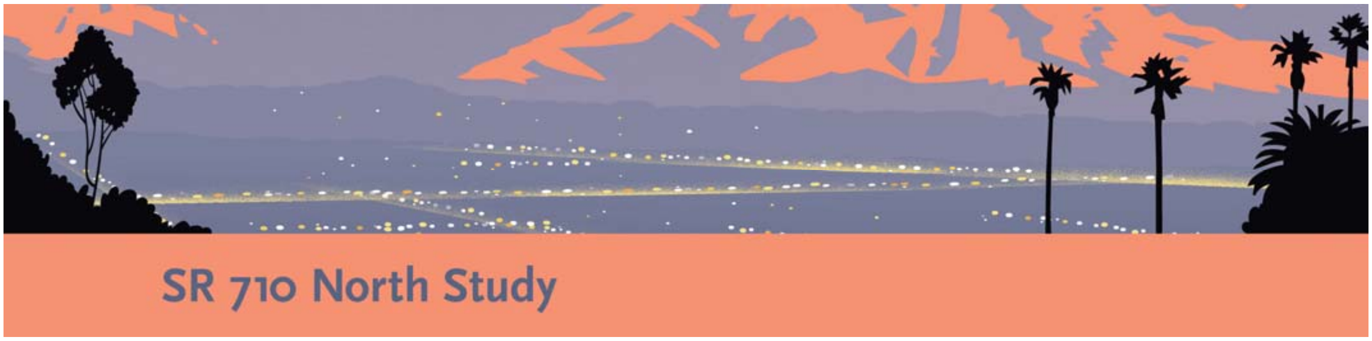
NM-B3 – NINYO AND MOORE, 1999
EMI-3 – EARTH MECHANICS INC, 2006
ES-2 – CALTRANS, 1974

CP – CROSS PASSAGE

LRT GEOLOGIC PROFILE

Attachment B

Temporary Power Requirements



TECHNICAL MEMORANDUM

Conceptual Stage: Power Needs for Tunnel Excavation

PREPARED FOR: CH2M HILL

PREPARED BY: Jacobs Associates

DATE: May 15, 2013

PROJECT NUMBER: 428908

Introduction

This memorandum summarizes approximate power requirements for the bored tunnel excavation of the freeway and light rail transit (LRT) alternatives; permanent power for tunnel operations is not considered herein. In addition to providing estimated power needs, this memo summarizes what is typically done with respect to power on other tunnel projects excavated with an earth pressure balance tunnel boring machine (TBM), the type of machine that is anticipated to be used to excavate these tunnels.

TBMs and the other supporting equipment necessary to excavate the tunnel require a significant amount of power. Primary power is usually supplied by utilities via high voltage transmission to a substation at the tunnel's construction portal area, and the backup emergency power is supplied by generators. Backup power for use in emergencies is only needed to support criticial activities such as tunnel ventilation lighting and is not within the scope of this memorandum. The site distribution system is generally designed by the contractor based on equipment requirements. This memorandum is being written as conceptual-level decisions are still being finalized and the details herein are subjust to change.

Portal Areas

Power provided by the utility is fed or dropped to a substation located at the construction portal(s). The portal substation consists of switchgear, transformers, and adjacent emergency generators. The substation will be used to provide and regulate power for the construction-related equipment at the portal areas, including the TBMs. The area (footprint) needed for these substations will vary depending on the power needs and the contractor's means and methods, but generally it will range from 4,000 to 8,000 square feet.

In the case of the SR 710 project, significant power will be required and close coordination between the utility and the contractor is anticipated. In many cases it will be in the interest of both the contractor and the utility to communicate early on about temporary and permanent power needs and to orchestrate a phased implementation of the supply. For example, the appropriate equipment can be commissioned and decommissioned at the portal as required before, during, and after the TBM excavation period, to best match the needs of the project schedule and the utility's capacity plan.

Power Needs

The following sections present an estimate of the power needed during tunnel construction for the twin-bore freeway alternative and the twin-bore LRT alternative. The loads for the freeway alternative are a preliminary estimate based on the information the design team has obtained from the Alaskan Way Viaduct (SR99) tunnel project, which is the only similar diameter project currently underway in the world. The SR-710 project is anticipated to have similar power requirements for each of its large diameter TBMs. It should be noted that the power requirements and site setup is highly dependent on the contractor's means and methods.

Twin-Bore Freeway Tunnel Alternative

The TBM-driven tunnels in this alternative are two 62-foot outside-diameter tunnels that will each be approximately 22,400 feet in length. It is anticipated that four TBMs will be required to excavate these tunnels—two mining north from the south portal and two mining south from the north portal. This configuration would make each TBM drive approximately 11,200 feet in length with two separate working portals.

Table 1 provides an estimate of the power demand at each of the two freeway TBM launch portal sites; at this time it is assumed that the power needs at both sites will be identical and that the construction at both portals woud be simultaneous.

TABLE 1.
Construction Power Demand at Each Freeway TBM Portal

Equipment Description	Power Factor	Real Power (kW)	Apparent Power (kVA)
Portal, Supporting Two Tunnel Drives			
Office Trailers	0.95	150	158
Dry House	0.95	25	26
Work Shop	0.85	125	147
Pumps	0.90	75	83
Lighting	0.95	100	105
Cranes	0.90	250	278
Grout/Foam Plant	0.85	300	353
Compressor Plant	0.85	1,200	1,412
Alimak Elevators	0.90	50	56
Portal Miscellaneous	0.90	500	556
Northbound Tunnel, L = 11,200 ft, OD = 62 ft			
Tunnel Boring Machine	0.85	24,500	28,824
Tunnel Lighting	0.90	125	139
Tunnel Ventilation	0.90	1,000	1,111
Tunnel Muck Conveyor	0.90	750	833
Southbound Tunnel, L = 11,200 ft, OD = 62 ft			
Tunnel Boring Machine	0.85	24,500	28,824
Tunnel Lighting	0.90	125	139
Tunnel Ventilation	0.90	1,000	1,111
Tunnel Muck Conveyor	0.90	750	833
Subtotal		55,500	65,000

Twin-Bore LRT Tunnel Alternative

The TBM-driven tunnels in this alternative are two 20.5-foot outside-diameter tunnels that will each be approximately 22,200 feet in length. It is anticipated that two TBMs will be used to excavate these tunnels, both TBMs mining north from the south portal. Each TBM would drive the full length of the alignment from only one working portal.

Table 2 provides an estimate of the power demand at the LRT TBM launch portal. The estimates in this table are limited to the power required at the portal sites. Construction power will be necessary at each of the underground stations; however that is not within the scope of this memorandum.

TABLE 2.
Construction Power Demand at the LRT TBM Portal

Equipment Description	Power Factor	Real Power (kW)	Apparent Power (kVA)
Portal, Supporting Two Tunnel Drives			
Office Trailers	0.95	100	105
Dry House	0.95	25	26
Work Shop	0.85	100	118
Pumps	0.90	75	83
Lighting	0.95	75	79
Cranes	0.90	250	278
Grout/Foam Plant	0.85	200	235
Compressor Plant	0.85	350	412
Alimak Elevators	0.90	50	56
Portal Miscellaneous	0.90	250	278
Northbound Tunnel, L = 22,200 ft, OD = 20.5 ft			
Tunnel Boring Machine	0.85	3,000	3,529
Tunnel Lighting	0.90	200	222
Tunnel Ventilation	0.90	1,000	1,111
Tunnel Muck Conveyor	0.90	1,000	1,111
Southbound Tunnel, L = 22,200 ft, OD = 20.5 ft			
Tunnel Boring Machine	0.85	3,000	3,529
Tunnel Lighting	0.90	200	222
Tunnel Ventilation	0.90	1,000	1,111
Tunnel Muck Conveyor	0.90	1,000	1,111
Subtotal		11,900	13,600

Duty Cycles and TBM Power Usage

Tables 1 and 2 provide the demand if 100% of the equipment were being used simultaneously. In reality, there are duty cycles for each piece of equipment, meaning that it may be in use only a certain percentage of the time. For example, the lighting and ventilation in the tunnel during construction will be on nearly 100% of the time and would therefore have a duty cycle of 100%, but much of the equipment in the workshops would have a duty cycle of, say, 50% or less since they are used only intermittently. At this point in the project, the design team does not have enough information to estimate duty cycles; therefore the full power demand is presented in the tables for planning purposes.

A TBM has a cutterhead that rotates and excavates rock/soil while jacks advance and steer the TBM through the ground. The cutterhead is typically driven by multiple variable frequency drive (VFD) motors For example, the TBM for the Alaskan Way project has 24 VFD motors to power its main cutterhead drive.

A TBM typically excavates a fixed distance, say 5 feet, and then stops excavating while the excavated ground is supported with a lining. The power use of a TBM reflects this cycle, as the TBM will have the greatest power draw during the excavation phase of its cycle, which can last between 30 to 40 minutes, followed by a ring building phase which is typically of the same duration. Once that lining (one ring) is erected, the TBM can excavate another 5 feet of ground. As a result of this mining cycle, the TBM cutterhead drive motors cycle on and off multiple times throughout a shift. The VFD controlled motors are energized such that they “soft start” to reduce the in-rush current to motors that otherwise could result in unacceptable surges on the utility side.

It is important to note that TBM trailing gear, which moves along with the machine as it excavates, houses its own substation. Power is typically supplied to the TBMs at medium voltage (in the range of 11-26 kV) from the portal substation described above and then transmitted via a heavy armored and shielded cable to the TBM substation where it is stepped down to 690V or 480V 3-phase for the majority of the components of the TBM. Smaller transformers supply 220V and 110V single phase power for miscellaneous equipment and small tools. Site voltage used is dependent on contractor and equipment requirements.

Construction Duration

The power demand listed in Tables 1 and 2 would only be required during TBM excavation. Other construction activities will occur both before and after TBM excavation is complete; however, the peak power will be required during the TBM excavation operation. Based on preliminary schedules, the mining duration estimates are as follows, and assumed that all TBMs are excavating simultaneously:

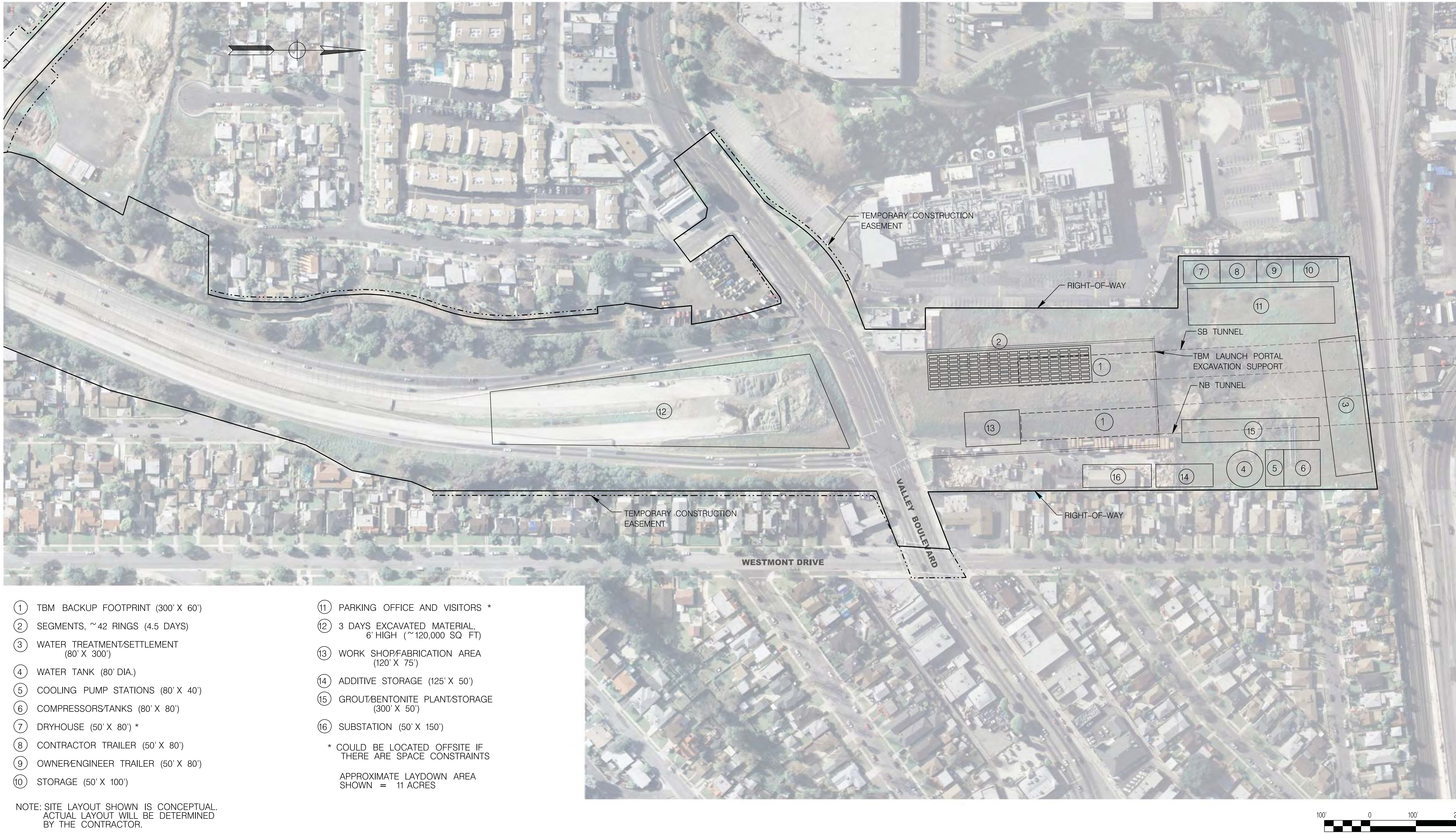
- Duration of mining an 11,200-foot length of freeway tunnel is approximately 2 years, and
- Duration of mining a 22,200-foot length of LRT tunnel is approximately 2 years.

Attachment C

Conceptual Site Layouts for Freeway Construction Portals

Metro

Caltrans



1. TBM BACKUP FOOTPRINT (300' X 60')
2. SEGMENTS, ~42 RINGS (4.5 DAYS)
3. WATER TREATMENT/SETTLEMENT (80' X 300')
4. WATER TANK (80' DIA.)
5. COOLING PUMP STATIONS (80' X 40')
6. COMPRESSORS/TANKS (80' X 80')
7. DRYHOUSE (50' X 80') *
8. CONTRACTOR TRAILER (50' X 80')
9. OWNER/ENGINEER TRAILER (50' X 80')
10. STORAGE (50' X 100')
11. PARKING OFFICE AND VISITORS *
12. 3 DAYS EXCAVATED MATERIAL, 6' HIGH (~120,000 SQ FT)
13. WORK SHOP/FABRICATION AREA (120' X 75')
14. ADDITIVE STORAGE (125' X 50')
15. GROUT/BENTONITE PLANT/STORAGE (300' X 50')
16. SUBSTATION (50' X 150')

* COULD BE LOCATED OFFSITE IF THERE ARE SPACE CONSTRAINTS

APPROXIMATE LAYDOWN AREA SHOWN = 11 ACRES

NOTE: SITE LAYOUT SHOWN IS CONCEPTUAL. ACTUAL LAYOUT WILL BE DETERMINED BY THE CONTRACTOR.

TWIN–BORE SOUTH PORTAL CONCEPTUAL SITE LAYOUT

SCALE: 1" = 200'

1 TBM BACKUP FOOTPRINT (300' X 60')
2 SEGMENTS, ~42 RINGS (4.5 DAYS)
3 WATER TREATMENT/SETTLEMENT (80' X 300')
4 WATER TANK (80' DIA.)
5 COOLING PUMP STATIONS (80' X 40')
6 COMPRESSORS/TANKS (80' X 80')
7 DRYHOUSE (50' X 80') *
8 CONTRACTOR TRAILER (50' X 80')
9 OWNER/ENGINEER TRAILER (50' X 80')
10 STORAGE (50' X 100')
11 PARKING, OFFICE AND VISITORS *
12 3 DAYS EXCAVATED MATERIAL, 6' HIGH (120' X 1000')
13 WORK SHOP/FABRICATION AREA (120' X 75')
14 ADDITIVE STORAGE (125' X 50')
15 GROUT/BENTONITE PLANT/STORAGE (300' X 50')
16 SUBSTATION (50' X 150')

* COULD BE LOCATED OFFSITE IF THERE ARE SPACE CONSTRAINTS

APPROXIMATE LAYDOWN AREA SHOWN = 10 ACRES

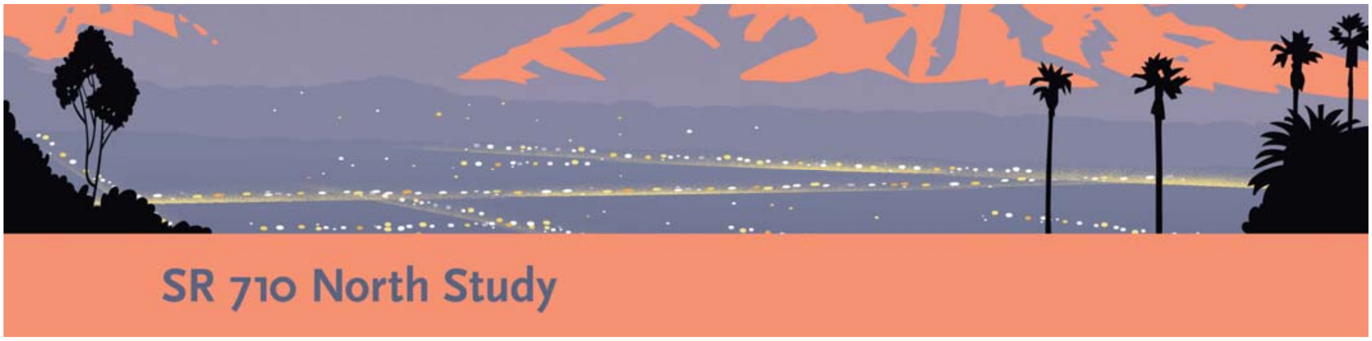
NOTE: SITE LAYOUT SHOWN IS CONCEPTUAL. ACTUAL LAYOUT WILL BE DETERMINED BY THE CONTRACTOR.

TWIN-BORE NORTH PORTAL CONCEPTUAL SITE LAYOUT

SCALE: 1" = 200'

100' 0 100' 200'

Appendix D
TM-4A Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings



TECHNICAL MEMORANDUM 4A

Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings

PREPARED FOR: Metro
COPY TO: Caltrans
PREPARED BY: Jacobs Associates/CH2M HILL
DATE: August 22, 2014
PROJECT NUMBER: 428908

1.0 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives involve tunnels for significant distances over their alignments.

The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) describes preliminary tunnel lining and cross passage design concepts for the twin-bore and single-bore variations of the Freeway Tunnel Alternative. Preliminary design drawings for the tunnel linings are provided in Attachment A. At this stage, the design and support details shown herein are conceptual, and this TM presents one feasible option for each of the design features in support of the environmental documentation and a cost estimate. A substantial amount of additional geotechnical investigations will be required before the design can be advanced beyond this conceptual stage.

Metro

Caltrans

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2.0 Tunnel Alignment and Geology

2.1 Tunnel Alignment

Figure 1 shows the tunnel alignment in profile. Both the twin-bore and single-bore variations have the same alignment. The twin-bore tunnels have a clear spacing of approximately 60 feet, or about one tunnel diameter for most of the alignment. This distance increases to approximately 70 feet at the northern end. The depth of cover from ground surface to tunnel crown ranges from approximately 40 to 280 feet, with an average of approximately 150 feet. See *Bored Tunnel Geometry* (JA, 2014a) for additional details of the tunnel alignment.

2.2 Geology

The geologic conditions along the alignment consist of Quaternary-age alluvium, Tertiary-age sedimentary rocks (Fernando, Puente, and Topanga formations), and crystalline basement complex rocks. The bored tunnel segment of the Freeway Tunnel Alternative crosses one active fault (the Raymond fault) and two potentially active faults (the Eagle Rock and San Rafael faults). Future studies may reveal that the Eagle Rock and San Rafael faults are inactive; however, for planning purposes, these two faults are treated as active faults. A generalized geologic profile along the tunnel alignment is shown in Figure 1. Detailed discussions of the geologic conditions along this alignment are provided in *Tunnel Ground Characterization* (JA, 2014b).

Portions of the tunnels for the tunnel alignment are expected to be excavated below the groundwater table. The water head ranges up to about 150 feet above the tunnel crown (see Figure 1). Perched water is expected in the alluvium, and fault zones act as water barriers and can result in large groundwater differentials on either side of the fault zones (up to approximately 170 feet in the case of the Raymond fault in the study area). The lowest groundwater levels can be found adjacent to the south side of the Raymond fault. Groundwater is below the tunnel invert for a short stretch adjacent to the North Portal. The highest groundwater heads are expected to occur when the tunnel passes under the eastern edge of the San Rafael Hills.

Tables 1 and 2 provide summaries of the recommended design parameters for the soil and rock mass units, respectively. Refer to JA (2014b) for additional details.

3.0 Overview of Underground Structures

3.1 Bored Running Tunnels

It is assumed that the freeway running tunnels would be excavated using a tunnel boring machine (TBM). The results of preliminary design analyses indicate that the tunnel excavation can be supported with a one-pass, 30-inch-thick bolted and gasketed precast concrete segmental lining. The lining would be composed of ten segments bolted together to form a ring and would serve as both the initial support and the final lining for the tunnel. It would be designed for ground loads, groundwater pressure, and other loads such as TBM thrust loads. Refer to drawings in Attachment A and Section 5 of this TM for details regarding the concrete segmental lining.

3.2 Internal Structure

The internal structure would consist of a double-deck roadway, and would be built inside the segmentally lined tunnel. Each deck has two 12-foot-wide travel lanes, a 10-foot-wide shoulder for vehicles, a 1-foot-wide shoulder, and enclosed walkways that would serve as the emergency egress routes in the event of an emergency such as a tunnel fire. The vertical clearance for vehicular traffic in the roadway is 15.5 feet. The roadways and walkways would be supported on walls that bear on corbels that are in turn supported on the segmental lining. The internal structure is discussed in detail in *Preliminary Design Concepts for the Freeway Internal Structure* (JA, 2014d). Vehicular traffic that would use the tunnel includes trucks and buses.

3.3 Cross Passages

Six pairs of cross passages are proposed along the twin-bore variation. Each cross passage pair consists of one upper level cross passage and one lower level cross passage. The upper and lower cross passages in each pair are separated by a minimum clear distance of approximately 100 feet. The cross passage pairs occur roughly every 3,000 feet along the tunnel alignment. Refer to drawings in Attachment A for details. No cross passages exist for the single-bore variation. The cross passages are intended as vehicular passages and are designed to permit a truck-sized vehicle (e.g., a fire truck) to move from one bore to the other. Additional details regarding the operational characteristics of these passages can be found in *Bored Tunnel Geometry* (JA,2014a).

3.4 Fault Section

At the locations where active and potentially active faults cross the tunnel alignment—the Raymond fault, Eagle Rock fault, and San Rafael fault—a special seismic section would be provided that can accommodate potential fault offset. In these areas additional clear space is provided by using thinner steel/concrete segments in lieu of the regular precast concrete segments. This additional clear space is sufficient to accommodate the estimated magnitude of potential fault offset. Details regarding this special seismic section can be found in *Preliminary Design Concepts for Fault Crossings* (JA, 2014e).

3.5 Portals and Cut-and-Cover Structures

Permanent portal structures and cut-and-cover portions of the tunnels near the portals are being designed by other members on the CH2M HILL-led team. Preliminary design concepts for the excavation and support for the temporary construction portals are discussed in *Preliminary Design Concepts for the Freeway Portal Excavation Support Systems* (JA, 2014c).

4.0 Design Criteria and General Requirements

4.1 Codes and Guidelines

The codes and guidelines applicable for design include, but are not limited to, the following:

- California Department of Transportation (Caltrans) Bridge Design Specification (Caltrans, 2011).
- California Department of Transportation Seismic Design Criteria, Version 1.7. April 2013 (Caltrans, 2013).
- AASHTO, LRFD Bridge Design Specifications, 4th ed., American Associations of State Highway and Transportation Officials, Washington, DC, 2007 (AASHTO, 2007).
- American Institute of Steel Construction, Code of Standard Practice for Steel Buildings and Bridges, 13th Edition (LRFD), 2005 (AISC, 2005).
- FHWA Technical Manual for the Design and Construction of Tunnels – Civil Elements, FHWA-NHI-10-034, 2009.
- John, M., and M. Bruno. 2003. Shotcrete Lining Design: Factors of Influence, In RETC 2003 Proceedings, 726–734. (John and Bruno, 2003).

- The Concrete Society, Technical Report No. 63: Guidance for the Design of Steel-Fibre-Reinforced Concrete, CCIP-017. 2007.
- ITA Guidelines for Structural Fire Resistance for Road Tunnels (ITA, May 2004).

4.2 Service Life, Durability and Fire Resistance

At this stage of the design, the required service life and durability of the various underground structures have not been established by Caltrans. Structures designed in accordance with Caltrans (2011) are expected to have a service life of 75 years. Key elements impacted by design service life include required concrete cover for conventional rebar, gasket design of segmental lining, design for potential fires in the tunnels, and corrosion design.

For fire resistance, the tunnel lining would be required to withstand the design fire event without loss of structural integrity. The design fire event has not been determined at this time. Section 7.0 provides key considerations for design of a fire-resistant tunnel lining.

According to the Preliminary Geotechnical Report (CH2M HILL, 2014), there is a low-to-moderate potential of encountering naturally occurring oil and/or gas within the Puente Formation or fault zones along the Freeway Alternative. Additionally, previous underground experience in local formational materials indicates that gassy ground may be encountered. Naturally occurring oil and/or gas could also be found within any of the geologic formations within the study area. Therefore, the segmental lining is shown with double gaskets for the purposes of controlling potential gas infiltration and to enhance seismic performance, similar to the systems that have been previously used by Metro for projects such as the Metro Gold Line Eastside Extension in Los Angeles, California. In addition to the double gaskets, cross gaskets could also be used to confine leakage and/or grouting between adjacent segments; however, this need will be evaluated in future phases of the projects.

Similar to the segmental lining, the cross passage final linings and the connection between the cross passage ring beam and the segmental lining would be fully enclosed with a membrane for providing an essentially impermeable water and gas barrier. The gaskets and membranes used should be non-degradable (by hydrocarbons for example) in the expected chemical environment and should not leak during static and dynamic loading conditions.

A project having significant gas mitigation requirements is Metro’s Westside Subway Extension project in Los Angeles. Gasket materials for segmental tunnel liners and permanent cut-and-cover structures being considered for the above project are ethylene propylene diene monomer (EPDM), chlorinated polyethylene (CPE), polychloroprene (Neoprene), and nitrile. In areas of elevated gas concentrations, a double liner is recommended with a hydrocarbon resistant (HCR) sandwich membrane. High Density Polyethylene (HDPE), a material that is resistant to degradation presence of hydrocarbons and hydrogen sulfide, is the recommended HCR material and has been used by Metro to date to control/mitigate gas infiltration in tunnels in the Los Angeles area.
If future geotechnical investigations reveal a potential for significant gas inflows, then appropriate materials and lining systems should be chosen for the segmental lining gaskets and the cross passage final lining membranes and a testing program should be implemented to prove that these would perform adequately throughout the design life of the tunnel structures.

The above mentioned gaskets and membranes essentially render the tunnel structure watertight. However, a perfectly dry tunnel is typically not guaranteed and some water infiltration should be anticipated. The generally accepted standards for realistically limiting water ingress in tunnels include:

- 1 gallon per minute per 1,000 feet of tunnel
- Local infiltration limited to 0.25 gallon per day for a 10 square foot area, and one drip per minute
- No water ingress that causes entry of soil particles

Water entering the tunnel and cross passages would be diverted into the tunnel drainage system, which would discharge into a low point sump. From there it would be pumped out of the tunnel. In case underground gas enters the tunnel through the infiltrating water, the ventilation system would be designed to extract these gases out of the tunnel.

4.3 Tunnel and Cross Passage Dimensions

Tunnel and cross passage sizing is described in *Bored Tunnel Geometry* (JA, 2014a). In general, the tunnels would have a double-deck roadway; each deck carries two lanes of traffic and also has shoulders and emergency egress walkways. Additionally, in the twin-bore variation, each deck level has six cross passages spaced throughout the tunnel, for a total of twelve cross passages.

The cross passages are sized to allow a fire truck-sized vehicle to move from one bore to the other. The vehicle clearance envelope inside the passages is 14.5 feet high by 20 feet wide. Angled cross passages were implemented due to accommodate the turning radius of a larger truck; the inside angle between the passage and the tunnel bore is currently shown to be approximately 50 degrees. The angle of the cross passages and the connections will be revisited when better ground information is available, and the operational need should analyzed to optimize the safe and economic construction of the cross passages in future phases of this study. Refer to the drawings in Attachment A for additional details.

Several utilities—including power lines and service items such as transformers and ventilation equipment—would be present in the tunnel. The exact location of these utilities within the tunnel cross sections has not been determined at this time.

Additionally, wet wells for drainage purposes would be required. Possible locations for a wet well could be in the utilidor below the bottom roadway in the running tunnels or in a sump structure below a cross passage. For the single-bore variation, as there are no cross passages, an alternative drainage concept may be necessary which could require additional excavation. Details of the drainage concepts will be further evaluated in future phases of this study.

4.4 Design Loads, Load Factors, and Load Combinations

4.4.1 Design Loads

The preliminary design of the tunnel lining, cross passage final lining, and the cross passage initial support is for:

- *Dead Loads (DC, DW):* Weight of structures and other permanent elements, including internal structures/facilities such as invert slab, walkway, and tunnel utilities.
- *Ground Loads (EH, EV):* Vertical and horizontal soil or rock loads.
- *Hydrostatic (Water) Loads (WA):* Groundwater pressure.
- *Seismic Loads (EQ):* Effects of design earthquake ground motions. The seismic events that apply to the tunnel and cross passage linings are the Functional Earthquake Event and the Safety Earthquake Event. The seismic event that applies to the cross passage initial support is the Construction Earthquake Event. Refer to CH2M HILL (2014) for details regarding these events.

Loads during construction on the segmental lining include TBM thrust and torque, segment grouting pressures, and stacking and handling loads. Of these, a preliminary check is performed on the segments for estimated values of TBM thrust and torque as these loadings can be significant; these details are discussed in Section 5.0. Grouting pressures and stacking and handling will be addressed during future design phases as these do not usually control lining design. Shrinkage and temperature loads during the casting process for the precast segmental lining and cross passage final linings will be considered during subsequent design stages. The use of shrinkage-reducing admixtures to minimize shrinkage cracking of the roadway decks could also be considered.

4.4.2 Load Factors and Load Combinations

4.4.2.1 Segmental Lining and Cross Passage Final Lining

These loads can be estimated based on mean or lower-bound geotechnical parameters (see Tables 1 and 2). With mean parameters, full code-specified load factors have been used. Lower-bound geotechnical parameters were also used to address potentially adverse conditions in the absence of a robust boring program. When lower-bound geotechnical parameters associated with adverse ground conditions are considered, the loads on the tunnel lining increases. The increased loads combined with the application of full code specified load factors may be overly conservative. Therefore, when load cases based on lower-bound geotechnical parameters are used to check the lining structural capacity in adverse ground conditions, we used a load factor of 1.0 for the ground-related loads. When ground parameters are better understood and project-specific design parameters are selected, they will be used in conjunction with Caltrans-recommended load factors in future phases of this design.

TBM thrust and torque have known maximums (due to the limits of the TBM), which cannot be exceeded and hence the load factors on these are also taken as 1.0. In addition, these loads are temporary, acting on the segment ring immediately after it is installed in the tail of the TBM, and the loads dissipate after the ring exits the TBM and additional rings are installed. This approach will be re-evaluated during future design phases ensuring that sufficient conservatism is included in the determination of the lower-bound parameters and maximum TBM reaction forces.

Static Design

The static design of the tunnel lining and cross passage final lining is for the STRENGTH I load combination per Caltrans (2011). The load combination and load factors are as below; the load factor of 1.0 in combination with adverse ground conditions was explained above.

1.25 (DC+ DW) + 1.35 (EH+ EV) + 1.35 WA (expected ground conditions based on mean soil and rock mass properties)

1.25 (DC+ DW) + 1.0 (EH+ EV) + 1.0 WA (adverse ground conditions based on lower-bound soil and rock mass properties)

Seismic Design

The seismic design of the tunnel lining and cross passage final lining is for the EXTREME I load combination per Caltrans (2011). The load combination and load factors are as below. The preliminary design analyses only consider the SEE event as that is the larger event and is expected to control design of the lining. A design check for the smaller FEE event will be performed during future design phases.

1.0 (DC+ DW) + 1.0 (EH+ EV) + 1.0 WA + 1.0 EQ_{SEE}

Service load combinations are also specified by Caltrans (2011). These are primarily used to estimate deformations and deflections. At this preliminary design stage, the focus is on the evaluation of strength and seismic requirements in order to estimate a lining thickness and reinforcing provisions that will be structurally adequate. During future design phases, segmental lining deformations and associated impacts will be evaluated as part of service load combinations.

4.4.2.2 Cross Passage Initial Support

Load factors for initial support (e.g., shotcrete and rock dowels) are not specified in the codes. The load factors and load combinations for the cross passage initial support are based on our experience with similar projects; the load factor of 1.0 in combination with adverse ground conditions was explained above.

Static Design

1.35 (DC+DW) + 1.35 (EH+ EV) + 1.35 WA (expected ground conditions based on mean soil and rock mass properties)

1.0 (DC+ DW) + 1.0 (EH+ EV) + 1.0 WA (adverse ground conditions based on lower-bound soil and rock mass properties)

Seismic Design

The initial support for cross passages is a temporary structure with a design service life of less than 2 years. A 100-year event is likely an appropriate seismic event for the initial support. It is not expected that this event will control the design of the initial support; however, it will be included in the design in future design phases.

4.4.3 Material Resistance Factors

Material resistance factors are used to reduce the material strengths, e.g., concrete crushing strength or steel yield strength from their ultimate values. These are referred to as the ϕ-factors. The load factors divided by the material resistance factors can be thought of as a safety factor in the design. The material resistance factors for concrete, reinforcing steel and structural steel were in accordance with Caltrans (2011) for static design and Caltrans (2013) for seismic design. The material resistance factors for shotcrete lining design were in accordance with the *Technical Report 63: Guidance for the Design of Steel-Fibre-Reinforced Concrete* (Concrete Society, 2007).

5.0 Preliminary Design Concepts of the Segmental Lining

5.1 Static Analysis and Design

5.1.1 Analysis Sections

Figure 1 shows that the tunnels in Reaches 1, 2, 3, 5, 6 and 7 are in rock. The rock types in these reaches are the Puente, Fernando, and Topanga formations. An appreciable thickness of alluvium overlies the rock in these reaches up to about Sta. 1585, after which the tunnels are entirely in rock. The total cover over the tunnel crowns in these reaches ranges from approximately 60 feet at the South Portal to a maximum of 278 feet at Sta. 1610 in the Topanga formation. Groundwater above tunnel crowns ranges from approximately 35 feet at the south portal to 140 feet between Sta. 1590 to 1630. In addition to the highest rock cover, the tunnels at Sta. 1610 therefore also have the highest groundwater head in rock. This section, shown on Figure 2, was therefore selected for analysis of the tunnels in rock for both the mean and lower-bound geotechnical parameters set out in Tables 1 and 2. In Figure 2, the “w” stands for weathered rock, and the “f” stands for fresh rock.

Reach 4 has mixed-face conditions with the tunnels partly in rock and partly in alluvium. The tunnels are entirely in alluvium in the northern portion of Reach 4 (beyond Sta. 1645). In Reach 8 again the tunnels are entirely in alluvium. Between Reach 8 and the northern portion of Reach 4, the thickest alluvium cover occurs at Sta. 1650 in Reach 4 with approximately 160 feet of alluvium above the tunnel crowns. The highest groundwater between Reach 8 and the northern portion of Reach 4 also occurs in the vicinity of Sta. 1650, with the groundwater approximately at tunnel crown. This section, shown on Figure 3, was therefore selected for analysis for the tunnels in alluvium for both the mean and lower-bound ground parameters set out in Tables 1 and 2. The alluvium at this section generally consists of sands and silty sands (SP-SM), with sandy silt (ML) and sandy silty clay (CL-ML) interlayers. The granular layers are medium dense to dense and the cohesive layers are generally stiff.

Fault zones typically impose large loads on tunnels, due to the weak nature of the rock present in these zones and the potential for squeezing conditions. The Raymond fault is present in Reach 5, the San Rafael fault is present in Reach 7, and the Eagle Rock fault is at the break between Reaches 5 and 6. The ground cover at these locations, however, is only approximately 150 feet, 100 feet, and 150 feet respectively. Even if the load on the tunnel due to squeezing reaches full overburden load, the total ground load on the tunnel would be on the order of only 100 feet to 150 feet of ground. Due to this low cover, a check was first performed of the ground loads on the rock section with the highest rock cover of 278 feet as shown in Figure 2 and this load was compared to the “full ground load” assumption in the fault zones.

The selected sections are expected to produce the highest loading on the tunnel in rock, soil, and the fault zones. Additional sections will be analyzed during future design phases to confirm that the sections selected during this concept design phase provide an upper bound for the loading on the tunnel lining.

5.1.2 Assessment of Ground Loads

Ground loads on tunnel linings can be estimated based on the empirical Terzaghi's (1946) approach (Proctor and White, 1968; Deere et al., 1969) and on the numerical method using the concept of ground relaxation or convergence-confinement (Carranza-Torres and Fairhurst, 2000; and Graziani et al., 2005). Both these approaches were used for estimating ground loads on the tunnel lining.

5.1.2.1 Loads Based on Terzaghi's Approach

These loads are empirical loads on tunnel supports based on a qualitative description of the rock and soil and were originally proposed by Terzaghi (1946). The rock loads gradually increase from a "Hard and Intact" rock to a "Completely Crushed but Chemically Intact" rock. Squeezing and Swelling rock is also included. Proctor and White (1968), in their publication on rock tunneling and steel supports, summarize these as originally presented by Terzaghi. Deere et al. (1969) proposed loads that are essentially similar to the Terzaghi loads but somewhat reduced for certain rock types. Rose (1982) proposed loads that are even lower than those proposed by Deere et al. Table 3 shows a summary of the rock loads from these sources.

Rock Section

While the Topanga formation selected for analysis would not produce as much load as a "completely crushed" rock, the load from it can be conservatively estimated as per a "very blocky and seamy" classification. For this classification, for a circular tunnel and the upper-bound values from Table 3, the rock load range would be approximately 2.2 diameters, or 130 feet, of rock based on Terzaghi and Deere approaches. Lateral loads can be taken as the vertical load multiplied by the lateral stress coefficient in Tables 1 and 2.

Soil Section

Using the sand and gravel classification for the alluvium, for a circular tunnel and the upper-bound values from Table 3, the soil load range would be approximately 2.8 diameters, or 160 feet, of soil based on the Terzaghi and Deere approaches. Lateral loads can be taken as the vertical load multiplied by the lateral stress coefficient in Tables 1 and 2.

5.1.2.2 Loads based on Ground Relaxation

Ground relaxation refers to the amount of ground load that is redistributed by the ground prior to the installation of the lining. In a tunnel excavated with a pressurized-face TBM, the applied face pressure controls ground movements by simulating the in-situ soil and groundwater pressures. Tail void grouting also controls ground movements prior to installation of the segmental lining ring. Even so, some ground convergence into the tunnel excavation is inevitable at the face and into the shield and tail void gap. As the ground converges into the unsupported tunnel excavation, some of the ground load is redistributed by the ground. The remaining load appears on the tunnel lining. A ground reaction curve can be used to estimate ground relaxation occurring prior to installation of segmental lining and the ground load on the lining.

Rock Section

Figure 4 shows the ground reaction curves generated based on the mean and lower-bound ground parameters for the rock section. PLAXIS 2D (2010) was used to generate the ground reaction curves. From the ground reaction curves, it can be seen that a ground relaxation of approximately 50% would occur when about 50% of total elastic deformation has developed following tunnel excavation. This magnitude of deformation can be generally assumed to correspond to the level of ground relaxation that would occur prior to the installation of segmental lining. Therefore, the total ground loads on the segmental lining can be determined based on a ground relaxation of 50% in the numerical analyses. With this ground relaxation value, an estimated ground load is equivalent to about 140 feet of ground cover for the rock section. This ground load appears to be consistent with what is estimated based on the Terzaghi's approach discussed above.

For reference purposes, ground relaxation values of about 50% to 60% for tunneling in weak rock were assumed in the initial support design for the Caldecott 4th Bore Tunnel (JA, 2007) and the Transbay DTX mined tunnel (JA, 2010), respectively. The weak rock in these projects have strength and stiffness comparable to the Topanga and Puente Formations. The tunnel diameters for the above projects are in the 45- to 50-foot range and are comparable to the SR-710 Freeway Tunnel size. Though these tunnels were designed based on the SEM (Sequential Excavation Method, also called New Austrian Tunneling Method [NATM]) and the ground relaxations for a SEM tunnel could be different than that for a TBM tunnel, our experience from these previous projects provides a reasonable basis for the estimated ground relaxation of 50% for the Freeway Tunnel segmental lining. During future design phases, the allowable values of relaxation will be investigated in more detail with improved geotechnical data. The various formations through which the tunnels pass will also be included in the investigation.

Soil Section

Figure 5 shows the ground reaction curve for the mean and lower-bound ground parameters for the soil section. PLAXIS 2D (2010) was used to generate the ground reaction curve. The curves for the lower-bound and even the mean parameters show the potential for excessive ground movements into the tunnel excavation for a ground relaxation larger than about 10% to 15%. This implies that the tunnel liner would need to be installed and grouted in place before any significant relaxation and associated ground movements can occur. With this result, full ground load was assumed in the tunnel lining design in alluvium. This corresponds to 160 feet of ground load, which is also in keeping with the loads estimated using the Terzaghi and Deere approaches described previously.

5.1.2.3 Fault Zone Section

It was discussed earlier that the total ground cover in the fault zones is also within the same range of 100 to 150 feet of rock. If the worst case of "full ground load" is assumed in the fault zone, due to squeezing ground, the fault zone would also produce a load in the range of 100 to 150 feet of rock. From the above discussion on loads on the rock sections, the design rock load is approximately 130 to 140 feet of rock based on the Terzaghi and the ground relaxation approaches. A lining designed for this amount of rock load is therefore expected to be adequate to withstand loads in the rock sections and also in the fault zones. Further discussion of the tunnel lining in the fault zones is discussed in *Preliminary Design Concepts for Fault Crossing* (JA, 2014e).

5.1.2.4 Loads from Unstable Rock Wedges/Blocks

Rock wedges or blocks that are daylighted by tunnel excavation can also produce loads on the lining. These wedges form along pre-existing planes of discontinuities in the rock such as joints, fractures, and bedding planes. Such failures, however, typically only result in jointed hard rock that has significant strength and stiffness. At the rock section determined critical for ground loads, the rock is weak, such that it will undergo a shear/plastic failure before slip along the discontinuities can occur. As a result, loads due to rock wedges or blocks are not considered to result in loads in excess of those determined by Terzaghi loads/ground relaxation. If future geotechnical investigations indicate the potential for this type of failure in the rock, then this loading will be considered in lining design.

5.1.3 Tunnel Lining and Ground Material Models

The segmental lining was modeled as a structural beam element with the parameters listed previously. The segmental lining joints were directly modeled as hinges, with axial and shear capacity but no moment capacity. This procedure was adopted in preference to modeling the lining as continuous but with a reduction in the moment of inertia to account for segment joints (Muir-Wood, 1975). The alluvium was modeled as an elastic-perfectly plastic material that behaves according to a Mohr-Coulomb strength envelope (Plaxis 2D, 2010). The weathered rock was also modeled as a Mohr-Coulomb material, but with the strength envelope parameters derived from the corresponding Hoek-Brown parameters. The fresh rock was modeled as a material following a

Hoek-Brown strength envelope (Plaxis 2D, 2010). All materials were modeled as drained. When more geotechnical information becomes available, undrained materials, if they exist, can be included in the modeling. The lining was modeled as a watertight element to account for the water pressure loads on the lining.

The Hoek-Brown model relates the major and minor principal stresses in the rock mass at the point of shear failure in the rock mass. Tension failure is also included in the model. The primary input parameters for the model are the uniaxial compressive strength of the rock and the Geological Strength Index (GSI) of the rock mass. The GSI is a parameter that depends on the spacing of rock joints and the condition of the joints. The model is shown graphically in Figure 6. The Mohr-Coulomb material model also relates the major and minor principal stresses at the point of shear failure in the soil. This model is typically applied to soils, and no tension is allowed in the soil mass. The parameters that define the strength envelope are the cohesion and friction angle of the soil. The model is graphically shown in Figure 7. Note that the Mohr-Coulomb strength envelope is also non-linear, similar to the Hoek-Brown model, but is typically assumed to be linear as shown in Figure 7.

5.1.4 Effect of the Internal Structure

The internal structure would be supported on the lining and attached to it, which would induce forces in the lining. Conversely, such restraint would also induce forces in the internal structure. To avoid these effects, the vertical walls of the internal structure may be designed with a flexible connection between the wall top and the lining that would allow radial lining movement under static loads. With this connection, the vertical walls cease to become a point of restraint. The upper horizontal deck is not continuous across the tunnel diameter. Therefore, this slab would not offer restraint to lining movement. Under these conditions, the lining may be analyzed without the internal structure. The connections between the internal structure and the lining are described in *Preliminary Design Concepts for the Freeway Internal Structure* (JA, 2014d).

5.1.5 Methodology

Lining design was performed using PLAXIS 2D (2010). The program uses a finite-element technique to model soil structure interaction. Ground relaxation is fundamental to the lining design, as was discussed in Section 5.1.2. The program can induce a specified amount of ground relaxation prior to lining installation and can therefore be used to determine the lining load. The analysis for the rock and soil sections were run with the relaxation values discussed previously. The typical modeling procedure for the PLAXIS analyses is outlined below. The program output, among other items, is the moment, thrust, and shear in the lining.

- *Stage I*: Establish initial stress condition prior to excavation with the selected K_0 values and a specified water table.
- *Stage II:* Excavate one tunnel. Apply the selected relaxation (load carried by ground alone) to the unsupported excavation.
- *Stage III:* Install lining and activate the remaining portion of ground stresses.
- *Stage IV:* Repeat for the second tunnel excavation (applicable to twin-bore option only).

5.1.6 Analysis Results and Lining Design

A moment-thrust interaction analysis was performed with 28-day concrete strengths analyzed of 6000 psi, 7000 psi and 8000 psi. Overall, thrust governs lining design, which is expected given the number of joints in the segmental lining ring. With ten joints, lining moments are greatly reduced. The reinforcing ratio for these strengths was maintained at 0.5% since moments do not control lining design and the bulk of the thrust is resisted by the concrete section alone. The reinforcing does not provide any significant contribution to thrust resistance. For 6000 psi, the lining thickness required is in the range of 34 inches. For 7000 psi and 8000 psi the lining thickness required is 30 inches and 26 inches, respectively. The 30-inch-thick, 7000 psi lining or the 26-inch-thick 8000 psi lining both appear to be reasonable strength-thickness combinations from design and construction standpoints. The 6000 psi concrete strength leads to a very thick lining. At this stage, the 7000 psi, 30-inch-thick lining is recommended pending a more detailed design and detailed cost estimates of strength-thickness combinations. Figure 8 shows the moment-thrust interaction for the lining.

Note the 0.5% reinforcing ratio is the main segment reinforcing; other reinforcing such as distribution reinforcing in the longitudinal direction, segment joint reinforcing (see Section 5.3), and stirrup reinforcing would also be required. A check was performed on the shear in the lining and it was determined that shear does not control design of the lining.

5.1.7 Foundation Corbels

The corbels that support the internal structure would transfer loads to the segmental lining. The internal structure is designed with pinned supports at the corbels (refer to JA 2014d also the drawings in Attachment A). With this connection, the primary load from the internal structure is vertical loads with horizontal loads of a smaller magnitude. These loads are transferred into the lining via drilled and grouted dowels. Preliminarily design indicates that four #10 dowels would be required in each corbel at a 12-inch longitudinal spacing. The corbels would induce local forces in the segmental lining; however, as compared to the thrust and moment in the lining from ground loads for example, these forces are minor and do not have a significant impact on the segmental lining design.

5.1.8 Lining Check for TBM thrust and Torque

The TBM thrust and torque that the lining would be subject to was preliminarily estimated to be approximately 58,000 kips and 33,000 kip-feet, respectively. In comparison, the TBM for the SR-99 project currently under construction in Seattle, WA, has a maximum thrust of 44,000 kips and a torque of approximately 38,000 kip-feet. Note that the segmental lining would not be required to resist all the torque; the skin friction between the TBM tail shield (sitting on the excavation) and the ground would account for a significant portion of the resistance.

For the thrust, assuming that the number and arrangement of the thrust jacks–56 jacks total, and bearing shoe dimensions of approximately 16 inches by 32 inches–to be similar to the SR-99 TBM, the bearing stress under each jack is approximately 2000 psi, which is well within the allowable compressive stresses for 7000 psi concrete, estimated to be approximately 3800 psi. Reinforcing would be required at the longitudinal joints. A discussion regarding this reinforcement is provided in Section 5.3. The torque induces shear in the longitudinal connectors. The estimated shear on the longitudinal connectors is on the order of 45 kips each, assuming that all the torque is resisted by the lining. The shear bicones (e.g., Anixter SOF 345 bicones) alone are expected to have a resistance of 84 kips each; bolts would further contribute to shear resistance. As such, bolts and shear bicones are expected to provide adequate resistance to torque. A detailed analysis of the connectors will be undertaken during subsequent design phases.

5.2 Preliminary Seismic Design

5.2.1 Analysis Sections

The same rock and soil sections were analyzed in order to evaluate response of the lining during seismic events. These are the same sections considered for the static design as shown in Figures 2 and 3. Effective dynamic ground properties are considered for the seismic analysis.

5.2.2 Assessment of Seismic Loads

The effect of seismic ground motions caused by the design earthquakes on the segmental lining is evaluated to check its performance. Parameters for obtaining seismic loads are summarized in Tables 4 and 5. These parameters were developed based on information in the Preliminary Geotechnical Report (CH2M HILL, 2014). The response spectra and other parameters presented in that document were derived based on procedures outlined in Caltrans (2013). Two levels of seismic event, consisting of a Safety Evaluation Earthquake (SEE) and a Functional Evaluation Earthquake (FEE), must be considered for the Freeway Tunnel design.

The SEE is a seismic event that has a 5 percent chance of being exceeded in 50 years, which is equivalent to a 975-year (1,000-year nominal) return period earthquake. The performance requirement under the SEE is that the lining should be able to survive the seismic event without collapse, and inelastic behavior is permitted in the structure. The FEE is a smaller seismic event with a 100-year return period. The performance requirement under FEE is that the structure remains fully functional with minimal damage.

Seismic design loads on the lining result from seismic ground motions. Under such motions, the tunnel lining is subjected to a racking effect (Wang, 1993; Hashash et al., 2001; Hashash et al., 2005). The lining distorts or ovals under the shearing deformations induced by the ground motions. While ovaling may be caused by waves propagating horizontally or obliquely, vertically propagating shear waves are the predominant cause of ovaling.

The seismic effect of ovaling deformations on the tunnel lining system may be evaluated using closed-form elastic solutions for circular tunnels or numerical modeling. The loads from the seismic analysis would then be combined with the loads from static analysis to determine if the structural design criteria are satisfied. The seismic thrusts were both added to and subtracted from the static thrusts in order to obtain the worst case range in lining loads. The load combination to be considered is the EXTREME I event, as explained previously. For this combination, the mean or expected geotechnical properties are considered.

At the location of the fault crossings, the final lining would include a special design for the fault offset loads. Refer to *Preliminary Design Concepts for Fault Crossing* (JA, 2014e) for more detail.

5.2.3 Ground and Lining Material Models

The ground materials for the seismic case are considered to be linear-elastic, weightless, and drained for both geologic sections. This is a typical assumption for racking analyses (Wang, 1993; Hashash et al., 2001; Hashash et al., 2005). Effective dynamic ground properties are considered—namely, Young's modulus, shear modulus, primary and shear wave velocities, and peak ground acceleration (PGA) and peak ground velocity (PGV). Dynamic properties of the various formations for analysis are shown in Tables 3 and 4. The segmental lining was modeled as described for the static case.

5.2.4 Methodology

5.2.4.1 Numerical Model

The seismic effect of ovaling deformations on the tunnel lining was evaluated using the numerical modeling software Plaxis 2D (2010). A model was used to analyze potential ovaling deformations of the tunnel during the SEE event. For each geologic section, the estimated free-field seismic displacement was applied to the ground in order to evaluate the resulting forces on the tunnel lining.

In the analysis, ground deformations due to the free-field shear strain were applied to the model in order to estimate the resulting structural forces and deformations. The effect of seismic shear wave was simulated by applying displacement at the far external boundaries of the model. Displacements are estimated based on the free-field shear strain of the ground (i.e., ratio of PGV to effective shear wave velocity).

The segmental lining was modeled as a series of two dimensional structural beam element with the parameters listed previously. The segmental lining joints were directly modeled as hinges, with axial and shear capacity but no moment capacity. This procedure was adopted in preference to modeling the lining as continuous but with a reduction in the moment of inertia to account for segment joints.

The interface between the lining and the ground may be modeled as a full-slip, no-slip, or in-between case. The in-between case is the condition for most tunnels; however, the no-slip case is more conservative and was considered in the model. The no-slip condition was modeled by removing the interface between the lining and ground.

The internal structure was included in the model, since this represents the completed condition under which the earthquake would occur. This also provides the forces in the internal structure when it displaces sympathetically because of the racking deformation of the tunnel. A discussion on the racking forces in the internal structure can be found in *Preliminary Design Concepts for the Freeway Internal Structure* (JA, 2014d). The connections between the upper slab of the internal structure and the tunnel lining were modeled as pin connections, while the connections between the top walls and the lining were modeled as relatively flexible connections that provide limited resistance in tension and compression before yielding. These flexible connections were used to simulate the potential behavior of the bracket connections at the top of the wall that are bolted to the lining at regular spacing along the wall.

5.2.4.2 Closed-Form Solutions

Closed-form solutions were used to check the numerical model results. These methods for estimating ground-structure interaction are based on the assumptions that the tunnel lining is an elastic, thin-walled tube located in ground consisting of an infinite, homogeneous, and isotropic medium (Wang, 1993). Closed-form solutions summarized in FHWA (2009) were used to develop initial estimates of the forces and deformations induced in the tunnel lining because of seismic shear waves. These solutions assume either full-slip or no-slip conditions existing along the interface between the lining and the ground. A no-slip interface is assumed for the determination of maximum thrust in the lining, since a full-slip interface assumption may significantly underestimate thrust.

Two closed-form methods by Wang (1993) and Penzien and Wu (as summarized in Hashash, et al., 2005) were used to estimate the forces and deformation in the lining cross section. For both methods, free-field strain is calculated as a ratio of seismic peak ground velocity and effective shear wave velocity in the ground medium. The closed-form solutions are a function of free-field deformation and take into account the relationship between ground stiffness and the extensional and flexural stiffness of the tunnel lining.

Seismic waves intersecting the tunnel at an angle not perpendicular to the longitudinal axis of the tunnel would cause strains in the lining because of axial compression-extension and curvature (bending) along the tunnel. Hashash et al. (2001) provides a method to determine the longitudinal strains generated by axial and bending deformations, obtained by treating the tunnel as an elastic beam. In the rock section, the free-field strain value is assumed to be adequate as an upper-limit estimate, but in soil, a ground-structure interaction approach is used to estimate strain and resulting stresses in the longitudinal direction.

5.2.5 Results

Results from the numerical and closed form solutions for the tunnel lining were comparable in terms of moment, thrust, and shear in the lining. Figure 9 shows the moment-thrust interaction for the lining for the EXTREME I combination. The seismic results presented here are from the numerical analyses. The solid lines are the limits with material resistance factors per the Caltrans (2011) code and are as before for the static case. For seismic cases, Caltrans (2013) allows the following to assess the moment thrust capacity:

- Setting all the material resistance factors to 1.0.
- Use of expected material strengths instead of the minimum specified strengths for concrete and steel. For concrete, this expected material strength is 30% greater than the specified strength. For reinforcing steel, the expected yield strength is approximately 13% greater than the specified yield strength. The moment-thrust envelopes that result from setting the resistance factors to 1.0 and using the expected material strengths represent the nominal capacity of the section.
- Inelastic behavior is permitted. This means that the moment thrust interaction points can fall outside the nominal limits, provided the section meets the inelastic provisions of the code.

Of the permitted three provisions above, only one—setting the resistance factors to 1.0—was used initially to assess seismic capacity of the section. The dashed lines in Figure 9 show the capacity resulting from this provision. It can be seen that all points actually fall within the nominal limit itself. The provisions related to increased material strengths and inelastic behavior need not be invoked. This indicates that the lining remains in the elastic range for the SEE seismic event. The 30-inch-thick lining as designed for the static case is therefore adequate for the seismic case; the seismic case, in fact, does not govern design. As with the static case, a check was performed on the shear in the lining; however, shear does not control lining design.

5.3 Segmental Lining Design and Details

This section provides a description of some of the design details of the segmental lining shown on the drawings.

5.3.1 Joint Design

The segment joints in both the radial and longitudinal direction would be subjected to bursting forces. The radial joints are subjected to the segment thrust; the longitudinal joint is subjected to the thrust from the TBM jacks during each advance cycle when the TBM "shoves off" the segmental lining. The contact between the segments in one ring and in between rings is typically smaller than the overall segment width. Because of this, the thrust tends to spread out and cause localized bursting forces. These forces have the potential to cause a tensile splitting failure in the segment near the joint. An example is shown in Figure 10; the cracks can be seen spreading out from the contact between the segments. Reinforcing that crosses the failure planes is required to confine the concrete and prevent the splitting failure. Design procedures listed in Swartz et al. (2002) can be used for segment joint design.

Compression of the gaskets, in order to seal against the external hydrostatic pressures, would result in an additional localized load on the segments at the extrados joint face. This localized load would also cause an edge shear type failure near the gasket groove (Figure 11). This failure has to be resisted by the concrete alone, since reinforcing is difficult to place in the zone near the gasket groove. The magnitude of the line load would be dependent on a number of factors, including the gasket material stiffness, the geometry and design of the gasket, the gasket groove geometry, and the amount of compression required to seal against the hydrostatic pressures.

5.3.2 Gaskets

Ethylene propylene diene monomer (EPDM) gaskets are a feasible option for providing a nearly watertight lining. The gaskets are glued to the segments inside the gasket groove. These gaskets have been successfully used on the Arrowhead Tunnel project (East and West tunnels) in Southern California to resist up to 900 feet of groundwater pressure for the East Tunnel and 575 feet for the West Tunnel. These water pressures are significantly higher than those anticipated for the SR-710 tunnels (200 to 250 feet). Additional details on the measures to control water and gas infiltration was described earlier in this TM.

5.3.3 Longitudinal and Radial Connectors

These connectors are primarily used to maintain gasket compression during ring build. The connectors also aid in aligning the segment pieces in one ring with the adjacent completed rings, and support the weight of the segment pieces during ring-build in case of a power loss. Because of the considerable segment thickness of 30 inches, high-capacity bolts would be required in both the radial and longitudinal directions. Dowels have not been shown on the drawings in either direction as these would not have the capacity to support the segment weight. A shear bicone, shown on the drawings, would be used as an additional longitudinal connector. The primary reason for this connector is to aid in resisting the torque that would be applied to the segments by the TBM as discussed previously; however, other means of resisting the torque should be investigated in future design phases.

5.3.4 Centering Cones and Injection Inserts

The segments as shown on the drawings are designed to be erected by a vacuum erector as is the case with most modern TBMs. The centering cones are used by the erector to push the segment against the adjacent and previous segments to compress the gaskets. Because of the high forces that the area around the cones would experience, the concrete around the cones may require confinement via spiral reinforcing.

Each segment piece, except the key segment, has one injection insert. These are intended to be used for secondary or contact grouting of the segments. The primary annulus grouting of the segments would be through the TBM tail shield. The insert/port provides the ability to perform secondary grouting through the segments to ensure that the annulus around the segments is completely grouted. Typically, the port/insert is drilled through and then the grouting is performed. Prior to drilling, these ports typically have a guillotine valve or equivalent installed to prevent uncontrolled inflows of material during the drilling process.

5.3.5 Plywood Packers

Plywood packers are sometimes used in the construction of tunnels with segmental linings. The primary benefit of the packer is that it acts as a cushion between adjacent segment pieces and between adjacent rings. The packers are glued on the appropriate radial and circumferential joints of the segment pieces. The radial joints are designed to rotate, and there is a significant benefit to incorporating a crushable packer to allow this rotation to occur without introducing large eccentricities and concentrated loading near the edge of a joint. Similarly on the circumferential joint, the packer assists in distributing loads across the segment bearing surface on the trailing edge, and can improve planarity to minimize point loading. For permanent structures, Marine Grade plywood packers are very durable. The packer thickness typically used is approximately 1/8 inch. With regards to fire resistance, the compression of the packing changes it quite a bit and substantially reduces voids and open spaces; only the wood grain remains. Even if the packer is exposed to a fire, there is very little material to burn. At this time, plywood packers are not a requirement of the design; however the issue should be investigated in future phases of this study.

6.0 Preliminary Design Concepts of Cross Passage Structures

6.1 General

Cross passages are expected to have a two-pass lining system consisting of the initial shotcrete lining and the cast-in-place concrete final lining. A water/gas proofing membrane would be installed between the initial and final linings.

Cross passages are expected to be excavated using the Sequential Excavation Method (SEM), also known as the New Austrian Tunneling Method (NATM). The key initial support and presupport elements considered for the cross passage excavations include cement-grouted rock dowels (RD), fiber-reinforced shotcrete lining (FRS), spiles (SP), and fiberglass face dowels (FD). Ground improvement measures using permeation grouting, chemical grouting, or ground freezing is also expected for groundwater control and to ensure stability for the excavation in alluvium. This design concept for the cross passages is described more in detail below; however, there may be other feasible concepts which can be explored in subsequent phases of this study. As discussed in Section 4, the operational need should be further analyzed to optimize the design of these passages in future studies.

6.2 Preliminary Design Concepts of Initial Support

6.2.1 Design Sections Selected for Analysis

For the design of initial support installed in cross passages, ground conditions at various cross passage locations have been categorized into three ground classes. Each ground class consists of certain rock mass types (RMTs), which are expected to respond similarly to tunneling operations and would require similar support types (STs). Ground Class 1 represents the better rock ground conditions (slightly weathered to fresh; weak to very strong rock; massive, moderately jointed to moderately blocky and seamy) over the anticipated range of ground conditions along each tunnel alignment, while Ground Class 2 represents the poorer rock ground conditions anticipated (highly to moderately weathered; extremely weak to very weak rock; massive, moderately jointed to completely crushed). Ground Class 3 represents the soft ground or mixed-face (alluvium over bedrock) ground conditions anticipated. Table 6 summarizes the three ground classes defined, associated RMTs, overburden depths, and anticipated general ground conditions. Cross Passages 1, 2, and 6 fall under Ground Class 2; Cross Passages 3 and 4 fall under Ground Class 1; and Cross Passage 5 falls under Ground Class 3. Additional details regarding ground classes and rock mass types can be found in *Tunnel Ground Characterization* (JA, 2014b).

Three representative sections, one for each of the three ground classes, were selected for the preliminary design analyses as shown on Table 6. The locations selected for preliminary design are located at Cross Passages 1, 3, and 5.

6.2.2 Assessment of Ground Loads

Ground loads on the initial support were assessed based on the principles of SEM. During excavation, it is assumed that the tunnel is drained, meaning that the effect of groundwater pressure and seepage on the initial lining are minimal and can be ignored. Since the sections selected for analysis can be considered deep sections because of the size of the cross passage, the ground relaxation ahead of the tunnel face is assumed to be 50 percent. This value will be evaluated more rigorously during future design phases based on additional geotechnical information and three-dimensional modeling of the cross passage excavation.

The pillar between the two bored tunnels in which the cross passage is excavated would experience an increase in vertical stress following the running tunnel excavation because of the vertical stresses flowing around the tunnels and concentrating in the pillar. The average increase is on the order of 25%, as determined from the PLAXIS analysis of the segmental lining in Section 5.0. For cross passage analyses, in addition to the ground loads at the design sections, this increase in vertical loading was also accounted for in the analysis and design.

6.2.3 Shotcrete Lining and Ground Material Models

The following key modeling assumptions are made in the FLAC analyses:

- The modulus of elasticity of the shotcrete varies from 725 to 2,175 ksi (5 to 15 GPa) to account for the effect of early-age creep (Max and Bruno, 2003).

- The response of the rock to static loading is modeled to be elastic-perfectly plastic. The plastic response of both soil and rock is governed by the Mohr-Coulomb yield criterion.

6.2.4 Methodology

Numerical methods were used to design the initial support system. The numerical design analyses were carried out using the commercially available computer program FLAC (Fast Lagrangian Analysis of Continua), Version 5.0 (Itasca, 2005). The FLAC analyses were performed using a ground-structure interaction approach to simulate the sequence of underground excavation and initial support installation. The analyses evaluate the behavior of underground openings during the sequential excavation to assess feasible excavation sequences and initial ground support requirements so as to maintain a stable excavation and minimize large convergences of the ground into the excavation.

The typical modeling procedure for the FLAC analyses is outlined below.

- *Stage I:* Establish initial stress condition prior to excavation. Cycle to equilibrium. Where treated soil is modeled, modify ground properties in this zone to account for this provision, and cycle to equilibrium.

- *Stage II:* Excavate the top heading. Relax forces to 50 percent around the perimeter of the top heading to simulate the amount of relaxation that has occurred ahead of the tunnel face. Cycle to equilibrium.

- *Stage III:* Install cable elements to represent rock dowels, if applicable, and install beam elements around the perimeter of the top heading to represent the shotcrete lining. Cycle to equilibrium.

- *Stage IV:* Steps II and III are repeated for the bench excavation.

The program output, among other items, is the structural forces in the initial support elements. For the shotcrete lining the output is the moment, thrust, and shear. For the rock dowels the output is the tension load in the dowel.

6.2.5 Preliminary Design Concepts for Excavation Sequence and Initial Support Requirements

The preliminary design concepts for the support types for each ground class expected along the alignment are detailed below.

6.2.5.1 Support Type 1 for Ground Class 1

Support Type 1 is required for excavation in Ground Class 1 (RMT Tt-1) ground conditions, and is expected to be required for excavation in two pairs of cross passages. Approximate locations of these cross passages are shown in Figure 1.

- **Excavation Sequence.** The top heading would be excavated in a full face with a design maximum round length of 4 feet. The minimum lag between the top heading face and the bench face is 8 feet. The bench/invert excavation would be carried out in a full face with a design maximum round length of 8 feet. In-tunnel groundwater control measures would be implemented as required during excavation to reduce inflows and hydrostatic pressure on installed shotcrete lining.

- **Support Requirements.** Support Type 1 consists of 12-foot-long, fully grouted, 1-inch rebar rock dowels (Fy= 60 ksi) with a spacing of 4 feet on center and 8-inch-thick fiber-reinforced shotcrete. Face support consisting of 2-inch-thick face sealing shotcrete would be installed as required to support potential unstable wedges encountered in the face.

Figures 12 and 13 show the loads on the rock bolts and the moment-thrust interaction for the shotcrete lining, respectively.

The analyses predict loads in the 1-inch rebar rock dowels ranging from 1 to 9 kips. These forces are about 2 to 15 percent of the yield limit of 59 kips. A moment-thrust interaction diagram for an 8-inch-thick shotcrete lining in the ST1 section shows that this thickness is adequate.

6.2.5.2 Support Type 2 for Ground Class 2

Support Type II is required for excavation in Ground Class 2 (RMTs Tf, Tp-2, and Tt-2) ground conditions, and is expected to be required for excavation in three pairs of cross passages. Approximate locations of these cross passages are shown in Figure 1.

- **Excavation Sequence.** The top heading would be excavated in a full face with a design maximum round length of 3 feet. The minimum lag between the top heading face and the bench face is 6 feet. The bench/invert excavation would be carried out in a full face with a design maximum round length of 6 feet. In-tunnel groundwater control measures would be implemented as required during excavation to reduce inflows and hydrostatic pressure on installed shotcrete lining.

- **Support Requirements.** Support Type 2 consists of 10-inch-thick fiber-reinforced shotcrete. Presupport consisting of 20-foot-long rebar or pipe spiles with a spacing of 12 inches on center would be installed above the crown every other advance as required. Face support consisting of 20-foot-long face fiberglass dowels with a spacing of 4 feet on center and/or 2-inch-thick face sealing shotcrete would also be installed as required to maintain the face stability.

A moment-thrust interaction diagram for a 10-inch-thick shotcrete lining in the ST2 section shows that this thickness is adequate. Rebar spiles are expected to be 1 inch in diameter at 12-inch centers. Pipe spiles are expected to be 2 inches in diameter at 12-inch centers. Figure 14 shows the moment-thrust interaction for the shotcrete lining.

6.2.5.3 Support Type 3 for Ground Class 3

Support Type 3 is required for excavation in Ground Class 3 (alluvium) ground conditions, and is expected to be required for excavation in one pair of cross passages. Approximate locations of these cross passages are shown in Figure 1.

- **Excavation Sequence.** Ground improvement using permeation grouting, chemical grouting, or ground freezing would be required prior to cross passage excavation. After the treated ground reaches a minimum compressive strength of 400 psi, the top heading would then be excavated in a full face with a design maximum round length of 3 feet. The minimum lag between the top heading face and the bench face is 6 feet. The bench/invert excavation would be carried out in a full face with a design maximum round length of 6 feet. In-tunnel groundwater control measures would be implemented as required during excavation to reduce inflows and hydrostatic pressure on installed shotcrete lining.

- **Support Requirements.** Support Type 3 consists of 12-inch-thick fiber-reinforced shotcrete. Presupport consisting of 20-foot-long pipe spiles or canopy pipes with a spacing of 12 inches on center would be installed above the crown every other advance as required. Face support consisting of 20-foot-long face fiberglass dowels with a spacing of 4 feet on center and/or 2-inch-thick face sealing shotcrete would also be installed as required to maintain the face stability.

A moment-thrust interaction diagram for a 12-inch-thick shotcrete lining in the ST3 section shows that this thickness is adequate in conjunction of ground improvement measures. Rebar spiles are expected to be 1 inch in diameter at 12-inch centers. Pipe spiles are expected to be 2 inches in diameter at 12-inch centers. Figure 15 shows the moment-thrust interaction for the shotcrete lining.

6.3 Preliminary Design Concepts of the Final Lining

6.3.1 Design Sections

Two design sections are selected for the final lining evaluation. These two sections are the ST1 and ST3 sections, and represent the anticipated bounding conditions in terms of ground, hydrostatic, and seismic loads. Application of the design for these sections to all cross passages is considered conservative.

6.3.2 Assessment of Loads on the Final Lining

The concept of load sharing is employed to assess static ground loads on the final lining for the cross passages. This design concept has been gaining increased acceptance by tunnel designers in recent years and has been applied to the design of many NATM tunnels worldwide (Sun et al., 2013). With this concept, the following can be assumed:

- The static ground loads initially carried by the initial support system are supported by both the initial shotcrete lining and the final lining during the design service life of cross passage structures.

- The water pressure loads are carried entirely by the final lining.

- Seismic loads are carried entirely by the final lining, and sharing of seismic loads by the initial shotcrete lining is conservatively neglected.

Two essential components of the initial support system—the rock dowels, if applicable, and portions of the shotcrete lining in contact with the ground—may deteriorate with time if not fully designed and specified to be durable which may be uneconomic. Load sharing of the static ground load between the initial support and final lining is acceptable as long as the long-term capacity of the initial support system is evaluated in a manner that realistically considers the potential for long-term degradation of the structural properties of the initial support system. The actual amount of load transferred to the final lining depends on the relative stiffness of the initial and final linings (Sun et al., 2013) and the potential for future deterioration of the structural properties of the initial support system.

FLAC was used to assess load sharing of the static ground load with the following assumptions:

- The degradation of the initial shotcrete lining and corresponding reduction in axial and bending stiffness is modeled by:
  - reducing the cross-sectional area by 50 percent, and
  - reducing the moment of inertia by 100 percent.
- Rock dowels, if applicable, are assumed to fully degrade in the load-sharing evaluation (Hoek, 2002). Generally, rock dowels installed during tunnel construction are not considered permanent and are subject to corrosion during the tunnel design life.
- The final lining is considered to carry the full hydrostatic pressure, where applicable. In reality, the hydrostatic pressure would cause additional deformations of the final lining, which could affect the magnitude of load sharing. For simplicity, the effect of hydrostatic pressure on the load sharing of ground loads is not considered.

6.3.3 Lining and Ground Material Models

The final lining was modeled using 4,000 psi concrete. Initial lining and ground material models are the same as those mentioned previously for initial support design.

6.3.4 Static Design Methodology

The modeling steps used in the FLAC analyses for the final lining subject to static loading condition are as follows:

- *Step I:* Restore the saved FLAC file that contains internal forces (thrusts, shears, and moments) in the initial lining developed during and following tunnel excavation.
- *Step II:* Remove all structural elements that represent rock dowels, if included in the model.
- *Step III:* Install interface elements and structural elements that represent the final lining and waterproofing membrane.
- S*tep IV:* Reduce the initial lining properties (cross-sectional area and moment of inertia). These reduced properties are fixed during cycling.
- *Step V:* Reduce the internal forces (thrusts, shears, and moments) developed in the initial lining during tunnel excavation by 100 percent prior to cycling. These reduced internal forces (thrusts and shears) would change during cycling based on the relative stiffness of each of the linings.
- *Step VI:* Cycle to equilibrium.

6.3.5 Seismic Design Methodology

The effect of seismic ground motions caused by the design earthquakes on the cross passage final lining is evaluated to check its performance. The seismic analysis for the cross passages evaluates transverse effects of the SEE level design earthquake using closed-form solutions and a FLAC racking analysis. The background on these methods is presented in Section 5.

The forces (thrusts, shears, and moments) calculated from the racking analyses are then combined with those from the static analyses to evaluate the cross passage final lining seismic loads.

6.3.6 Results of Preliminary Analysis

The results from the analyses can be summarized as follows:

- The proposed final lining consisting of a 15-inch arch and a 24-inch invert would be adequate to support the potential combinations of static dead, ground, and hydrostatic loads based on a moment-thrust interaction diagram. Reinforcing ratio is expected to be 0.5% in the arch and 1% in the invert. This reinforcing is the main reinforcing; other reinforcing such as distribution reinforcing in the longitudinal direction and stirrup reinforcing would also be required.

- Under the combined static and seismic loading conditions (considering the SEE design earthquake and without considering the expected material strengths), the moment-thrust interaction in both the final lining arch and the invert are within the nominal envelope. Per Caltrans (2013) this means that the final lining shows essentially elastic behavior during the SEE event; no inelastic behavior of the final lining is expected.

Figures 16 to 19 summarize the moment-thrust interaction for the final lining arch and invert.

6.4 Preliminary Design Concepts of the Breakout Temporary Support

A temporary support scheme would be required at the cross passage breakout. This support provides a means for the thrust in the lining to flow around the opening that would be cut into several of the lining rings. Given the size of the tunnel and the thrust in the lining, a structural steel header/footer beam system supported by struts appears to be most suitable. This support scheme is shown on the drawings in Attachment A. The thrust in the lining is transmitted to the header and footer beams via stubs that are welded to the steel segments at the opening. The beams then transfer load to the struts. The arrangement shown provides enough clearance between the two center struts to accommodate a roadheader for performing the cross passage excavation.

Steel segments or concrete segments with embedded steel beams would be required around the breakout so that the segment is compatible (e.g., for welding) with the steel breakout support elements. In concept, the steel segments would consist of 1- to 1.5-inch-thick stiffener plates welded in the form of a grid. The four sides and the extrados would be closed by cover plates. The intrados would stay open. Similarly, concrete segments would contain steel beams. The steel beams would be the primary load-carrying element in the segment. All steel surfaces exposed on the intrados of the tunnel would be shotcreted for fire protection after completion of construction.

Preliminary sizing of the system using 50 ksi yield steel indicates that the header and footer beams would be double W36 to W40 wide flange beams, and the strut would be a 36-inch square welded box composed of 1.5-inch-thick plates. The stubs are W27 or W33 wide flange beams.

As shown on the drawings in Attachment A, the breakout support system should be preloaded to 25% to 50% of the total expected design load, prior to cutting out the segments. This takes out any slack in the system and keeps deformations to a minimum during load transfer from the segments to the breakout system. Hydraulic jacks can be used for this purpose. The rings in the vicinity of the breakout (say, ten rings) would have strain gauge and load cell instrumentation. This would provide vital information on the total design load that can be expected on the system and also on the behavior of the system during cross passage construction.

6.5 Preliminary Design Concepts for Permanent Ring Beam

A permanent reinforced concrete ring beam would be required around the cross passage opening at the location where the cross passage frames into the segmental lining. Due to the shape of the opening (refer to Attachment A), the shape of the ring beam is not a circle, but the concept of the ring beam is being used. This beam would permanently support the thrust in the lining at the cross passage intersection. This beam would be cast along with the final lining for the cross passage. It would be constructed such that the segmental lining would have complete bearing on the beam. Post-construction grouting with a high-strength nonshrink grout may be required between the ring beam and the segmental lining to restore contact that may be lost during shrinkage of the ring beam. Other positive connections between the beam and the lining, such as drilled and grouted bars for concrete segments or shear studs for steel segments, would also be required.

Considering the thrust in the segmental lining, and the size of the cross passage opening, the permanent ring beam would be a significant structural element. A STAAD (Bentley, 2012) beam-spring model of the ring beam was used to determine the moment and thrust for the static and seismic load cases. The beam element was modeled with the section properties of the ring beam, and the support springs were based on the properties of the segmental lining since the lining loads the ring beam but also supports it. The beam size assumed for modeling was 5 feet deep by 6 feet wide. The moment of inertia of the ring beam was reduced to 70% of the gross inertia, as typical for a column, per Caltrans (2011). The load in the model is the static or the combined static and seismic thrust in the lining. This load is applied along the crown of the ring beam and also along the invert of the beam. Moment-thrust analyses of the ring beam were carried out using 7,000 psi concrete and an average reinforcing ratio of approximately 4%.

Static design of the ring beam was in accordance with Caltrans (2011). The static moment-thrust interaction fell within the factored capacity envelope, and therefore the section is fully elastic for the static case. For the seismic case, the Caltrans (2013) provisions of using resistance factors of 1.0 and expected material strengths were initially used to generate the nominal moment capacity. However, the moment-thrust interaction falls outside the nominal capacity envelope of the section, indicating inelastic behavior. With this result, an investigation is required of the moment-curvature relation of the section. The moment-curvature relation of the section provides (at a given axial load level) the plastic moment, the corresponding curvature called yield curvature, and the ultimate curvature capacity of the section. The moment capacity at yield curvature is the plastic moment capacity of the section. The moment in the section can be larger than the plastic moment; however, the corresponding curvature cannot exceed the ultimate curvature of the section. The ultimate curvature of the section is the point at which the confined concrete crushes or the reinforcing fractures. Additionally, steel and concrete strains cannot exceed the maximums allowed by Caltrans (2013).

SAP2000 V15 (CSI, 2011) was used to determine the moment-curvature relationship of the section using the Mander confined concrete model (Mander et al., 1988) in accordance with Caltrans (2013). Confinement of the concrete was preliminarily modeled using #7 reinforcing bars in a tie and cross-tie arrangement. The combined static and seismic loading from the lining produces moments in the ring beam that are close to the plastic moment capacity of the section at the corresponding axial thrust level. As required by the code, the curvature of the section at the induced moment is well below the ultimate curvature, and the maximum steel and concrete strains are also within the Caltrans (2013) specified values.

In summary, static and seismic design of the beam indicates that the 5- to 6-foot-deep by 6-foot-wide beam with a reinforcing ratio in the range of 4% using Gr. 60 reinforcing is expected to be adequate. The 28-day concrete strength required would be on the order of 7,000 psi. Self-consolidating concrete may be required to avoid practical difficulties in vibrating the concrete, given the high reinforcing percentage. These results are preliminary, and additional analyses at each cross passage location will be required during future design phases with improved geotechnical data in order to refine beam size, reinforcing percentage, and concrete strength.

7.0 Fire Resistance

At the preliminary design phase, the design fire event has not been specified, and therefore no analysis has been performed on the tunnel lining with respect to fire resistance. The appropriate level of protection for the linings should be determined in future studies based on the design fire event. The fire protection should ensure that the internal structure can withstand the design fire event without loss of structural integrity. Depending on the results of the studies, the following measures may be considered to improve the performance of the internal structure exposed to a fire:

- Increase the concrete cover over the reinforcing steel to restrict the temperature rise in the reinforcing steel.
- Use polypropylene fibers in the concrete mix to minimize explosive spalling.
- Specify the use of aggregate that is thermally stable.
- Specify fire-resistant tunnel cladding, such as aluminum silicate insulation boards or a vermiculite cement coating.
- Fire suppression systems.

Prevention of spalling helps improve the overall structural performance of a concrete section during a fire because of the insulation protection the intact concrete cover offers to the reinforcement. The intact concrete limits the internal temperature rise and heat penetration into the concrete. Caner et al. (2005) illustrated this in their analysis of a 300-millimeter-thick (12-inch) concrete lining subjected to a surface temperature of 1100°C (2000°F). The data show that the temperature at a concrete depth of 75 millimeters (3 inches) stays below 300°C (570°F), even when the surface temperature is maintained at 1100°C for 2 hours.

The use of aggregates that are less prone to thermal expansion and splitting at high temperatures has been shown to improve the performance of concrete subject to high temperatures (International Federation for Structural Concrete, 2007). Other aggregate characteristics that improve the performance of concrete subject to high temperatures are: small size, rough surface, and angular shape. Aggregates, listed in order of decreasing thermal stability, are: lightweight, basalt, limestone, and siliceous. High-strength, dense, and low-permeability concretes are also more prone to explosive spalling.

The use of micro-polypropylene (PP) fibers has been shown to effectively reduce explosive spalling (Tatnall, 2002) by limiting the development of high vapor pressures within the concrete. Additional information on the PP fibers can be found in *Preliminary Design Concepts for the Freeway Internal Structure* (JA 2014d).

Fire-resistant cladding directly protects the structure from a fire by isolating the structure from the flames. However, the presence of fire protection panels could significantly influence the assumptions on which the design of the ventilation system is based because the panels reduce the heat absorbed by the structure. The use of fire protection panels and the associated increase in heat load could therefore have a major impact on both the type and cost of the ventilation system to be used. Conversely, the ventilation system has a significant impact on air temperatures and therefore the gradient of temperature penetration into concrete sections. Studies will be required to determine the effectiveness of the ventilation system in limiting the increase in the wall temperature of the final lining during the design fire to comply with NFPA (2011) criteria.

The tunnel lining could be directly exposed to a fire on the upper level roadway and high air temperatures as part of the ventilation ducts. The lining mainly acts as an arch subject to axial loads and has significant additional capacity and redundancy. The use of PP fibers and thermally stable aggregate in the lining should provide adequate protection against spalling and loss of capacity. Another key fire design consideration for the lining is progressive failure. Progressive failure of a structural system is caused by a series of local failures that systematically reduce the overall redundancy of the system until catastrophic failure (collapse) occurs. An example of progressive failure is continued spalling of sections of a concrete lining due to prolonged fire duration. PIARC (1999) recommends the use of fire-resistant panels in tunnels where progressive failure of the lining is possible.

8.0 Summary of Recommendations

The tunnel lining design concepts presented in this TM are considered preliminary. The concepts were developed with limited geotechnical data obtained along the nearly 4.4-mile-long bored tunnel alignment. Additional geotechnical investigations with more closely spaced borings would be necessary to better characterize the geologic and groundwater conditions along the tunnel alignment, including the fault zones. This characterization would be key for final design of the tunnel lining and cross passages. Other design items requiring further evaluation include, but are not limited to, the following:

- Interfaces between the segmental lining and final lining at the north and south portals. The detailing for these interfaces is especially important for seismic loading. Additionally, continuity of waterproofing across the joints would require special details and procedures to limit or prevent inflows at the interfaces.
- Geometry and connections of the vehicle cross passages.
- Evaluation of the method(s) and requirements for ground treatment in alluvium and groundwater control in rock at cross passage locations.
- The design fire event.
- Requirements for sump structures for drainage and potential locations.
- Requirements for corrosion protection from sources such as the ground and groundwater.
- Assessment of TBM loadings on the segments (including thrust and torque applied to the segments) and anticipated segment grouting pressures.
- The tunnel crossings in active or potentially active fault zones would require the installation of special steel segments. Design of the special steel segments for seismic sections is discussed in *Preliminary Design Concepts for Fault Crossings* (JA, 2014e).

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10.0 Revision Log

Revision 0	February 7, 2014	Internal Review
Revision 1	April 16, 2014	Metro/Caltrans Review
Revision 2	June 18, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Figures

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NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

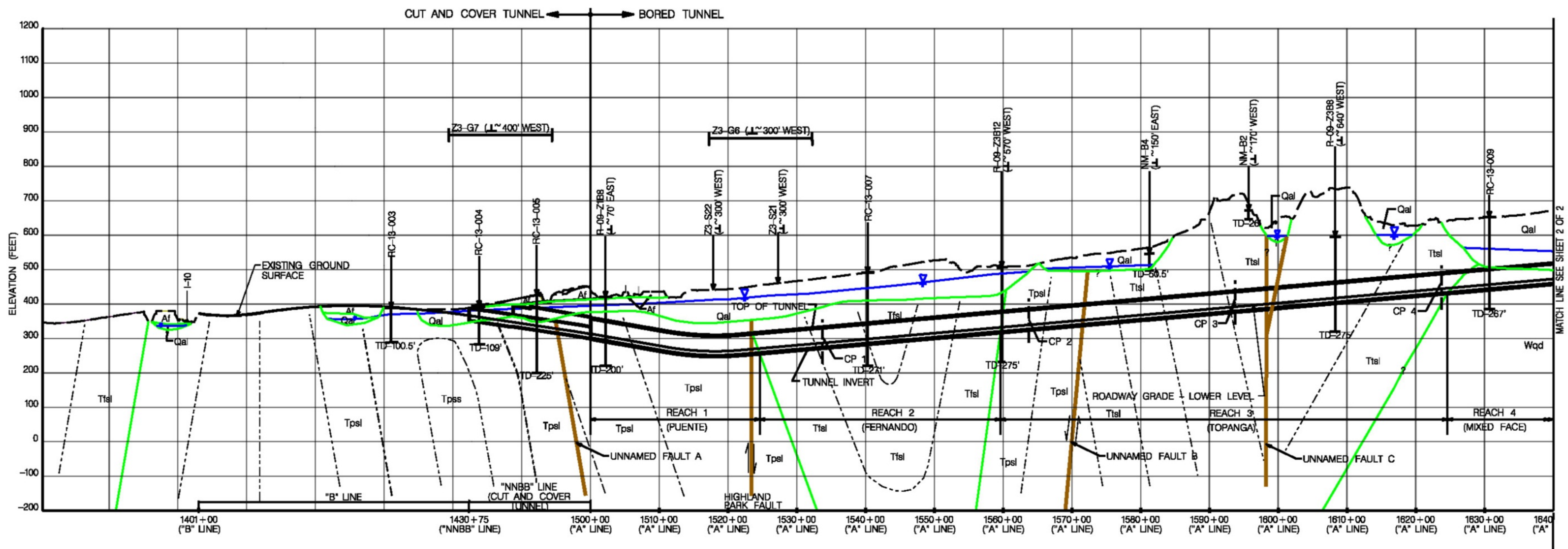
2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 1/11/2013.

5) PREDOMINATE GEOLOGIC FORMATION WITHIN EACH REACH IS SHOWN IN PARENTHESIS UNDER THE REACH NUMBER. "MIXED FACE" IS DEFINED AS SOIL OVER BEDROCK WITHIN THE FACE OF TUNNEL EXCAVATION.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

- Af ARTIFICIAL FILL
- Qal ALLUVIAL SOIL
- Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER
- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
- Tpsl PUENTE FORMATION, SILTSTONE MEMBER
- Tpss PUENTE FORMATION, SANDSTONE MEMBER
- Tt TOPANGA FORMATION, UNDIFFERENTIATED
- Ttss TOPANGA FORMATION, SANDSTONE MEMBER
- Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER
- Ttsl TOPANGA FORMATION, SILTSTONE MEMBER
- Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
- INTRAFORMATIONAL CONTACT
- GENERALIZED BEDDING
- ESTIMATED TOP OF GROUNDWATER TABLE
- Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION
- A-13-020 TD-149.8' GEOTECHNICAL BOREHOLE WITH TOTAL DEPTH AND PROJECTION:
  A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
  R-09-Z1B8 – CH2M HILL, 2010
  NM-B3 – NINYO AND MOORE, 1999
  EMI-3 – EARTH MECHANICS INC, 2008
  ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FIGURE 1
FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

Figure 1. Freeway Geologic Profile (Sheet 1 of 2)

NOTES:

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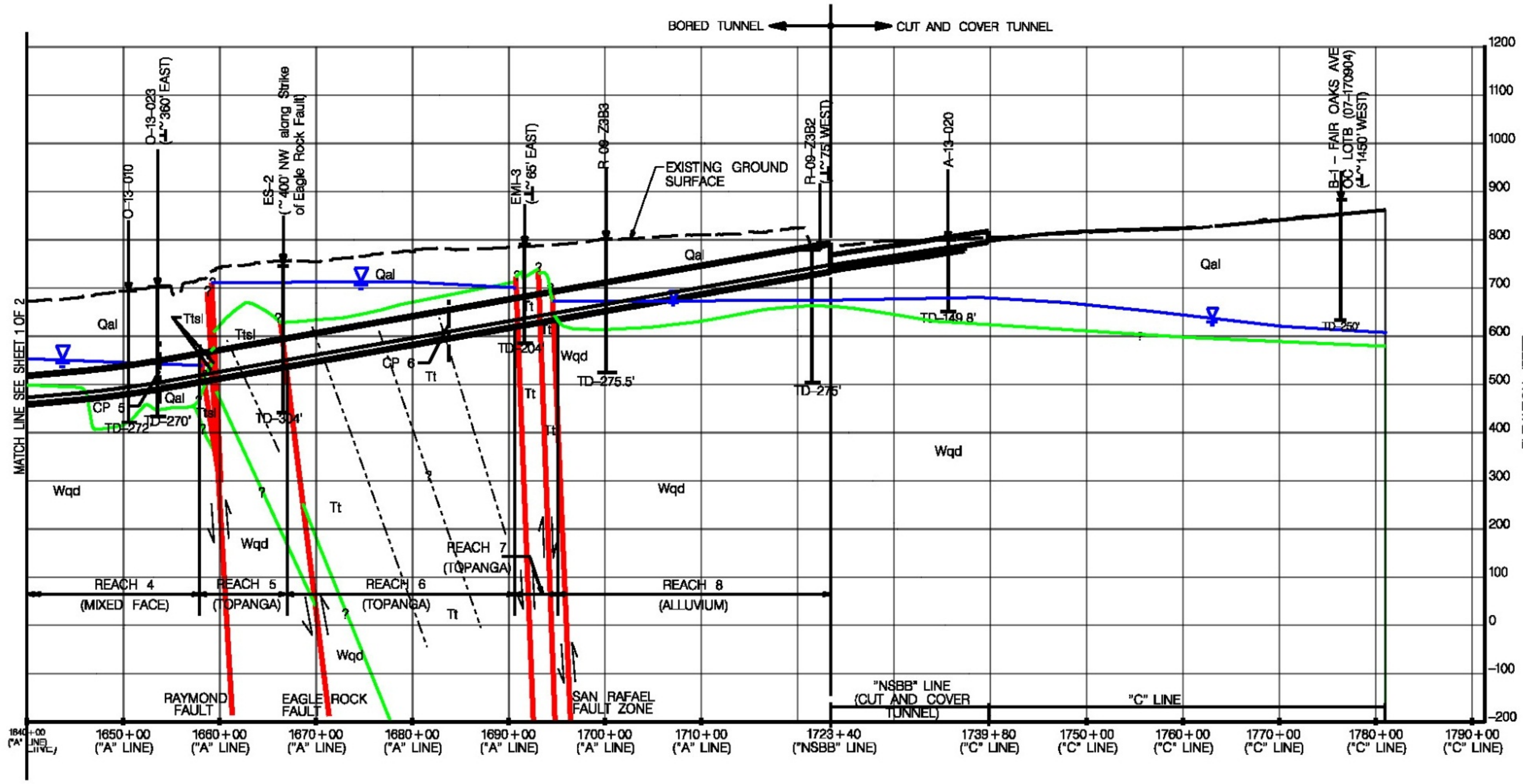
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

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- Qal ALLUVIAL SOIL
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- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
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  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FIGURE 1
FREEWAY GEOLOGIC PROFILE
SHEET 2 OF 2

Figure 1. Freeway Geologic Profile (Sheet 2 of 2)

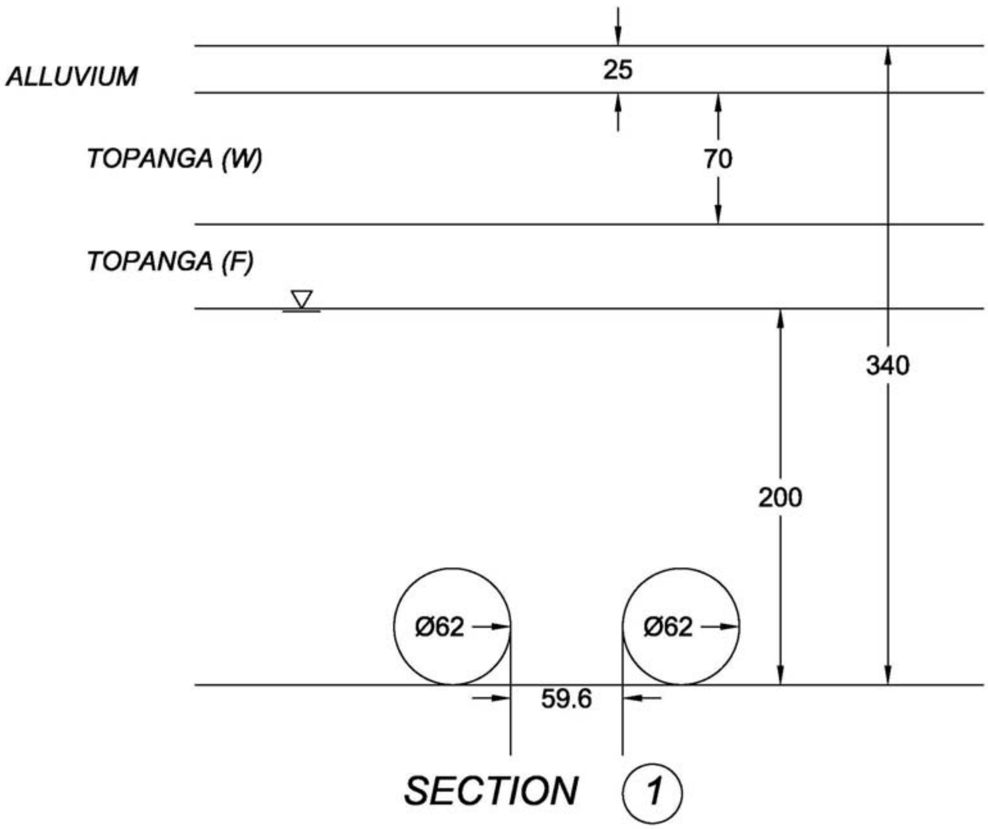

Figure 2. Lining Analysis Section in Rock, approx. Sta. 1610+00

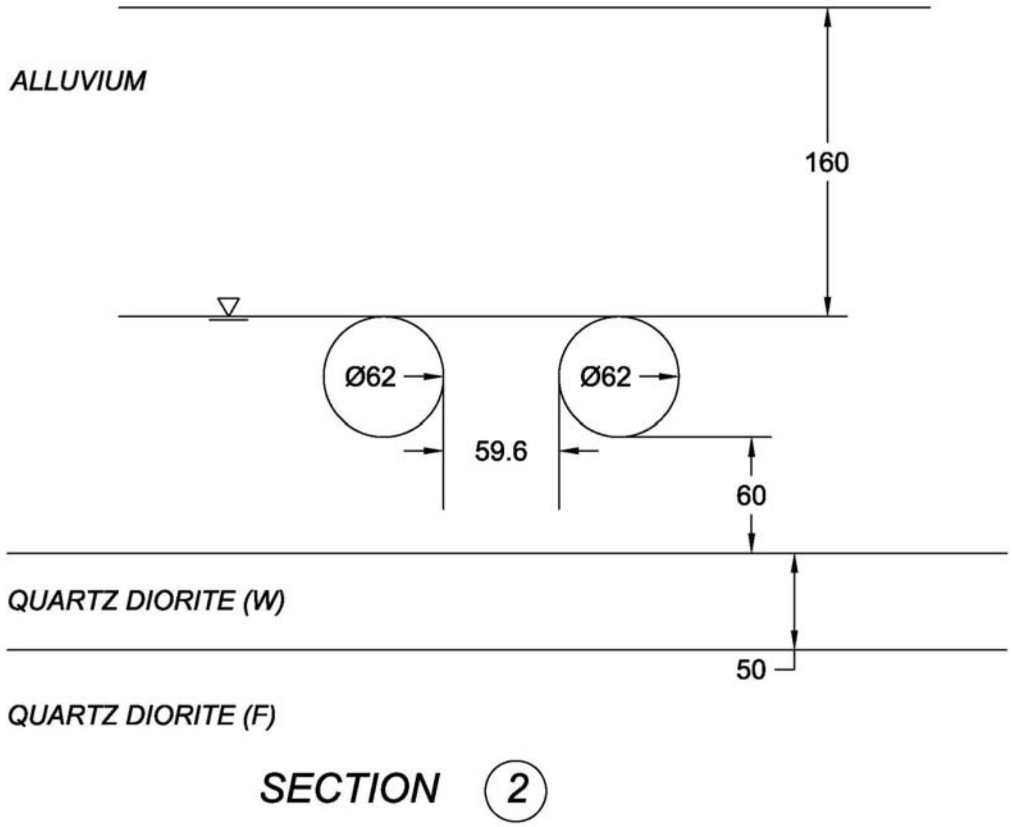

Figure 3. Lining Analysis Section in Soil, approx. Sta. 1650+00

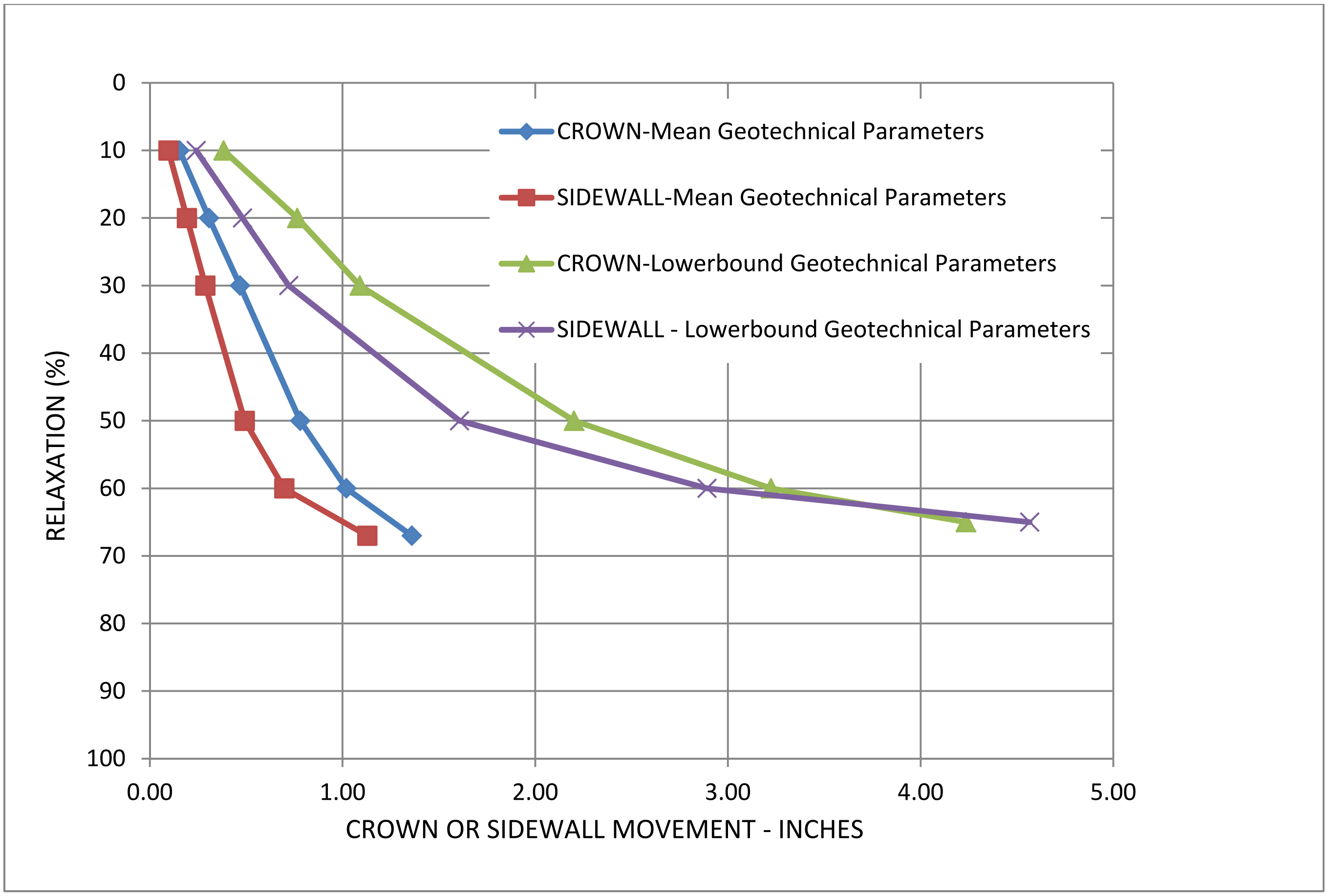

Figure 4. Ground Reaction Curves, Rock Section

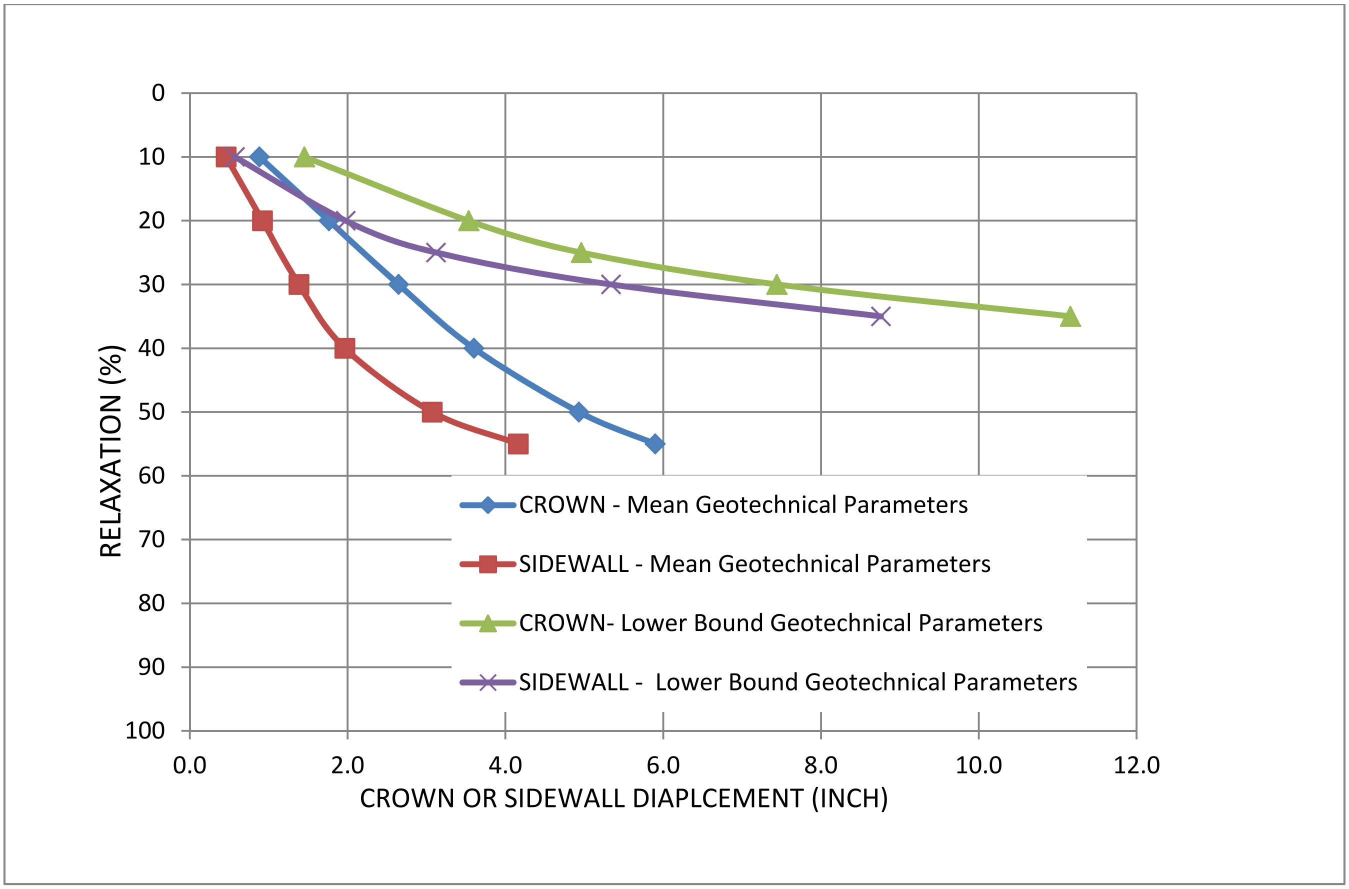

Figure 5. Ground Reaction Curve, Soil Section

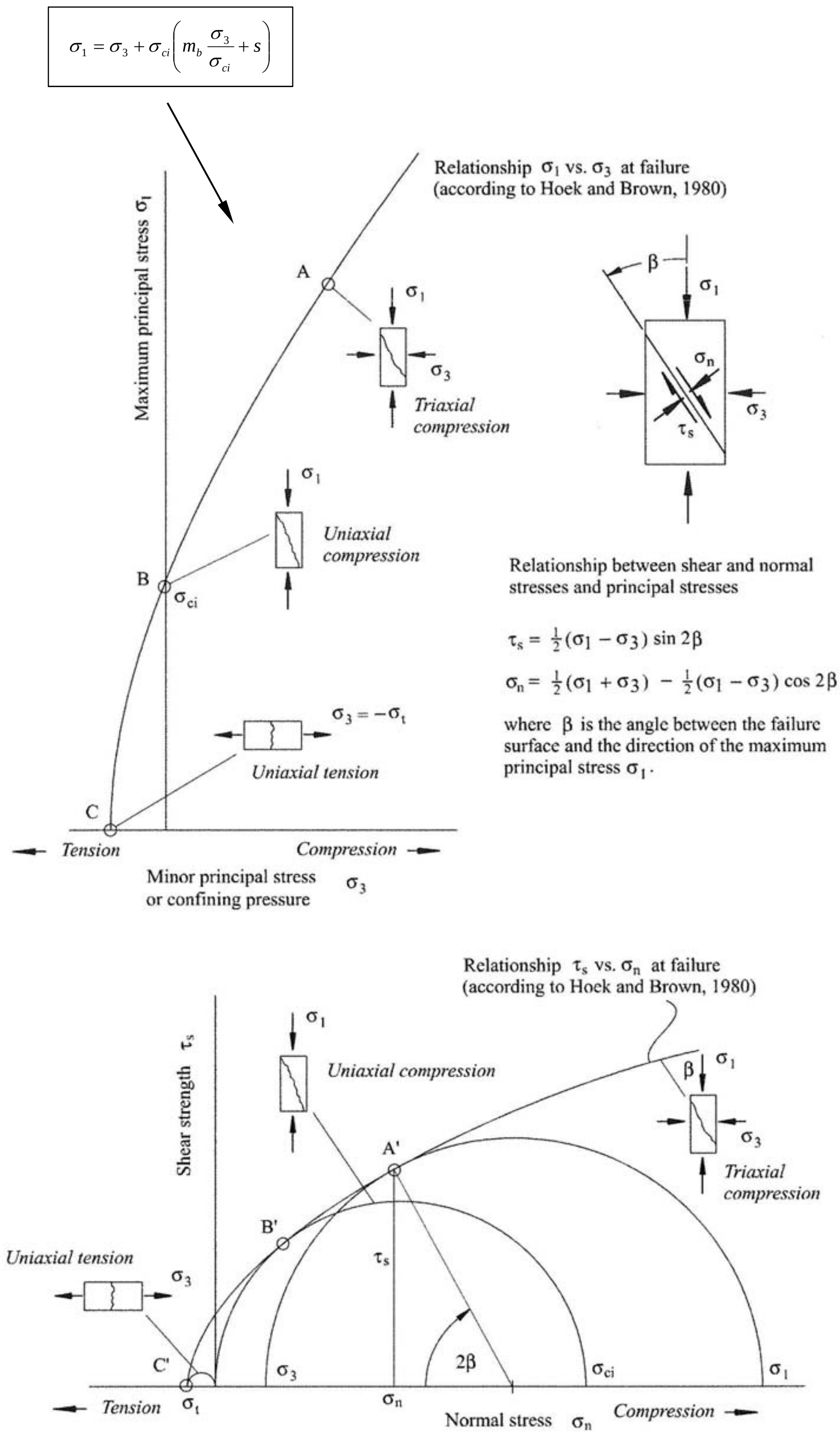

Figure 6: The Hoek-Brown Material Model for Rock

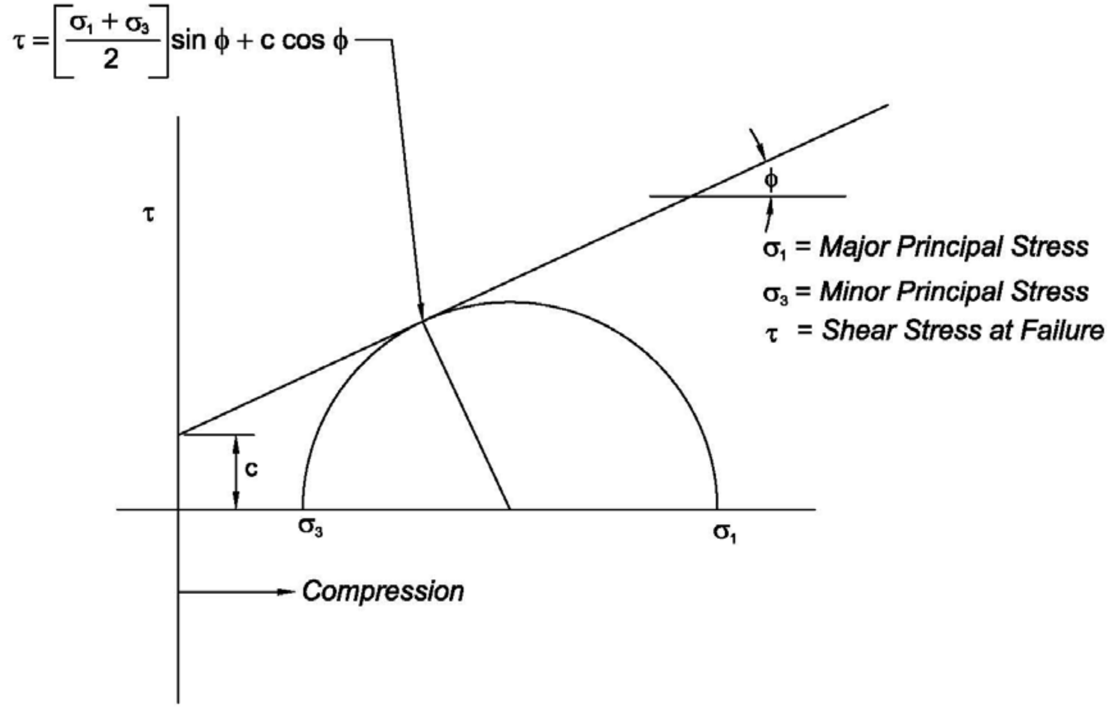

Figure 7: The Mohr-Coulomb Material Model for Soil

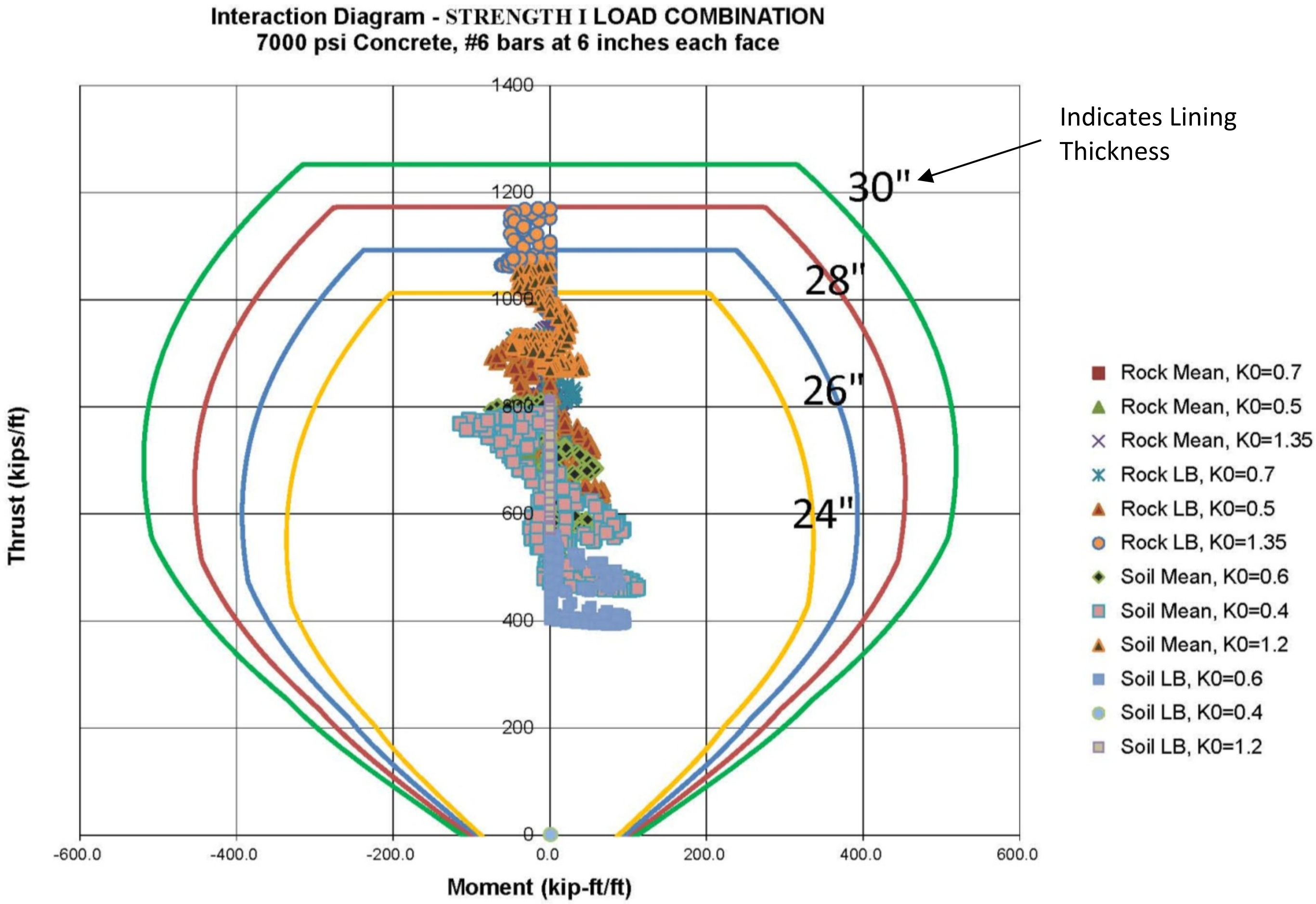

Figure 8. Moment-Thrust Diagram for the Segmental Lining, Strength I Load Combination

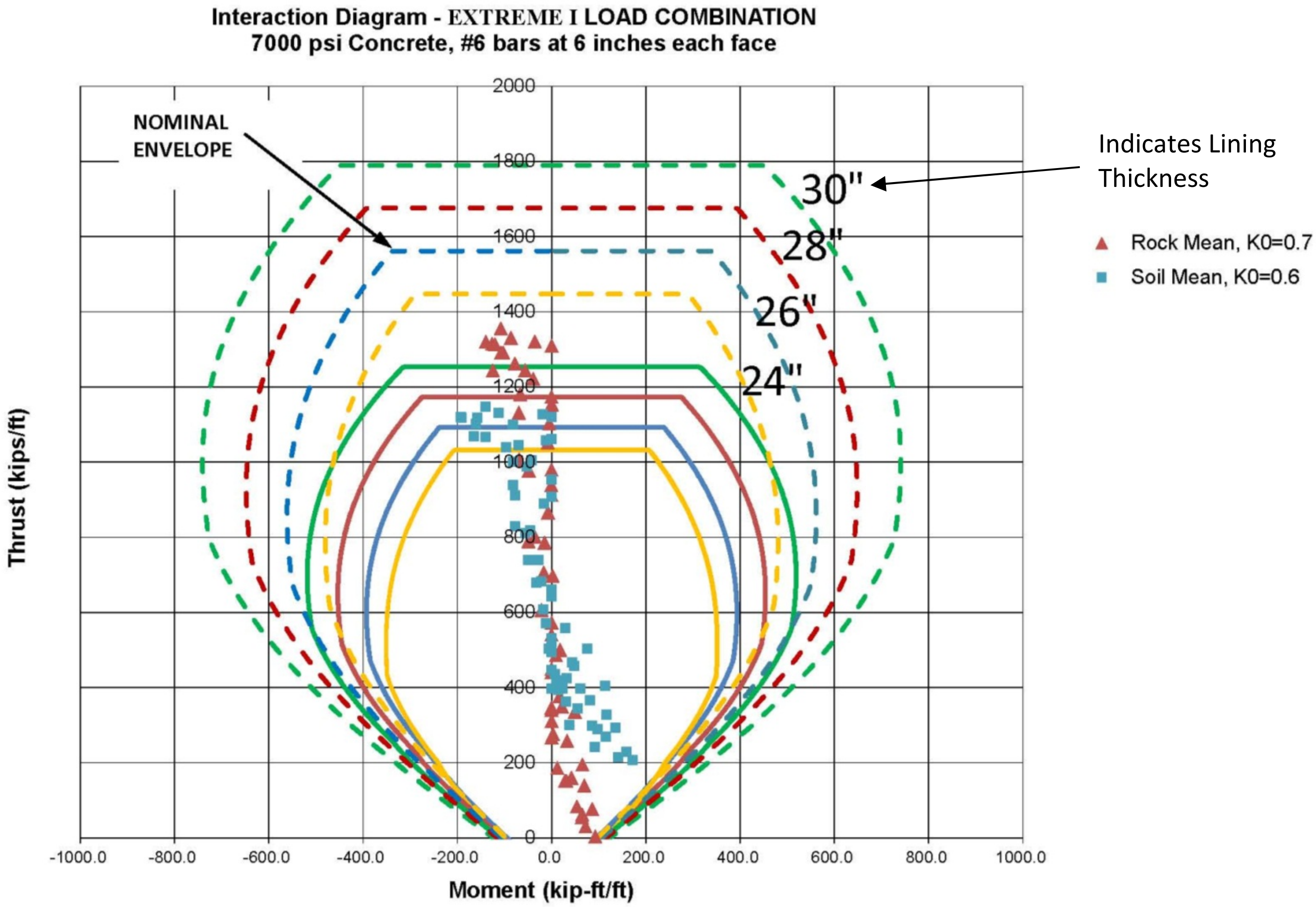

Figure 9. Moment-Thrust Diagram for the Segmental Lining, Extreme I Load Combination

Figure 10. Segment Testing Showing Tensile Splitting Cracks Originating at the Segment to Segment Contact (Swartz et al., 2002)

Figure 11. Segment Testing showing Tensile Splitting due to Gasket Compression (Swartz et al., 2002)

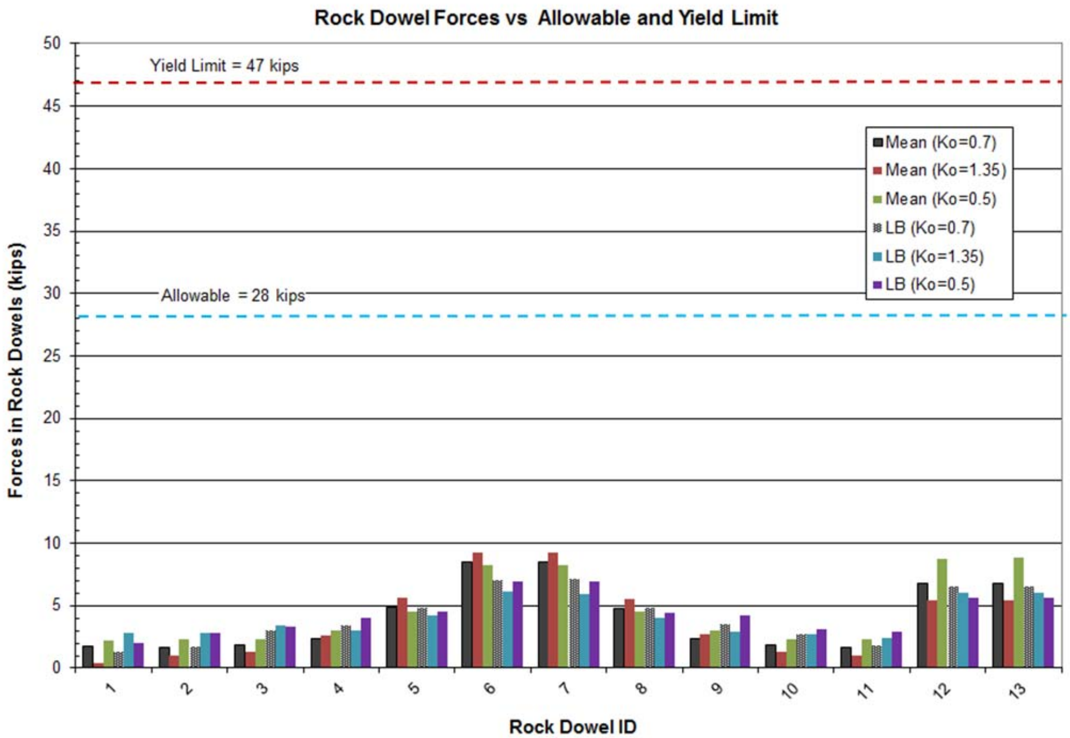

Figure 12. Forces in Rock Dowels in Rock Section

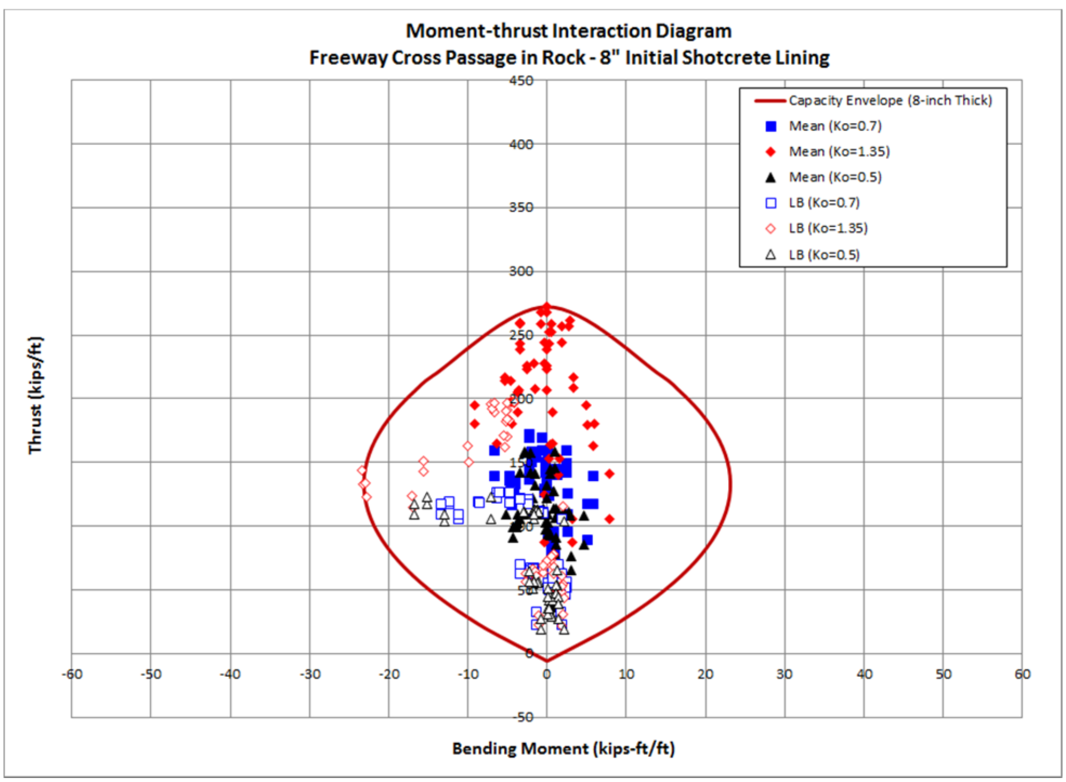

Figure 13. Moment-thrust Interaction Diagram for Shotcrete Lining in ST1 Section

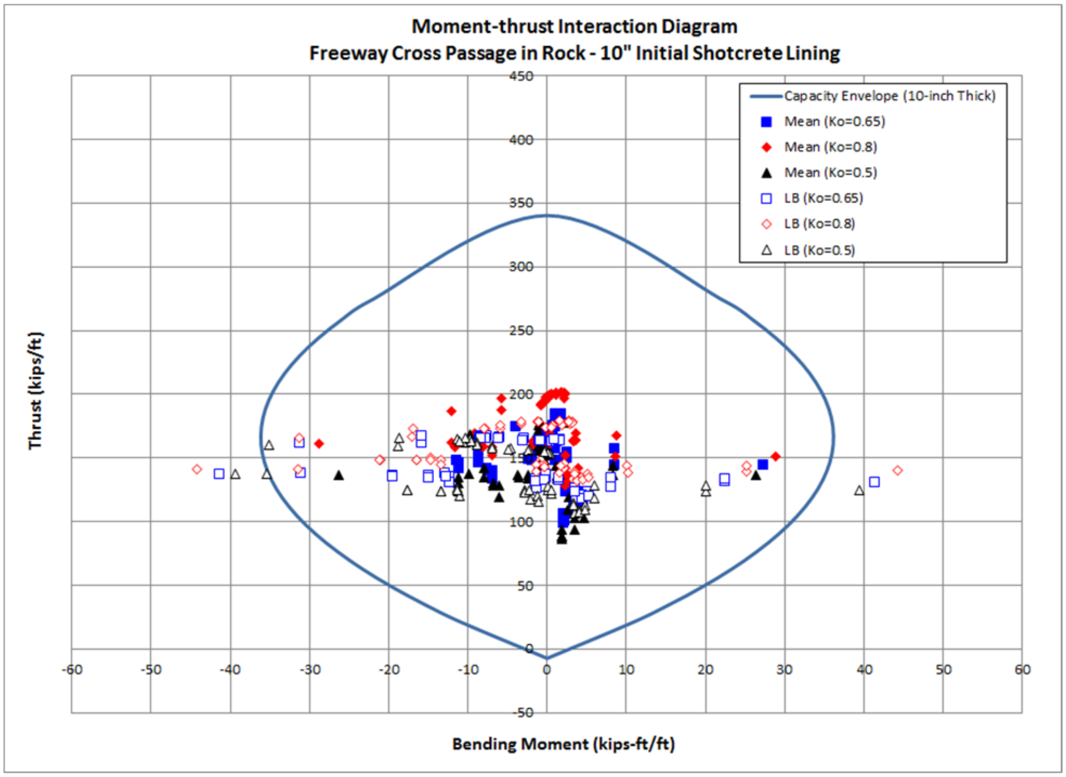

Figure 14. Moment-thrust Interaction Diagram for Shotcrete Lining in ST2 Section

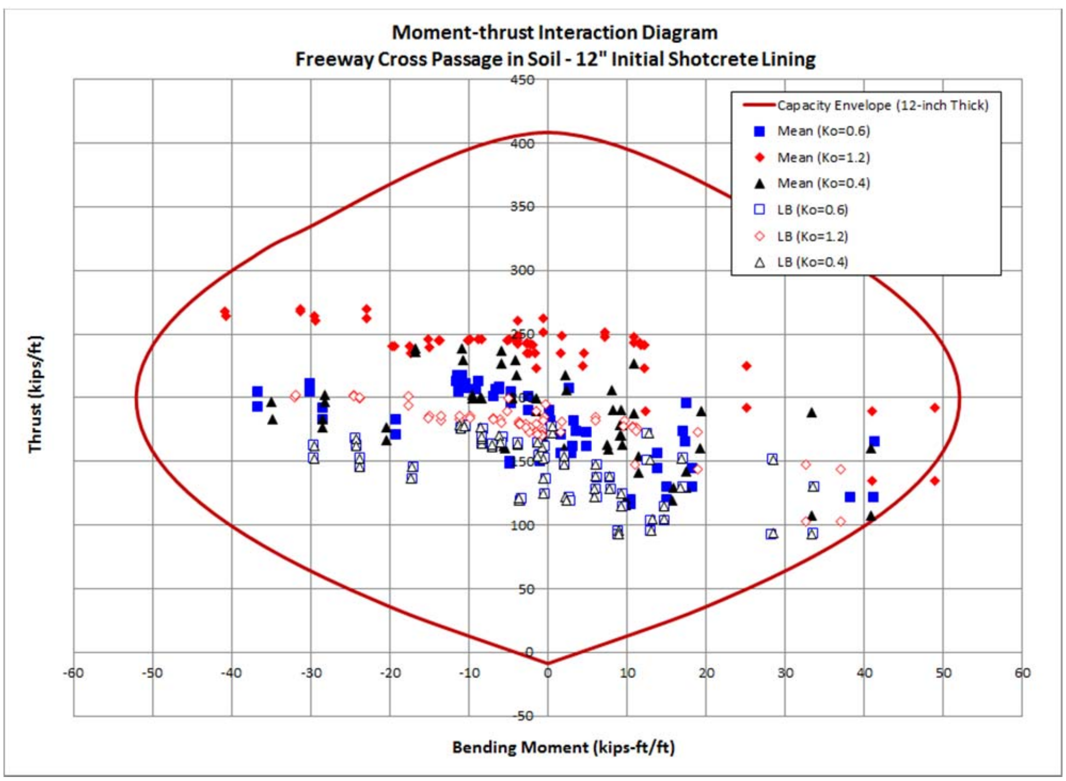

Figure 15. Moment-thrust Interaction Diagram for Shotcrete Lining in ST3 Section

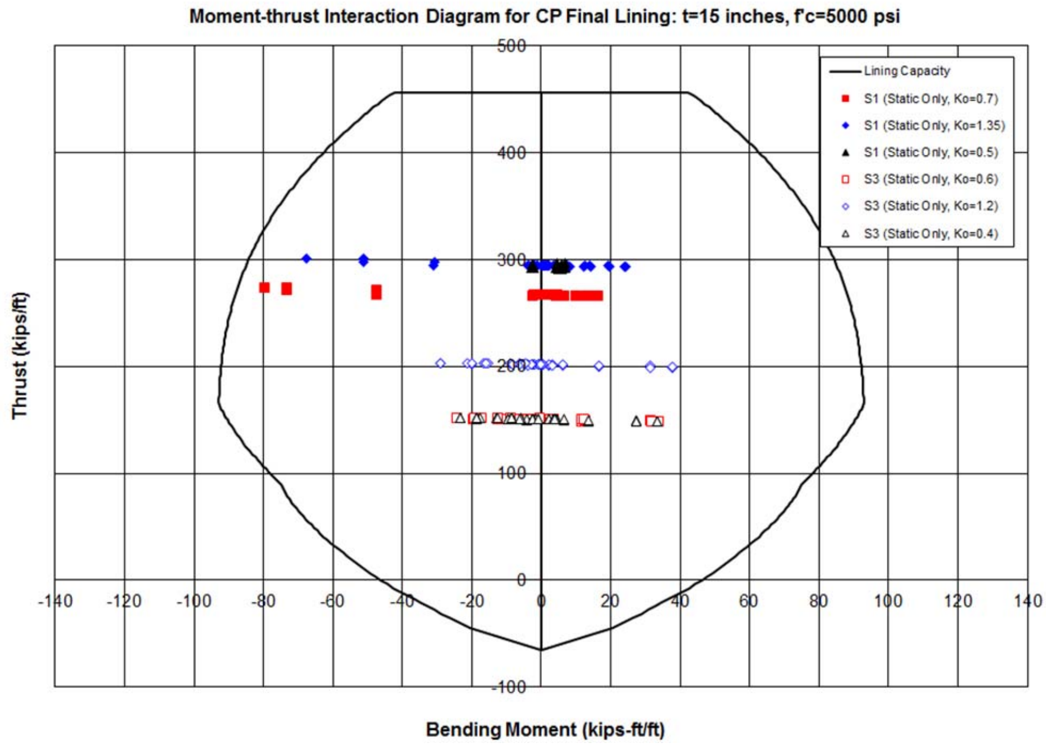

Figure 16. Strength I Moment-thrust Interaction Diagram for Final Lining Arch

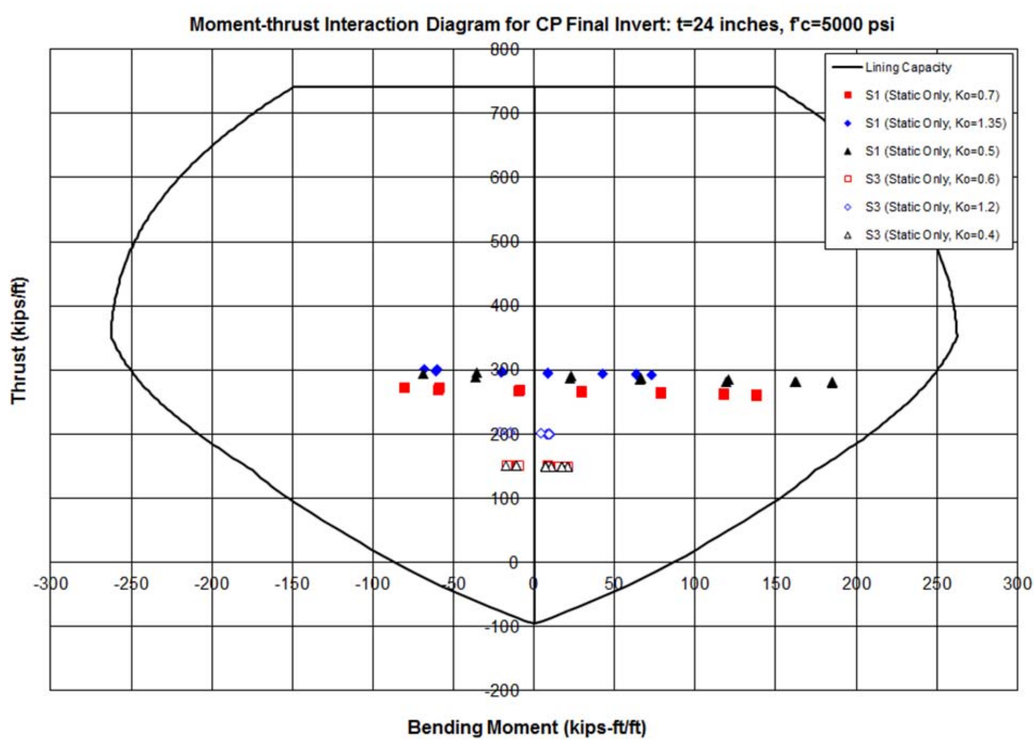

Figure 17: Strength I Moment-thrust Interaction Diagram for Final Invert

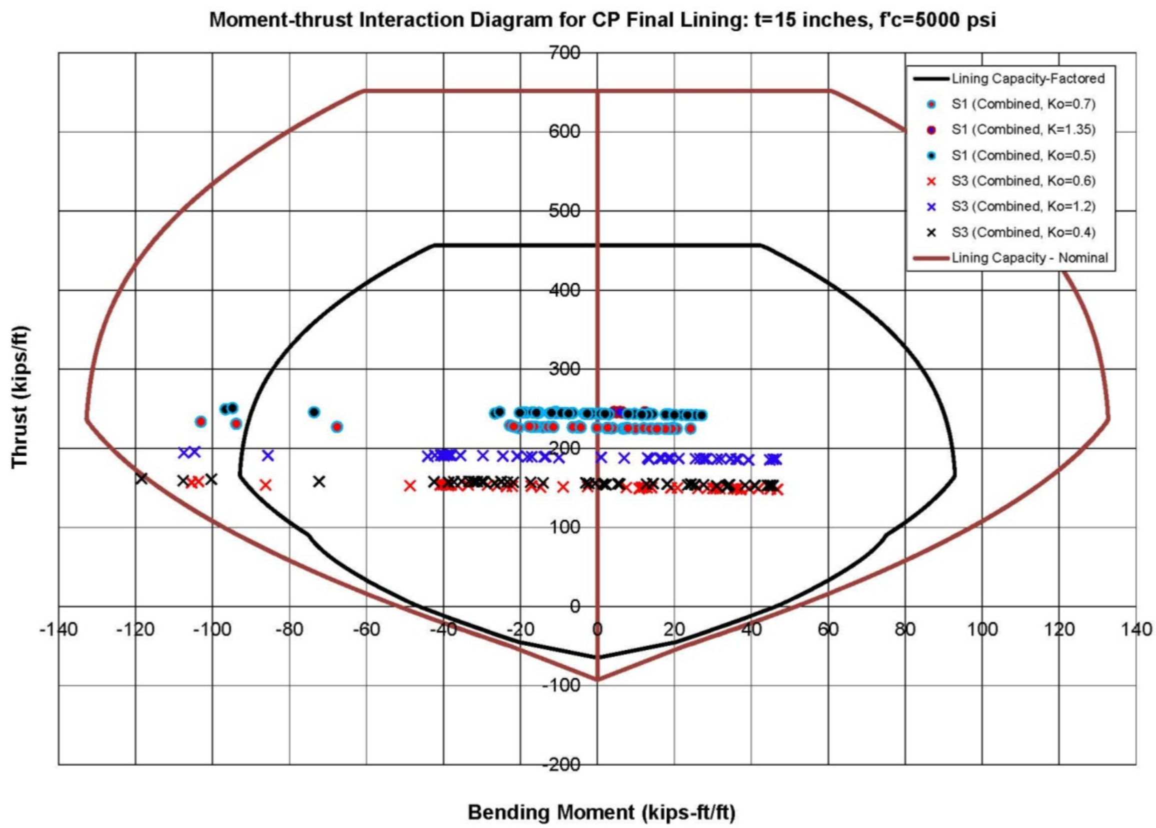

Figure 18. Extreme I Moment-thrust Interaction Diagram for Final Lining Arch

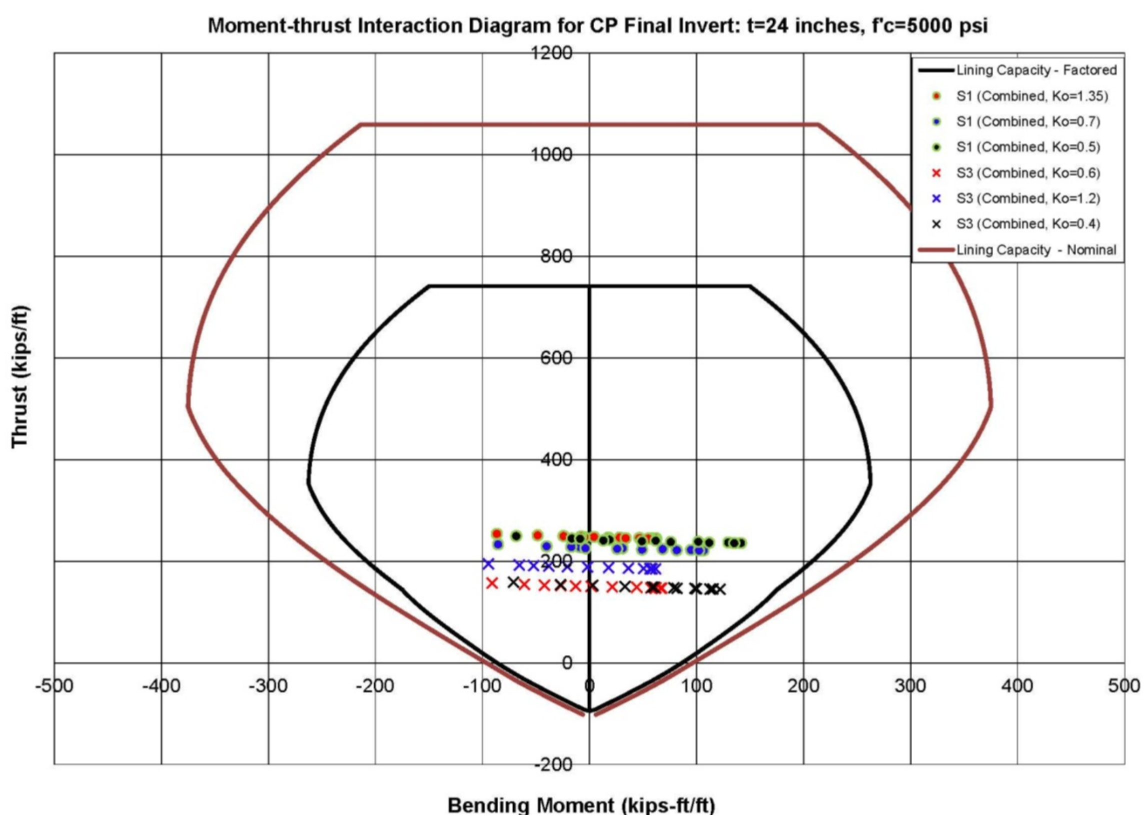

Figure 19. Extreme I Moment-thrust Interaction Diagram for Final Invert

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Tables

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Table 1. Soil Parameters

Soil Type	Range	Total Unit Weight (pcf)	Deformation Modulus (ksi)	Poisson's Ratio	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Horizontal-to-vertical Stress Ratio (K_0)
Fill	Mean	120	2.0	0.30	0	32	0.5
Old Alluvium	LB	125	6.9	0.35	0	32	0.6 (0.4–1.2)
	Mean		13.9		500	36	

Table 2. Rock Mass Parameters

Geologic Formation	Range	Total Unit Weight (pcf)	Average Applicable Depth (ft)	GSI Classification Parameters *				Hoek-Brown Model Parameters *			Equivalent Mohr-Coulomb Model Parameters		Deformation Modulus (ksi)	Poisson's Ratio	Horizontal-to-vertical Stress Ratio (K_0)
				Intact Rock UCS (psi)	GSI	mi	D	m_b	s	a	Cohesion (psf)	Friction Angle (degrees)			
Fernando Formation, Tf	LB	136	150	50	N/A						1,050	20	10	0.35	0.65 (0.5–0.8)
	Mean			300							1,950	29	25		
Puente Formation, Tp-2	LB	134	50	30	35	6	0	0.589	0.0007	0.516	250	17	10	0.30	0.7 (0.5–1.35)
	Mean			50	45	6	0	0.842	0.0022	0.508	400	22	15		
Puente Formation, Tp-1	LB	134	150	150	45	10	0	1.403	0.0022	0.508	1,300	26	35	0.30	0.7 (0.5–1.35)
	Mean			400	55	10	0	2.005	0.0067	0.504	2,300	36	70		
Topanga Formation, Tt-2	LB	134	50	30	40	6	0	0.704	0.0013	0.511	300	17	10	0.30	0.7 (0.5–1.35)
	Mean			60	50	6	0	1.006	0.0039	0.506	450	24	15		
Topanga Formation, Tt-1	LB	134	150	230	50	12	0	2.012	0.0039	0.506	1,800	32	40	0.30	0.7 (0.5–1.35)
	Mean			500	60	12	0	2.876	0.0117	0.503	2,950	41	100		

Geologic Formation	Range	Total Unit Weight (pcf)	Average Applicable Depth (ft)	GSI Classification Parameters *				Hoek-Brown Model Parameters *			Equivalent Mohr-Coulomb Model Parameters		Deformation Modulus (ksi)	Poisson's Ratio	Horizontal-to-vertical Stress Ratio (K_0)
				Intact Rock UCS (psi)	GSI	mi	D	m_b	s	a	Cohesion (psf)	Friction Angle (degrees)			
Basement Complex Rock, Wqd-2	LB	158	50	35	30	25	0	2.052	0.0004	0.522	450	25	15	0.25	0.5 (0.4–0.6)
	Mean			80	45	25	0	3.506	0.0022	0.508	800	35	20		
Basement Complex Rocks, Wqd-1	LB	158	150	250	50	25	0	4.192	0.0039	0.506	2,600	37	50	0.25	0.5 (0.4–0.6)
	Mean			680	60	25	0	5.991	0.0117	0.503	4,350	48	130		

Table 3. Empirical Rock/Soil Loads

Rock Condition	Terzaghi, 1946 as presented in Proctor and White, 1968	Deere, et al., 1969			Rose, 1982	
	Rock Load	RQD	Rock Load - Initial	Rock Load-Final	RQD	Rock Load
Hard and Intact	0	90-100	0	0	95-100	0
Hard Stratified or Schistose	0 to 0.5B	95-100 90-95	0 0	0.25B 0.5B	90-99	0 to 0.5B
Massive, Moderately Jointed	0 to 0.25B	90-95	0	0.5B	85-95	0 to 0.25B
Moderately Blocky and seamy	0.25B to 0.35 (B+Ht)	75	0	0.25B to 0.35C	75-85	0.25B to 0.2 (B+Ht)
Very Blocky and Seamy	0.35 to 1.1 (B+Ht)	50	0 to 0.6C	0.25C to 1.1C	30-75	0.2 to 0.6 (B+Ht)
Completely Crushed but Chemically Intact	1.1 (B+Ht)	10-25		1.1C	3-30	0.6 to 1.1 (B+Ht)
Squeezing Rock, Moderate Depth	1.1 to 2.1 (B+Ht)			1.1C to 2.1C		1.1 to 2.1 (B+Ht)
Squeezing Rock, Great Depth	2.1 to 4.5 (B+Ht)			2.1C to 4.5C		2.1 to 4.5 (B+Ht)
Swelling Rock	up to 250 feet, irrespective of (B+Ht)			up to 80m		up to 250 feet, irrespective of (B+Ht)

Soil Condition	Terzaghi, 1946 as presented in Proctor and White, 1968		Deere, et al., 1969			Rose, 1982	
	Initial Load	Final Load		Initial Load	Final Load		Load
Sand and Gravel - Dense	0.54C to 1.2C	0.62C to 1.38C		0.54C to 1.2C	0.62C to 1.38C		1.1 to 1.4 (B+Ht)
Sand and Gravel - Loose	0.96C to 1.2C	1.08C to 1.38C		0.96C to 1.2C	1.08C to 1.38C		

Notes:

1. Refer to Proctor and White, 1968; Deere, et al., 1969 and Rose, 1982 for additional details regarding Rock Loads and the Original Tables.
2. Load should be limited by total ground cover
3. B = Tunnel Width, Ht = Tunnel Height
4. C = B+Ht

Table 4. Seismic Design Parameters [1, 2]

Parameter		Freeway Alternative FEE (100 yr)	Freeway Alternative SEE (1,000 yr)
PGA (horizontal) (g)	Rock	0.21	0.75
	Soil	0.23	0.84
PGV (horizontal) (ft/s)	Rock	0.83	2.92
	Soil	0.92	3.33

[1] These are parameters associated with horizontal ground motions. The parameters associated with vertical ground motions can be estimated using the vertical-to-horizontal (V/H) ratio of 0.9.

[2] Sources: CH2MHill 2014.

Table 5. Soil and Rock Mass Dynamic Parameters for Seismic Design

Soil / Rock Formation	Total Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Primary Wave Velocity (ft/sec)	Effective Shear Wave Velocity [1] (ft/sec)	Effective Dynamic Shear Modulus (ksi)	Effective Dynamic Young's Modulus (ksi)
Fill	120	500	816	350	3.2	7.6
Old Alluvium (<50 ft Deep)	135	1,080	2,248	756	16.6	44.9
Old Alluvium (>50 ft Deep)		1,650	3,435	1,155	38.8	104.9
Fernando Formation	136	1,080	2,248	864	21.9	59.1
Puente Formation (Weathered) (Tp-2)	134	1,600	2,993	1,280	47.3	123.1
Puente Formation (Tp-1)		2,200	4,116	1,760	89.5	232.7
Topanga Formation (Weathered) (Tt-2)	134	1,300	2,432	1,040	31.3	81.3
Topanga Formation (Tt-1)		2,900	5,425	2,320	155.5	404.4
Basement Complex Rocks (Weathered) (Wqg-2)	158	1,600	2,771	1,280	55.8	139.6
Basement Complex Rocks (Wqg-1)		3,500	6,062	2,800	267.1	667.9

[1] Assumed to be equal to 0.7Cs for soil and 0.8Cs for rock.

Table 6. Locations and Characteristics of Cross Passages for Freeway Alternative

CP No.	Stationing	Ground Cover (ft)	Soil Overburden Depth (ft)	Ground Surface Elevation (XP Crown Elevation) (ft)	Ground Class/Ground Condition	Groundwater Table Elevation (Head at Crown Elevation) (ft)	Support Type	Selected Section for Analysis
1	1533+75	140–160	85	480 (310–330)	Ground Class 2: In Rock; Fernando Formation (Tf)	440 (110–130)	ST2	ST2 Section
2	1563+75	130–150	20	510 (360–380)	Ground Class 2: In Rock; Puente Formation (Tp-2)	490 (110–130)	ST2	N/A
3	1593+75	280–300	0	720 (420–440)	Ground Class 1: In Rock; Topanga Formation (Tt-1)	720[1] (280–300)	ST1	ST1 Section
4	1623+75	150–170	0	640 (470–490)	Ground Class 1: In Rock; Topanga Formation (Tt-1)	640[1] (150–170)	ST1	N/A
5	1653+75	150–170	250	700 (530–550)	Ground Class 3: In Soil; Alluvium (S)	540 (0–10)	ST3	ST3 Section
6	1683+75	125–145	100	780 (630–650)	Ground Class 2: In Rock; Topanga Formation (Tt-2)	710 (60–80)	ST2	N/A

[1] Approximate groundwater elevation was interpolated.

Refer to *Tunnel Ground Characterization* (JA, 2014b) for details on soil/rock formations.

Attachment A

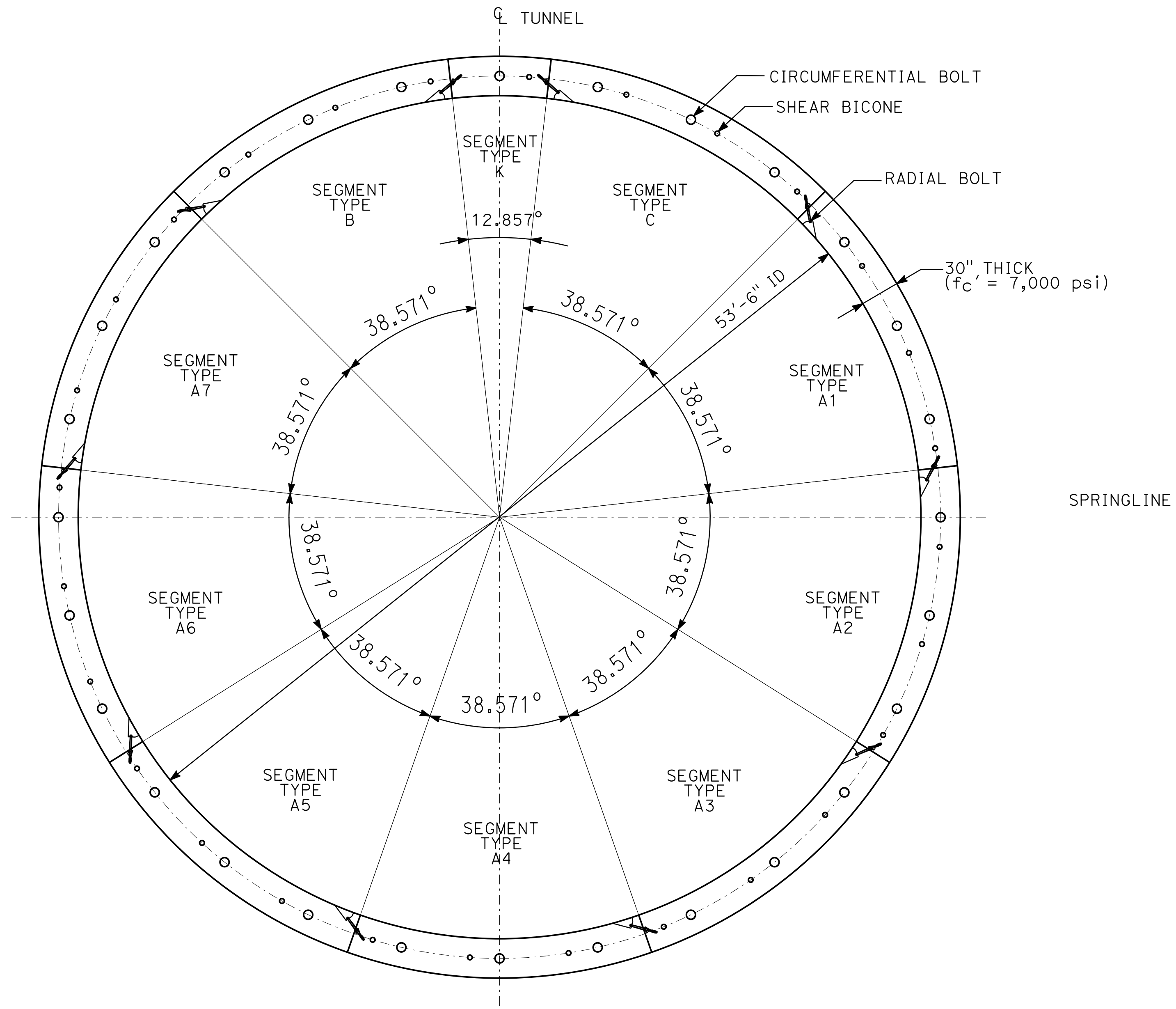
Drawings

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



NOTES

1. SEE S2.03 FOR DEVELOPED PLAN.
2. SEE S2.04 FOR TYPICAL DETAILS.

PRECAST CONCRETE SEGMENTAL LINING

S2.02

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN/J. VANGREUNEN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY
SEGMENTAL LINING SECTION

BRIDGE NO. TBD
UNIT:
SCALE: 1"=5'
PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S202.dgn

CONTRACT NO.: X

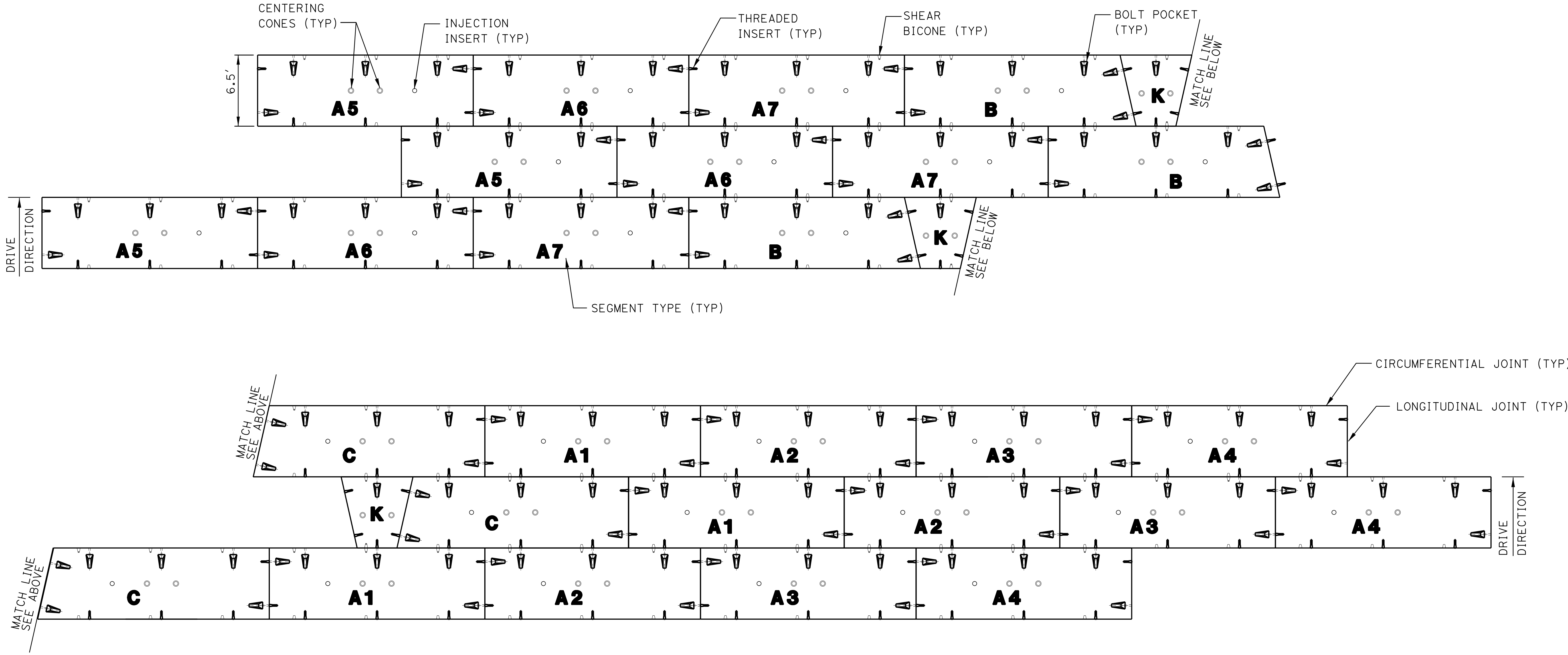
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 13:22

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



DEVELOPED PLAN OF SEGMENTS

S2.03

DESIGNED BY	M. HARIHARAN	DATE 9-20-13
DRAWN BY	J. TOLES	DATE 9-20-13
CHECKED BY	S. KLEIN/J. VANGREUNEN	DATE 9-20-13
APPROVED	S. DUBNEWYCH	DATE 9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

SEGMENTAL LINING PLAN

BRIDGE NO. TBD
UNIT:
SCALE: AS NOTED
PROJECT NUMBER & PHASE:
CONTRACT NO.: X

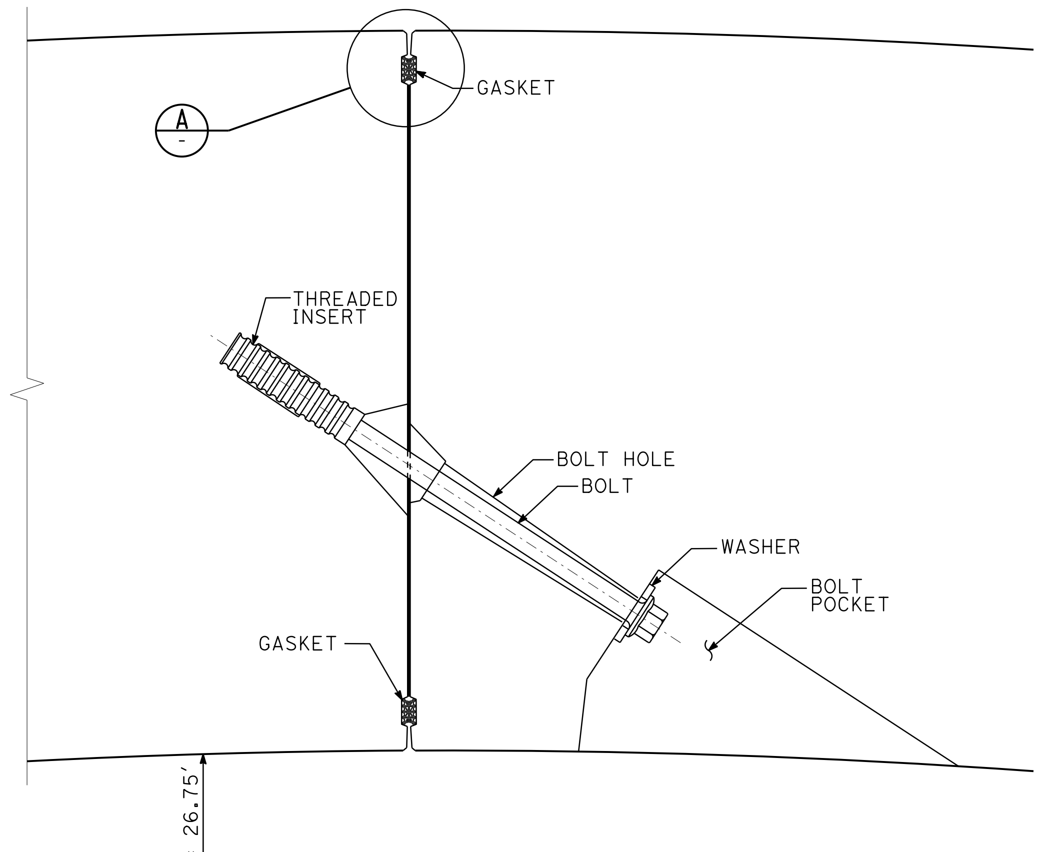
DESIGN OVERSIGHT
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

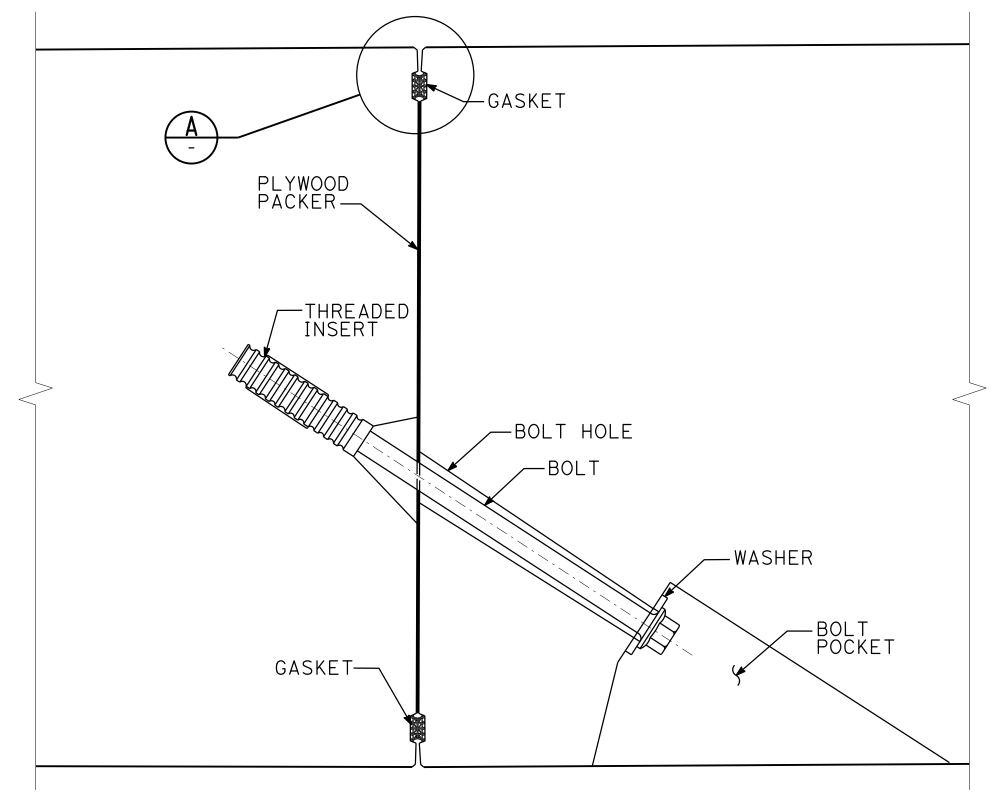
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

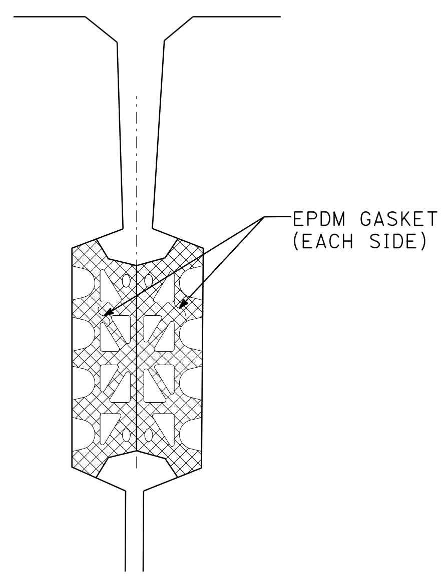
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



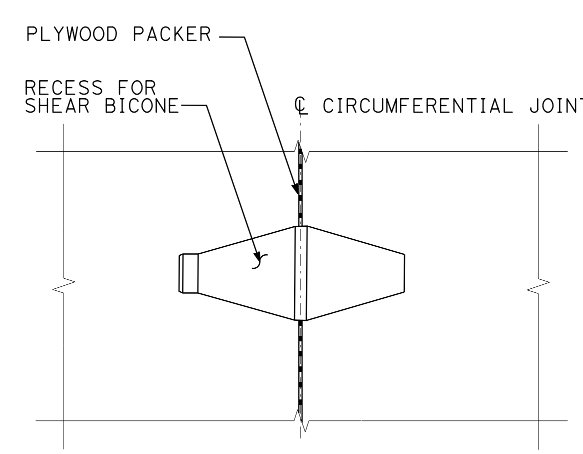
RADIAL JOINT CONNECTION



CIRCUMFERENTIAL JOINT CONNECTION



GASKET DETAIL (A / –)



SHEAR BICONE RECESS DETAIL (B / –)

S2.04

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN/J. VANGREUNEN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

SEGMENTAL LINING DETAILS

BRIDGE NO. TBD	UNIT:
SCALE: 1"=1'	PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S204.dgn

CONTRACT NO.: X

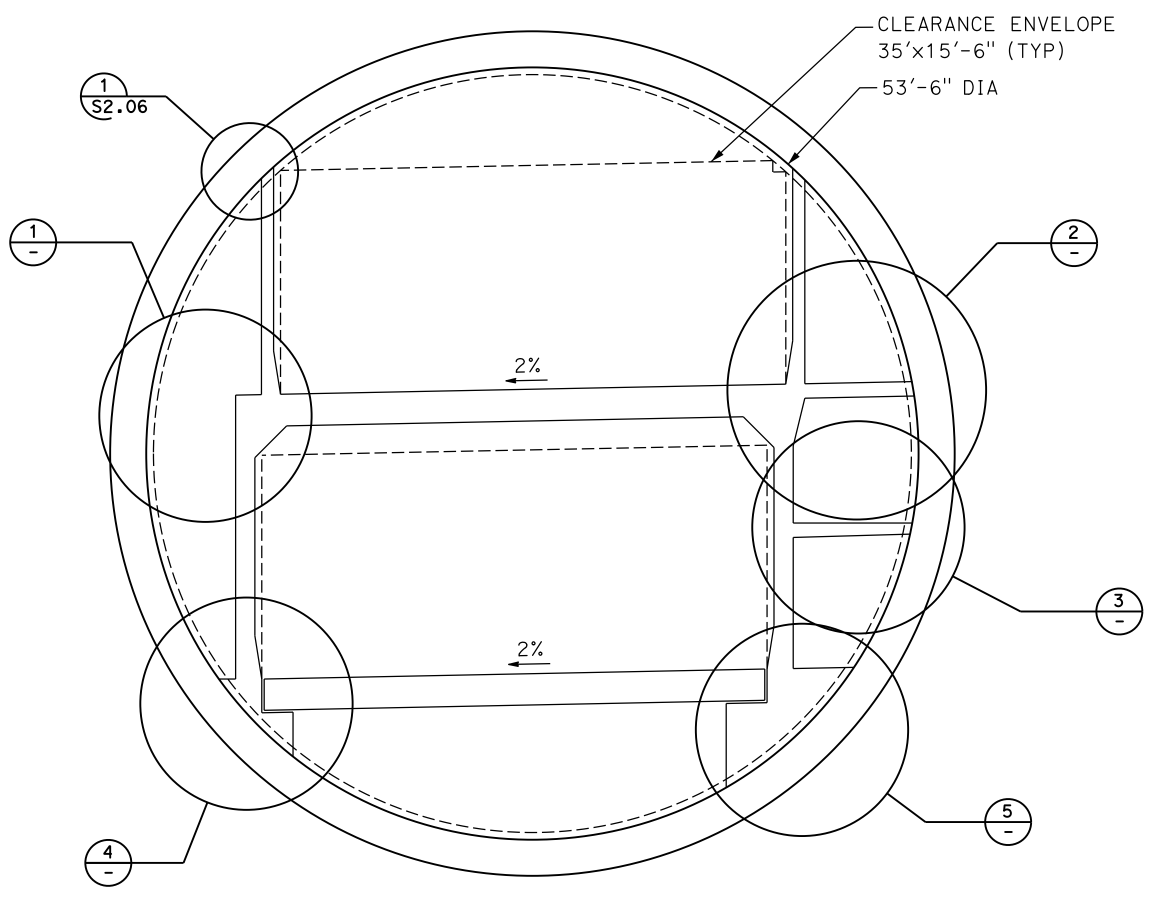
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 13:19

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

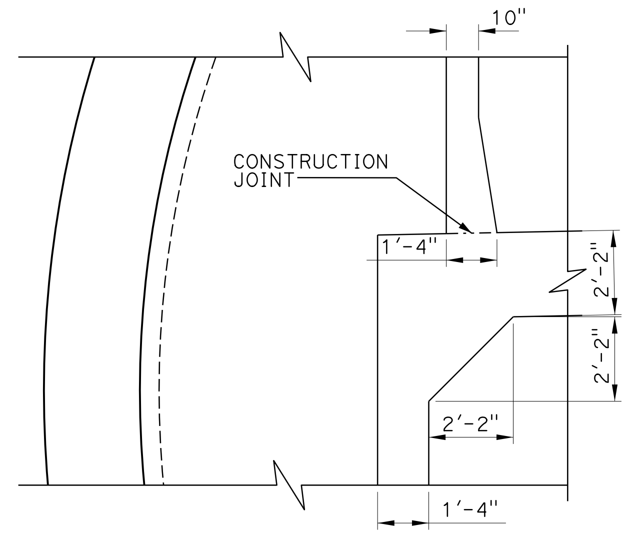
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

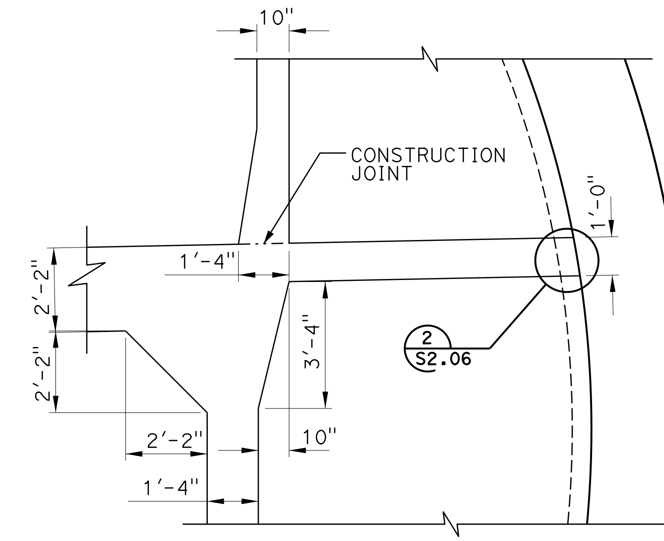
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



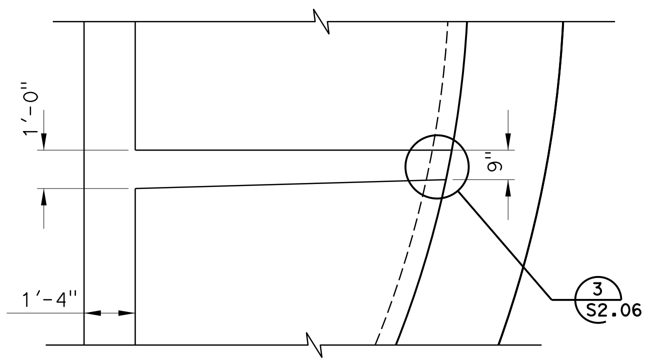
TUNNEL SECTION
NTS



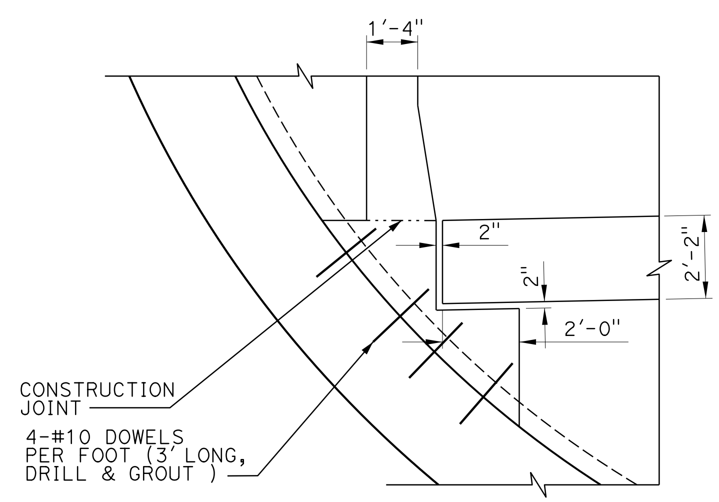
DETAIL 1 -
NTS



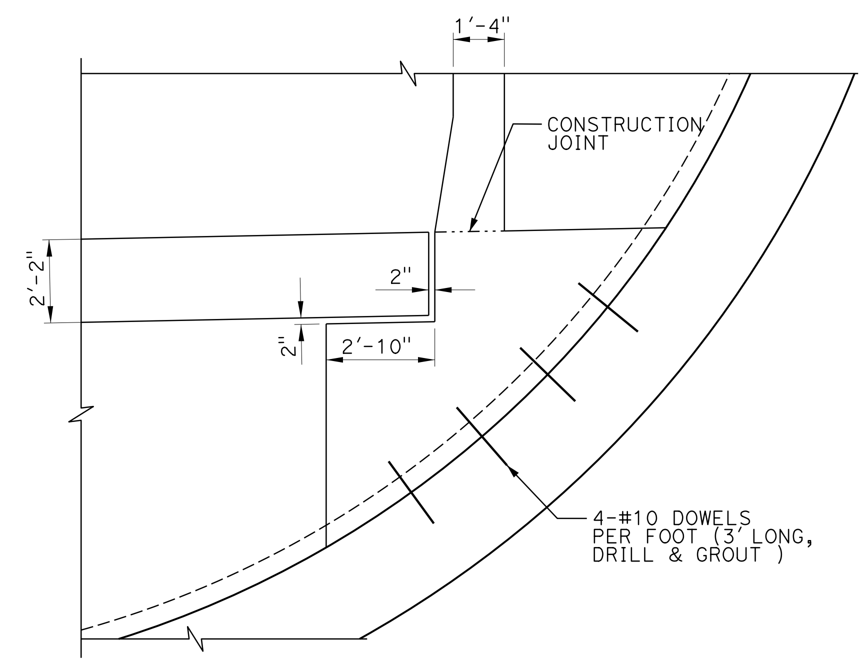
DETAIL 2 -
NTS



DETAIL 3 -
NTS



DETAIL 4 -
NTS



DETAIL 5 -
NTS

NOTE:

1. REFER TO S2.01 FOR DETAILS ON TYPICAL TUNNEL SECTION.

S2.05

DESIGNED BY	J. VANGREUNEN	DATE 9-20-13
DRAWN BY	J. TOLES	DATE 9-20-13
CHECKED BY	S. KLEIN	DATE 9-20-13
APPROVED	S. DUBNEWYCH	DATE 9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

INTERNAL STRUCTURE DETAILS

BRIDGE NO. TBD	UNIT:
SCALE: AS SHOWN	PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S205.dgn

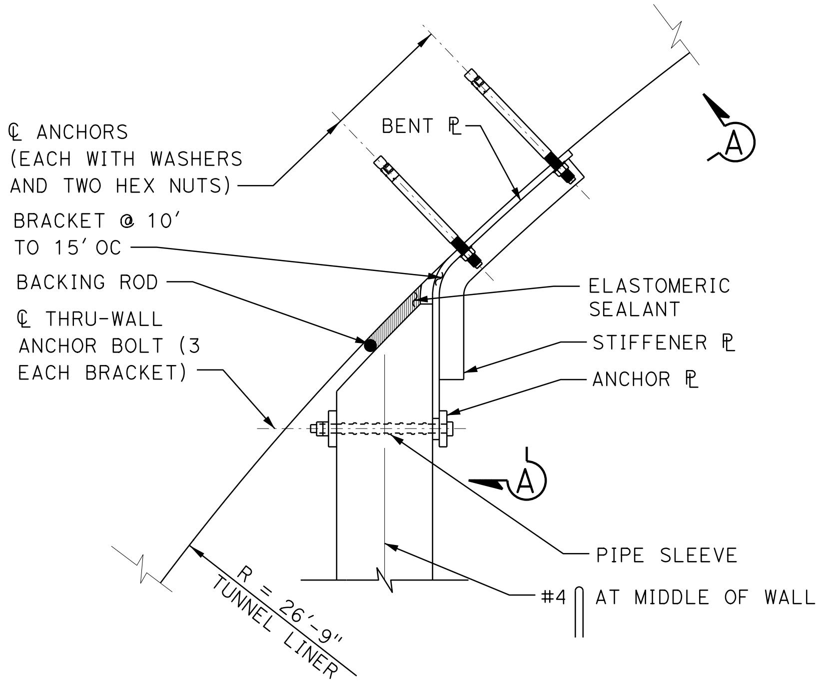
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 13:19

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

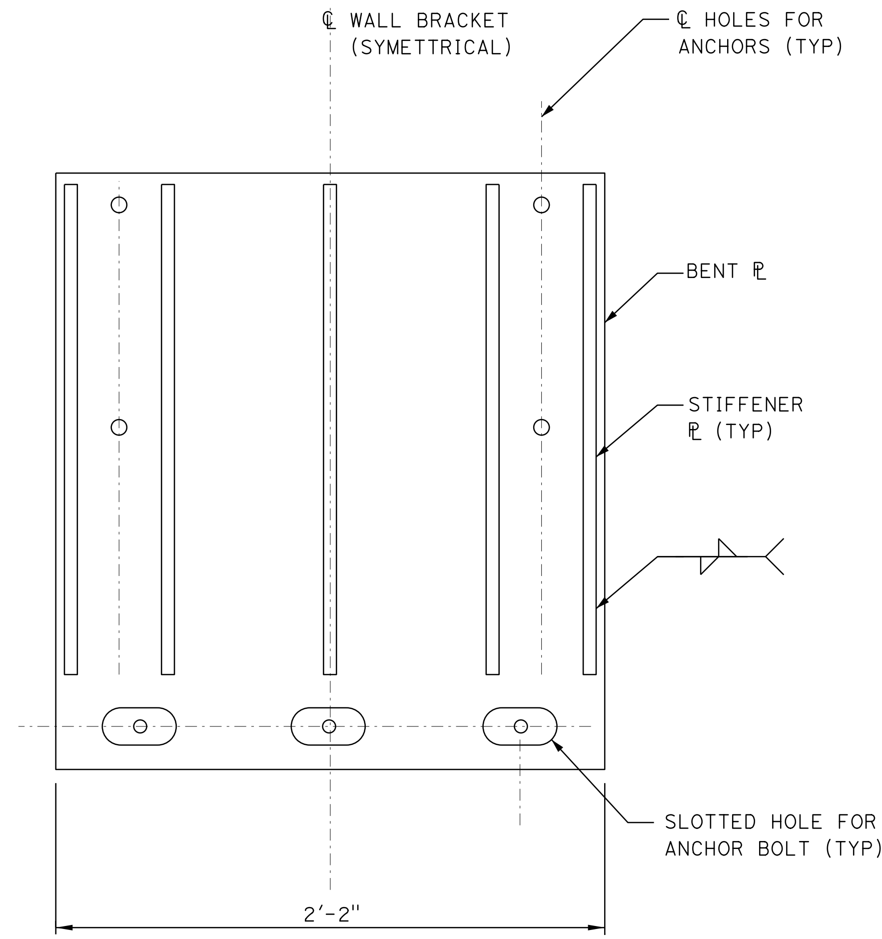
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

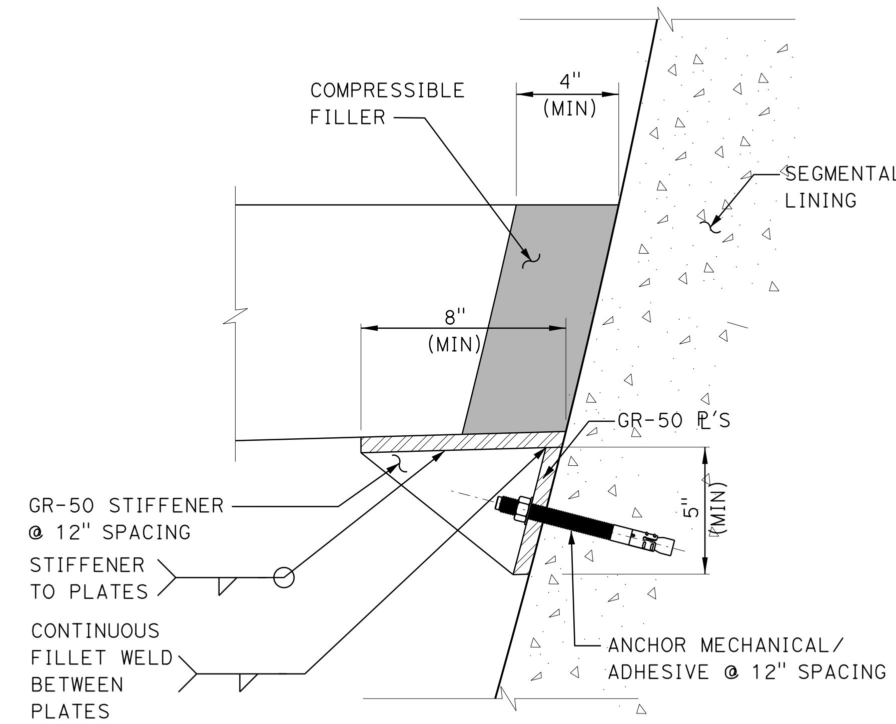
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



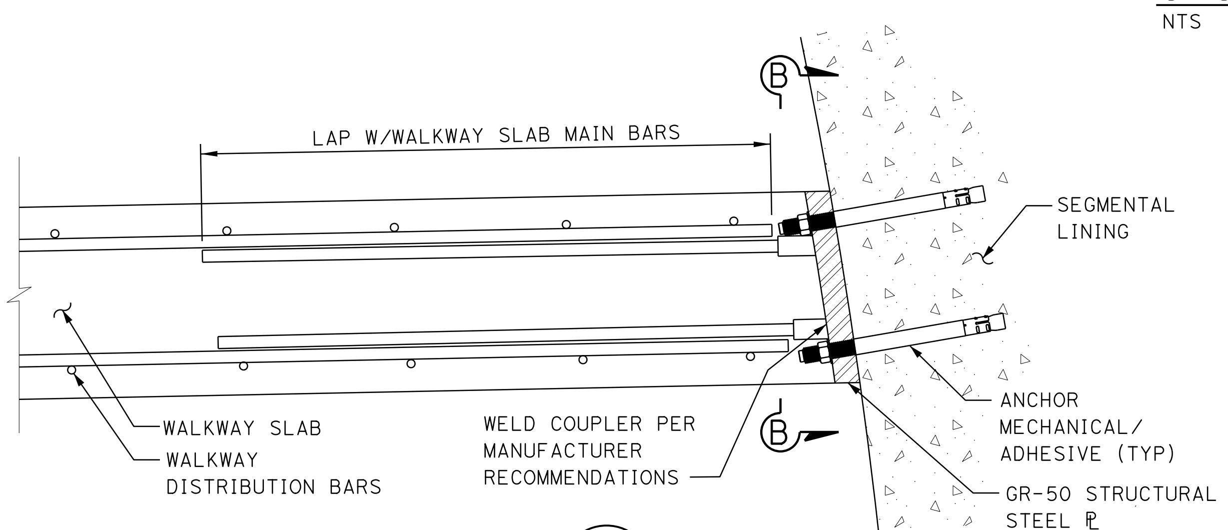
DETAIL 1 / – S2.05
NTS



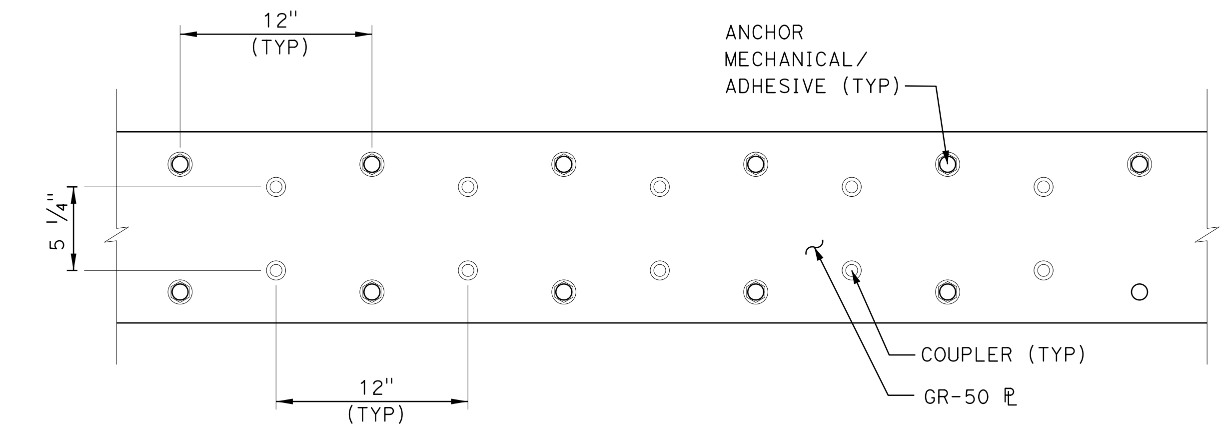
SECTION A / –
NTS



DETAIL 3 / – S2.05
NTS



DETAIL 2 / – S2.05
NTS



SECTION B
NTS

S2.06

DESIGN OVERSIGHT	
SIGN OFF DATE	

DESIGNED BY	J. YAO/M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	J. VANGREUNEN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY
INTERNAL STRUCTURE DETAILS

BRIDGE NO. TBD	UNIT:
SCALE: AS SHOWN	PROJECT NUMBER & PHASE:

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S206.dgn

CONTRACT NO.: X

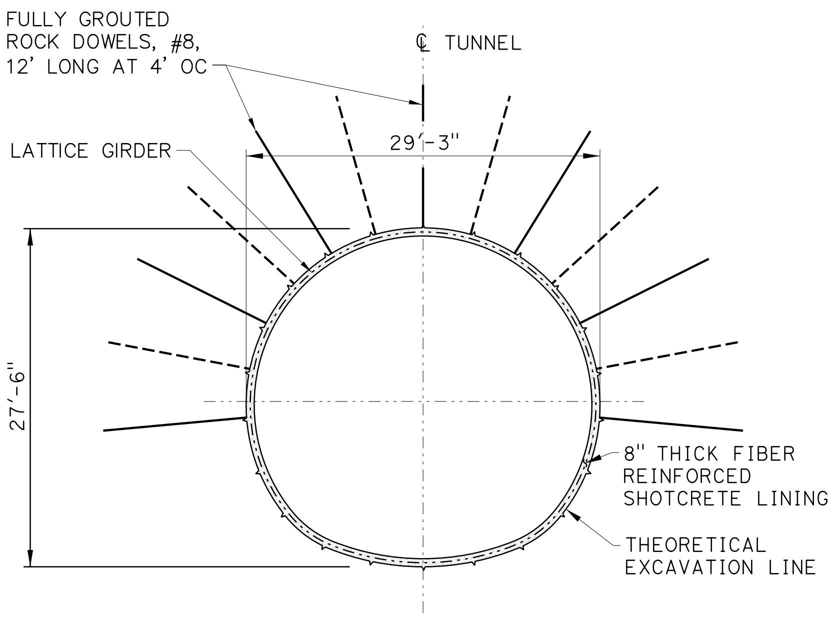
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:35

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

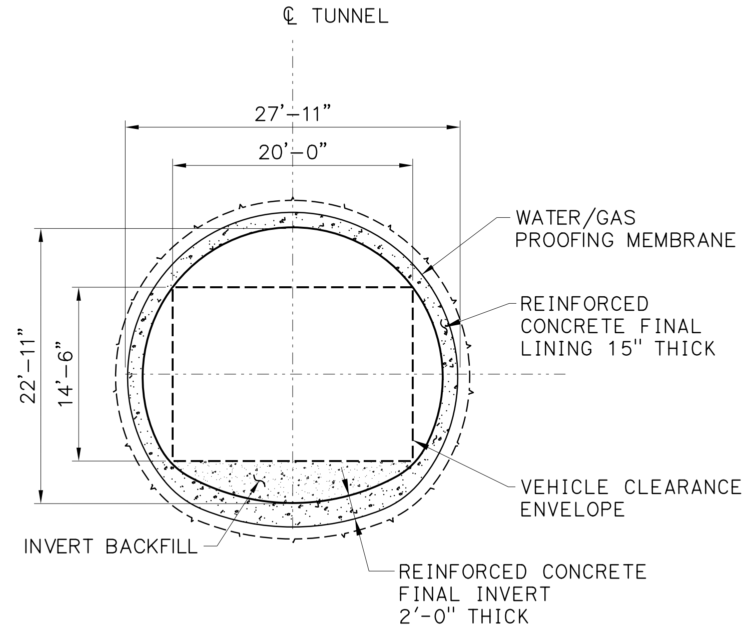
METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



INITIAL LINING SECTION

NTS



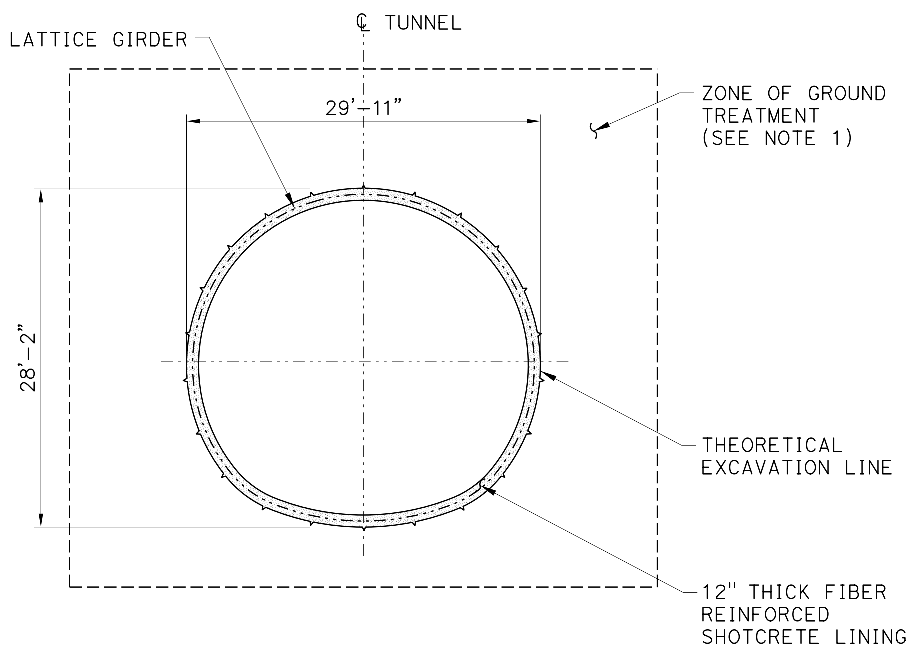
FINAL LINING SECTION

NTS

NOTES

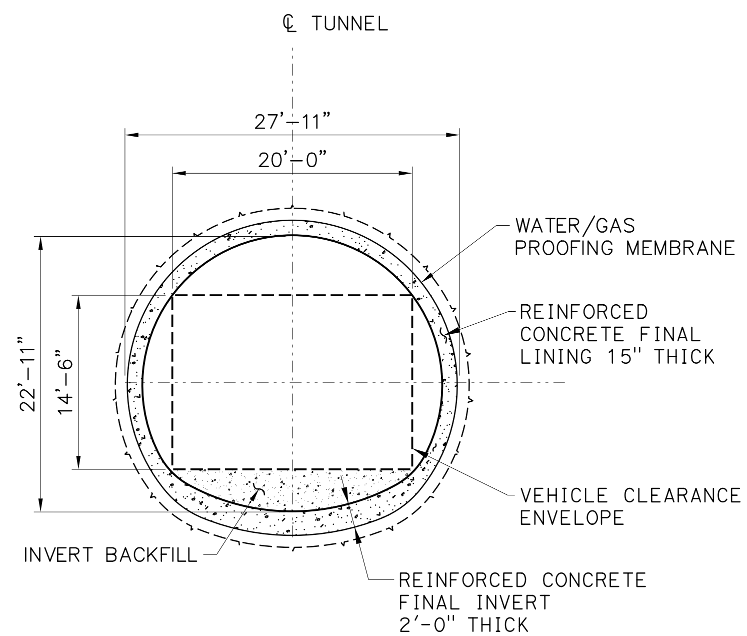
1. GROUND TREATMENT METHODS INCLUDE, BUT ARE NOT LIMITED TO, PERMEATION GROUTING, CHEMICAL GROUTING, OR GROUND FREEZING. TREATED GROUND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 400 PSI AT 28 DAYS.

CROSS PASSAGE IN ROCK



INITIAL LINING SECTION

NTS



FINAL LINING SECTION

NTS

CROSS PASSAGE IN SOIL

S3.01

DESIGNED BY	Y. SUN	DATE	7-23-2013
DRAWN BY	J. TOLES	DATE	7-23-2013
CHECKED BY	S. KLEIN	DATE	2-7-2014
APPROVED	S. DUBNEWYCH	DATE	2-7-2014

PROJECT ENGINEER

SR 710 NORTH STUDY

CROSS PASSAGE 1 OF 7

BRIDGE NO. TBD | UNIT:

SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

X
DESIGN OVERSIGHT

X
SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S301.dgn

CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:35

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

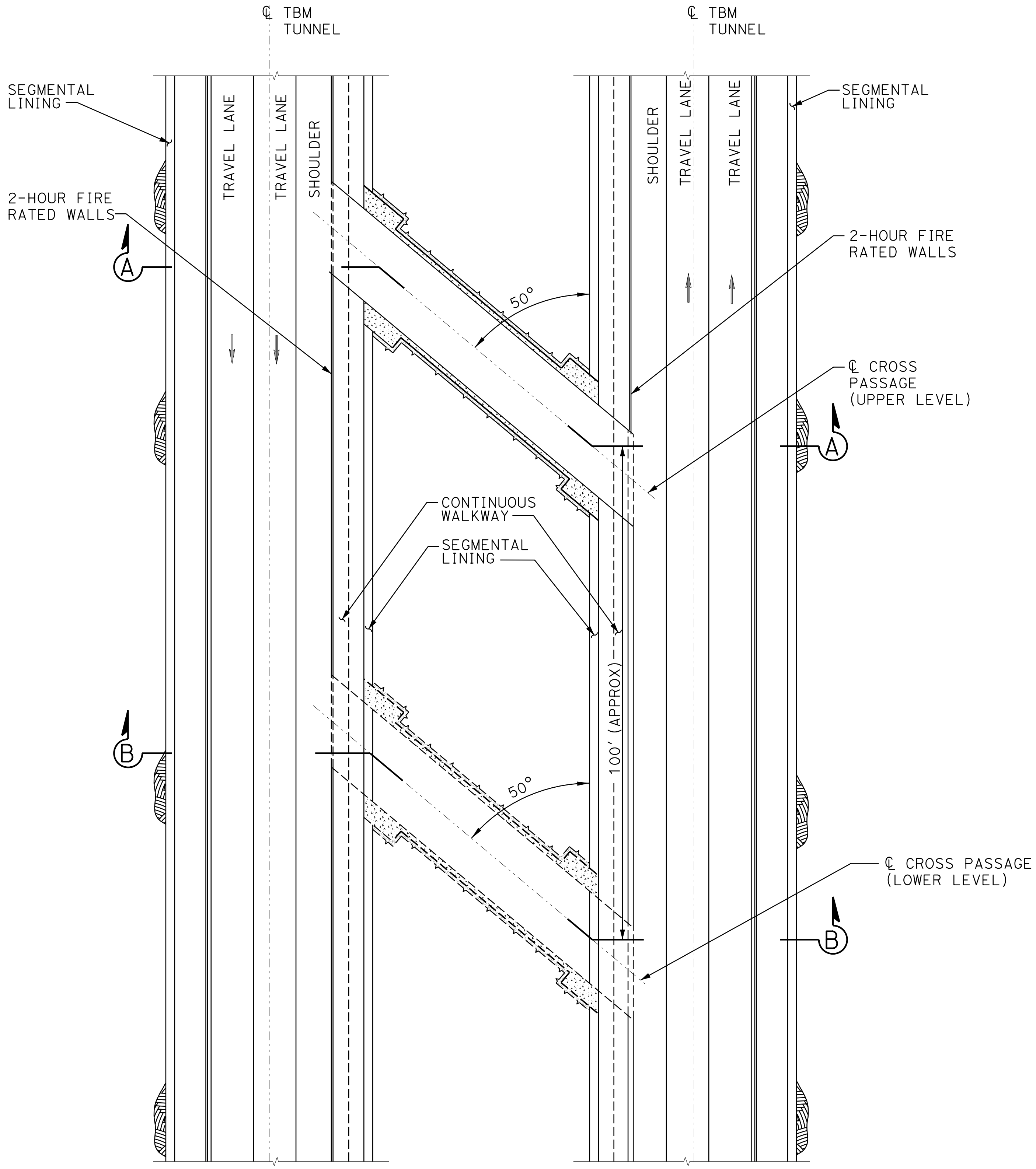
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

NOTES

1. SEE S3.03 FOR SECTIONS A AND B.



PLAN AT UPPER LEVEL ROADWAY

1" = 10'

S3.02

DESIGNED BY	Y. SUN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

CROSS PASSAGE 2 OF 7

BRIDGE NO. TBD | UNIT:

SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S302.dgn

CONTRACT NO.: X

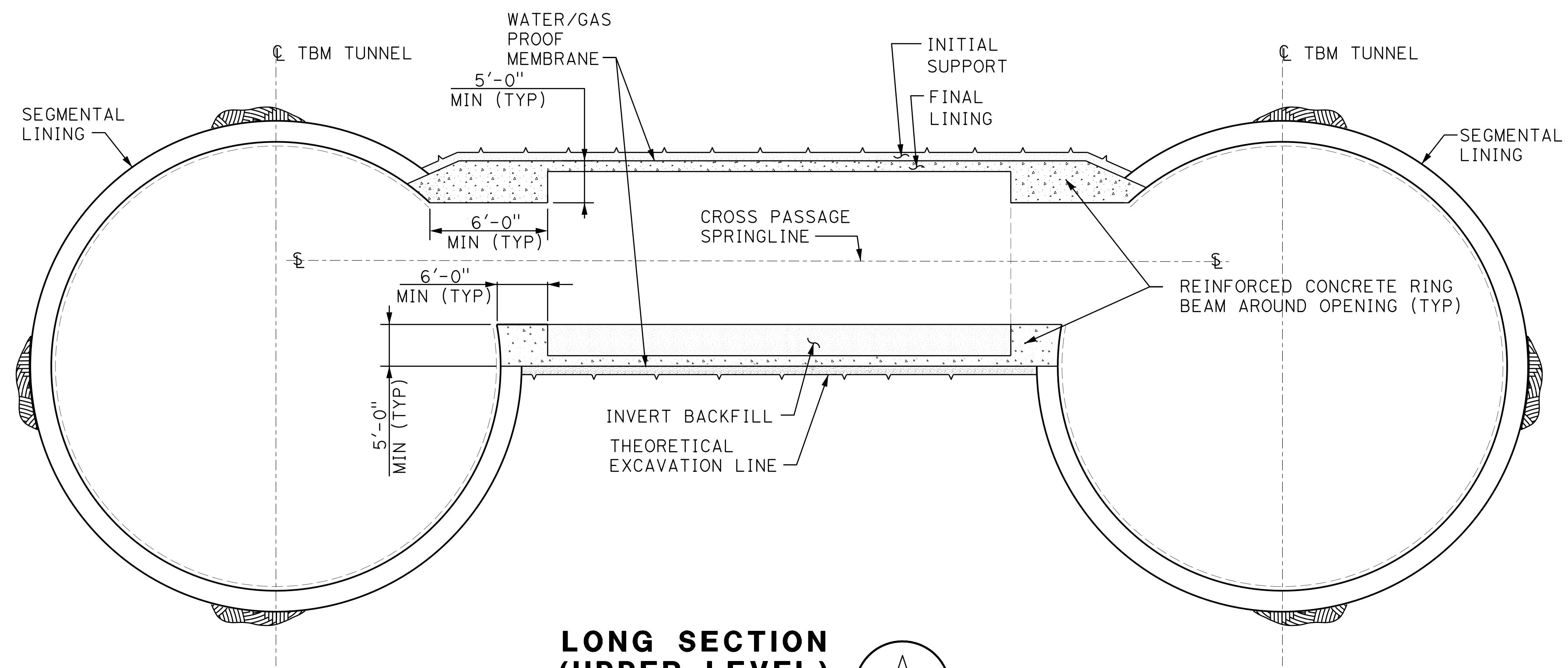
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:35

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

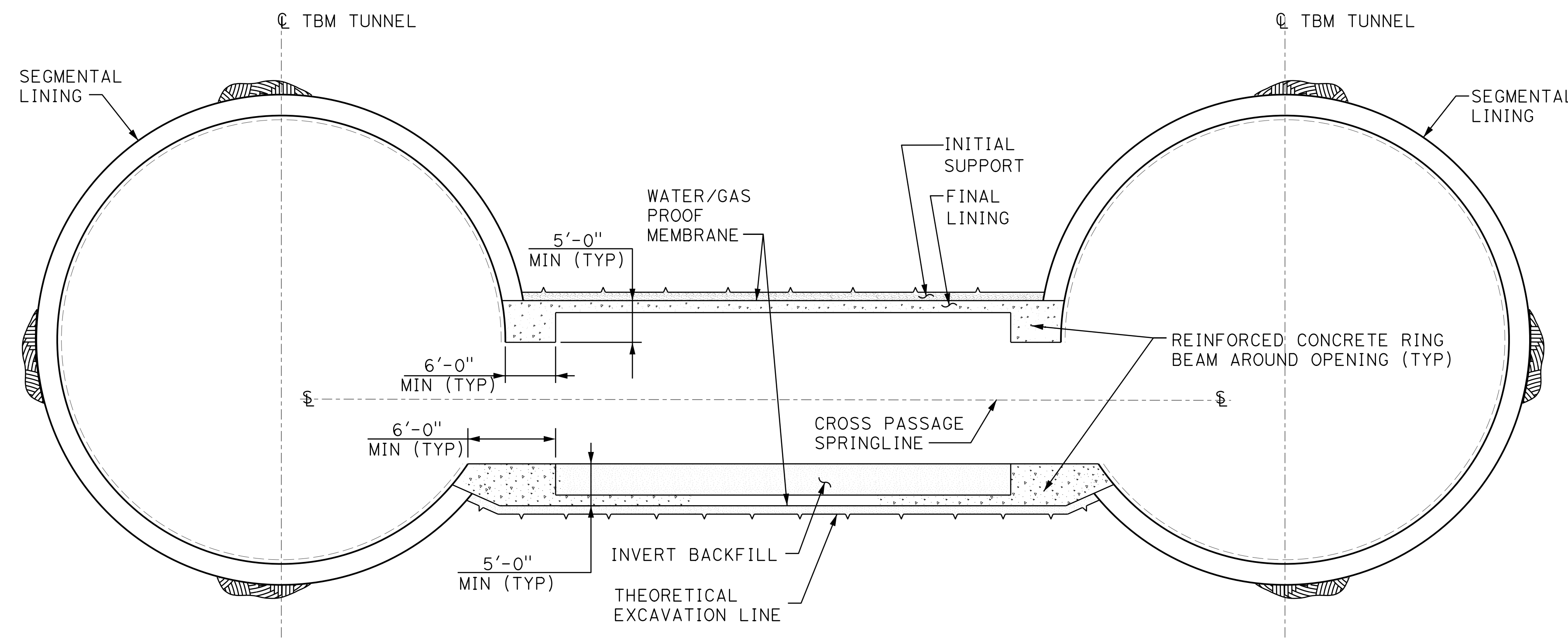
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



LONG SECTION (UPPER LEVEL)
NTS
A
S3.02

NOTES

1. INTERNAL STRUCTURE NOT SHOWN FOR CLARITY. INTERNAL STRUCTURE TO BE BUILT AFTER CROSS PASSAGE CONSTRUCTION.
2. REFER TO S3.01 FOR INITIAL SUPPORT AND FINAL LINING DETAILS.



LONG SECTION (LOWER LEVEL)
NTS
B
S3.02

S3.03

DESIGNED BY Y. SUN	DATE 9-20-13	
DRAWN BY J. TOLES	DATE 9-20-13	X PROJECT ENGINEER
CHECKED BY S. KLEIN	DATE 9-20-13	
APPROVED S. DUBNEWYCH	DATE 9-20-13	

SR 710 NORTH STUDY
CROSS PASSAGE 3 OF 7

BRIDGE NO. TBD | UNIT:
SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S303.dgn

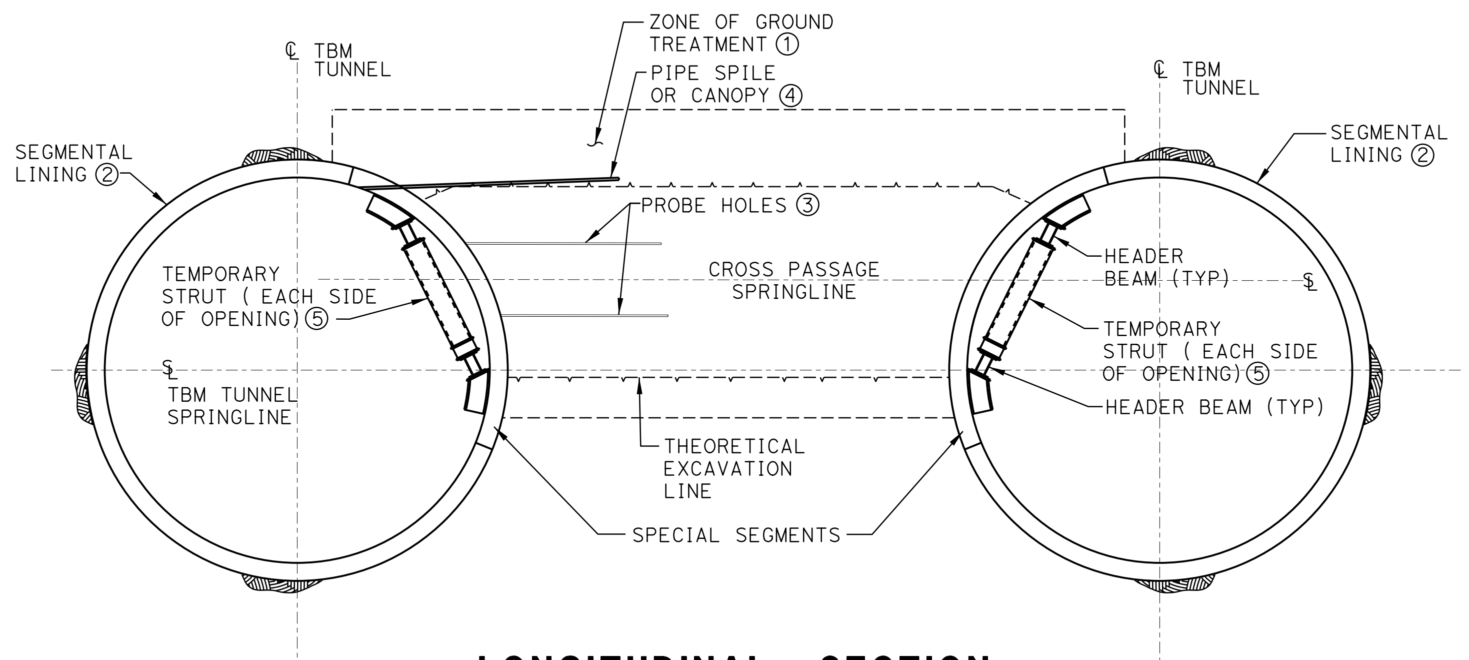
CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:35

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

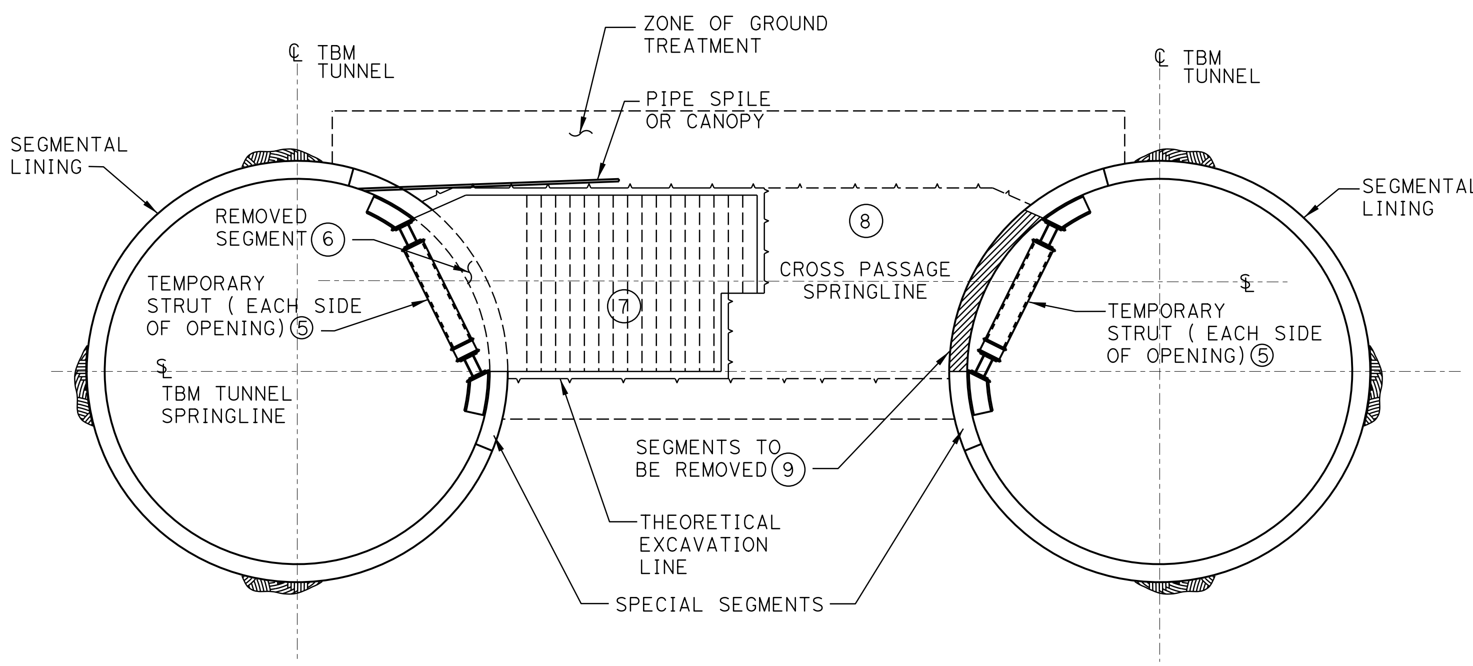
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



LONGITUDINAL SECTION (UPPER LEVEL)

A / –

NTS



LONGITUDINAL SECTION (UPPER LEVEL)

B / –

NTS

CROSS PASSAGE CONSTRUCTION SEQUENCE

1. PERFORM GROUND TREATMENT, IF REQUIRED, PRIOR TO TBM TUNNEL EXCAVATION.
2. EXCAVATE TBM RUNNING TUNNELS AND INSTALL STEEL SEGMENTS AS PART OF TBM SEGMENTAL LININGS. SEE S3.06 FOR LOCATION OF STEEL SEGMENTS.
3. DRILL PROBE HOLES THROUGH INSTALLED SEGMENTS IN BREAKOUT TBM TUNNEL AND DETERMINE/VERIFY CONDITIONS OF TREATED GROUND, IMPLEMENT GROUNDWATER CONTROL MEASURES AS REQUIRED.
4. INSTALL PIPE SPILES OR CANOPY FROM BREAKOUT TBM TUNNEL, IF REQUIRED.
5. INSTALL TEMPORARY BEAMS, STRUTS AND HYDRAULIC JACKS ADJACENT TO CROSS PASSAGE OPENINGS IN BOTH TBM TUNNELS AND PRELOAD STRUTS, SEE DRAWING S3.06.
6. REMOVE SECTIONS OF STEEL SEGMENTS TO FORM OPENING IN BREAKOUT TBM TUNNEL.
7. EXCAVATE CROSS PASSAGE AND INSTALL INITIAL SUPPORT IN STAGES IN ACCORDANCE WITH REQUIREMENTS SHOWN ON S3.07.
8. COMPLETE CROSS PASSAGE EXCAVATION.
9. REMOVE SECTIONS OF STEEL SEGMENTS TO FORM OPENING ON OPPOSITE TBM TUNNEL.

NOTES

1. CROSS PASSAGE CONSTRUCTION SEQUENCE SHOWN ALSO APPLIES TO LOWER LEVEL CROSS PASSAGE CONSTRUCTION.
2. ACTUAL CONSTRUCTION SEQUENCE MAY VARY DEPENDING ON CONTRACTOR'S MEANS AND METHODS.

S3.04

DESIGNED BY	Y. SUN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN	DATE	9-20-13
APPROVED	S.DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

CROSS PASSAGE 4 OF 7

BRIDGE NO. TBD	UNIT:
SCALE: AS SHOWN	PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

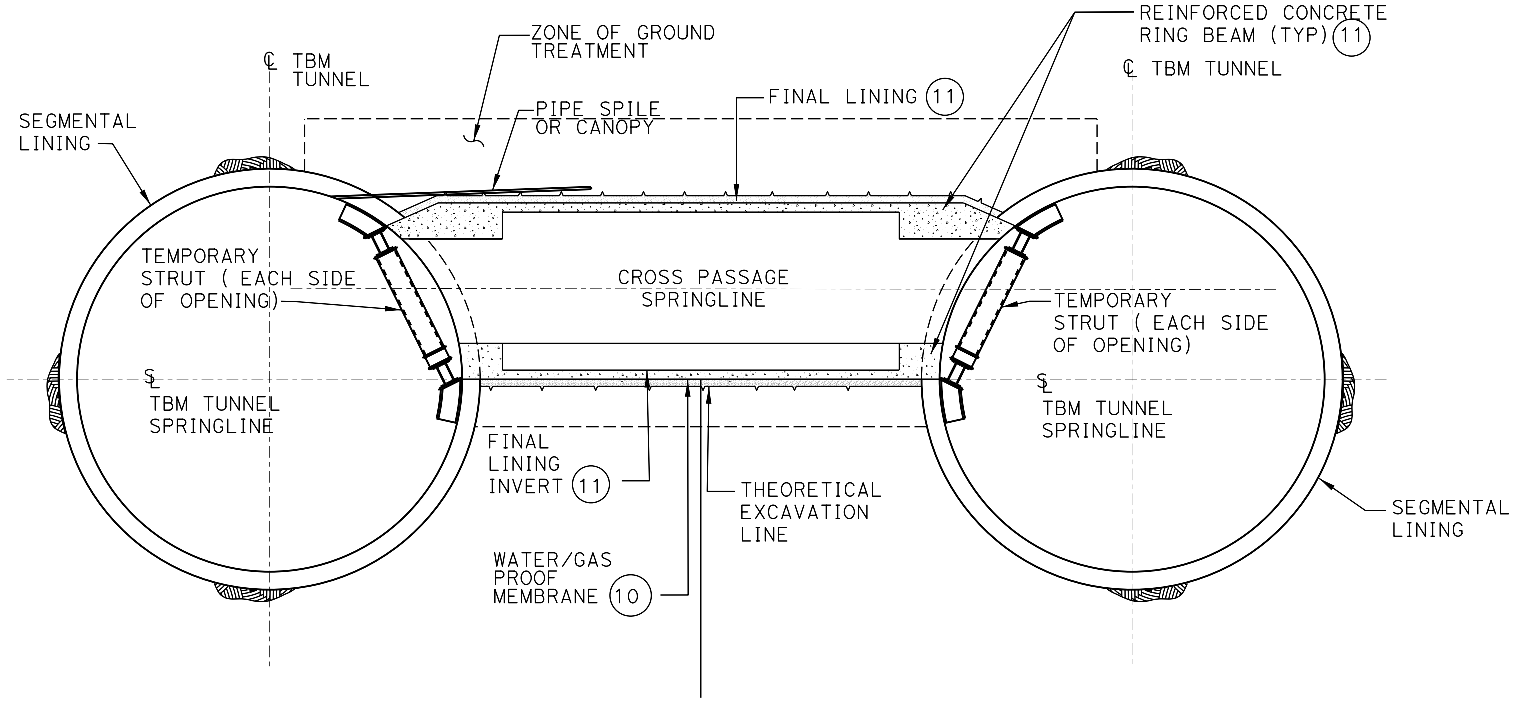
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

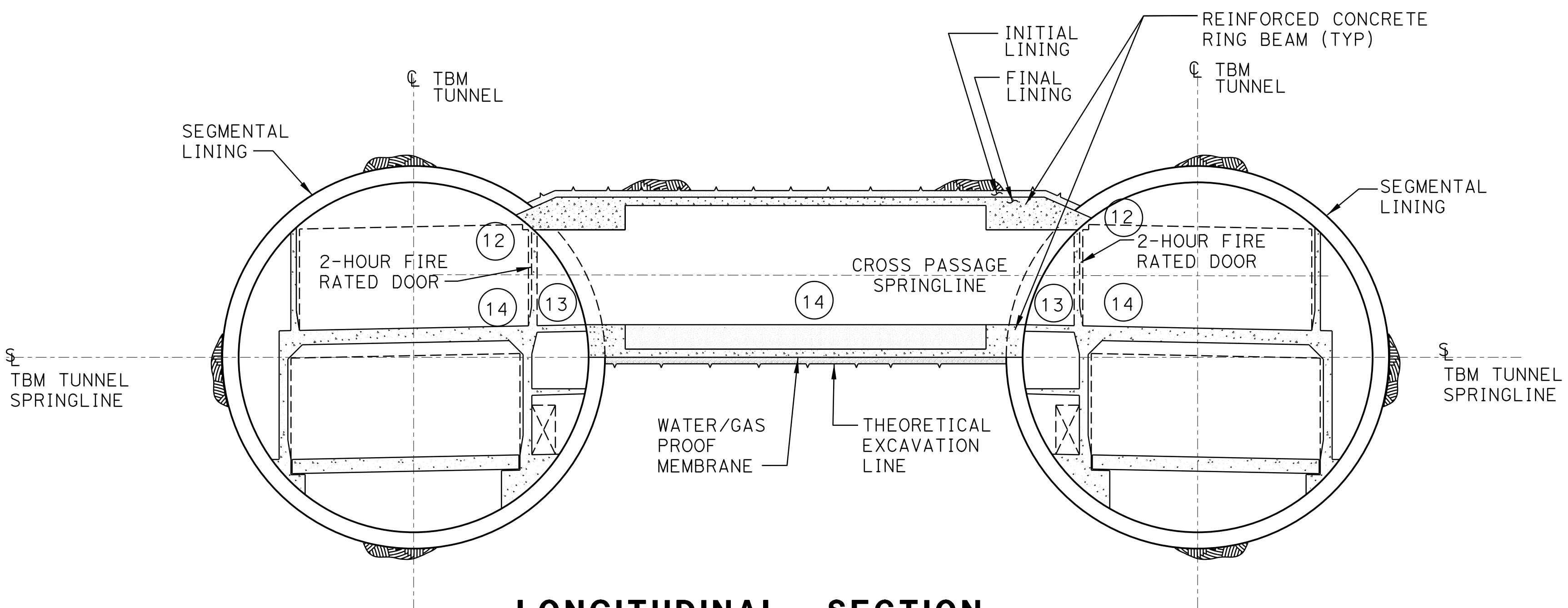


LONGITUDINAL SECTION (UPPER LEVEL) C/–

NTS

CROSS PASSAGE CONSTRUCTION SEQUENCE (CONTINUED)

10. INSTALL WATER/GAS PROOF MEMBRANE.
11. INSTALL REINFORCED CONCRETE RING BEAM AND INVERT.
12. RELEASE JACK PRESSURE AND REMOVE TEMPORARY STRUTS AND BEAMS AFTER FINAL LINING DESIGN STRENGTH IS MET
13. FILL AND ENCASE PERMANENT STEEL SEGMENTS AROUND OPENINGS WITH CONCRETE.
14. COMPLETE INVERT BACKFILL AND ROAD PAVEMENT IN CROSS PASSAGE AND INTERNAL STRUCTURES IN TBM TUNNELS.



LONGITUDINAL SECTION (UPPER LEVEL) D/–

NTS

NOTES

1. CROSS PASSAGE CONSTRUCTION SEQUENCE SHOWN ALSO APPLIES TO LOWER LEVEL CROSS PASSAGE CONSTRUCTION.
2. ACTUAL CONSTRUCTION SEQUENCE MAY VARY DEPENDING ON CONTRACTOR'S MEANS AND METHODS.

S3.05

DESIGNED BY Y. SUN	DATE 9-20-13	
DRAWN BY J. TOLES	DATE 9-20-13	X
CHECKED BY S. KLEIN	DATE 9-20-13	PROJECT ENGINEER
APPROVED S.DUBNEWYCH	DATE 9-20-13	

SR 710 NORTH STUDY

CROSS PASSAGE 5 OF 7

BRIDGE NO. TBD | UNIT:

SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

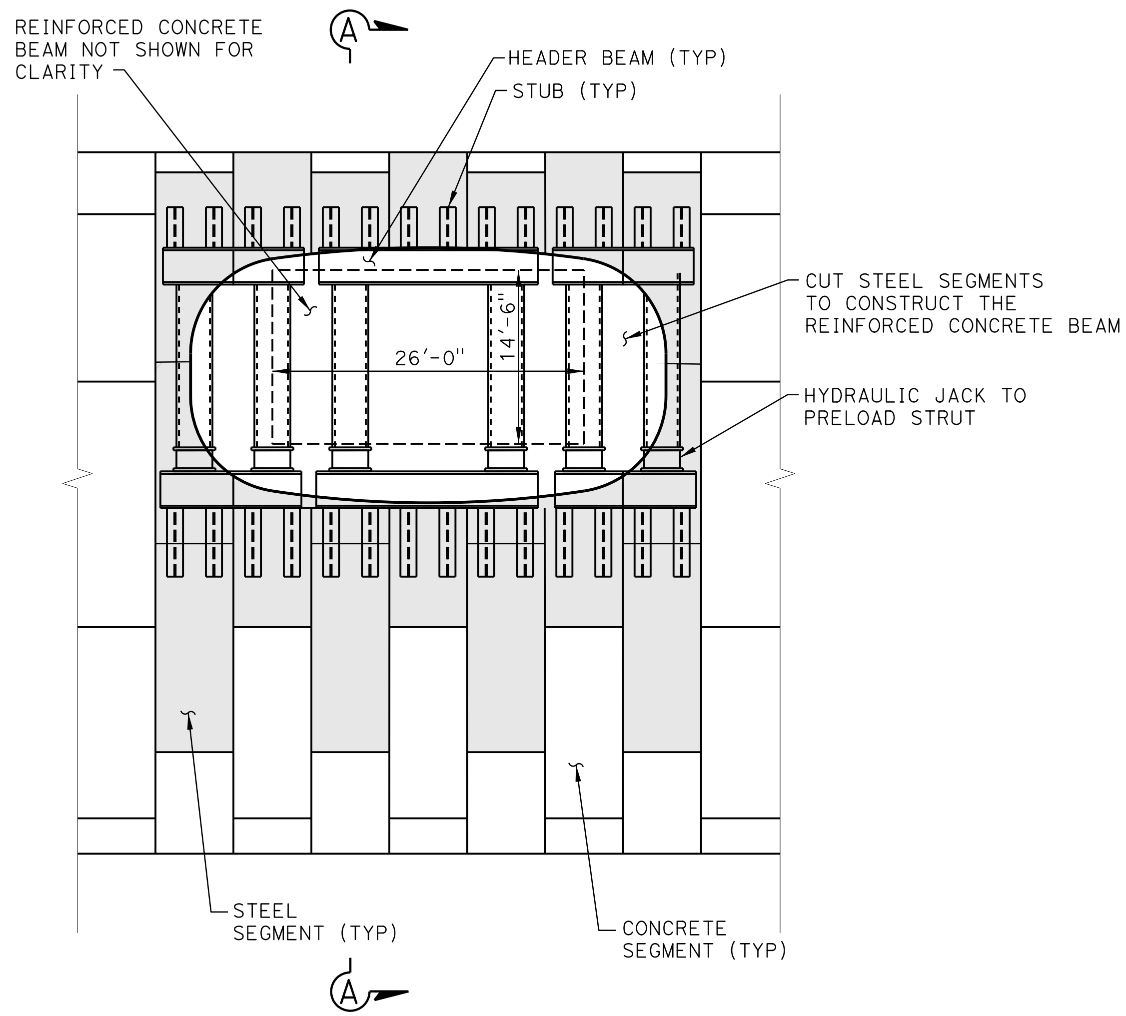
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

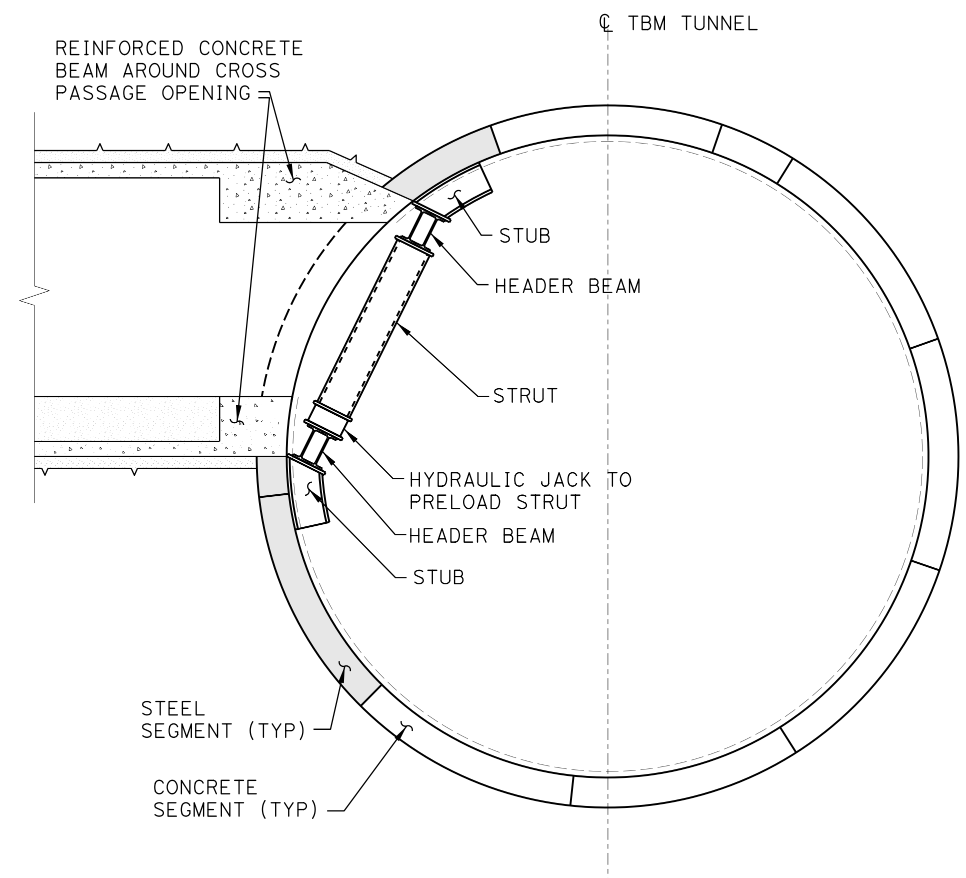
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



ELEVATION AT CROSS PASSAGE LOCATION
NTS



SECTION A / –
NTS

TEMPORARY FRAMING FOR CROSS PASSAGE BREAKOUT

S3.06

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J.TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN/J. VANGREUNEN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

CROSS PASSAGE 6 OF 7

BRIDGE NO. TBD — UNIT:

SCALE: AS SHOWN — PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

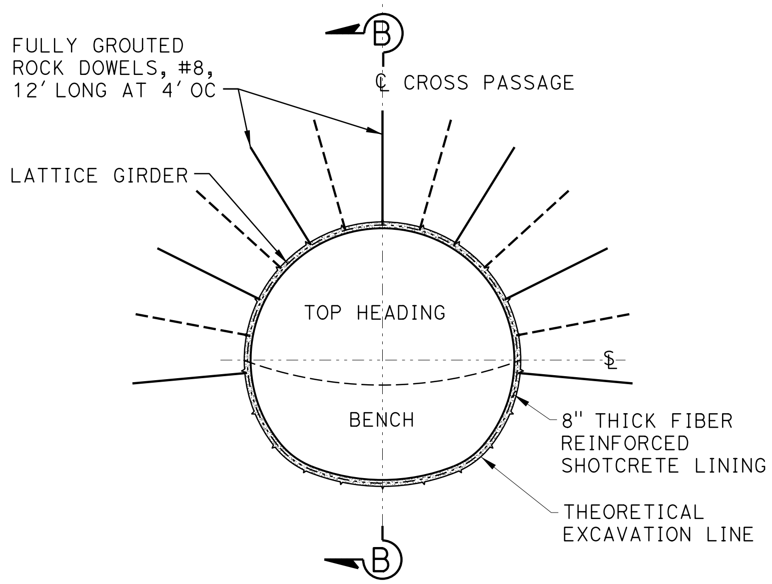
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

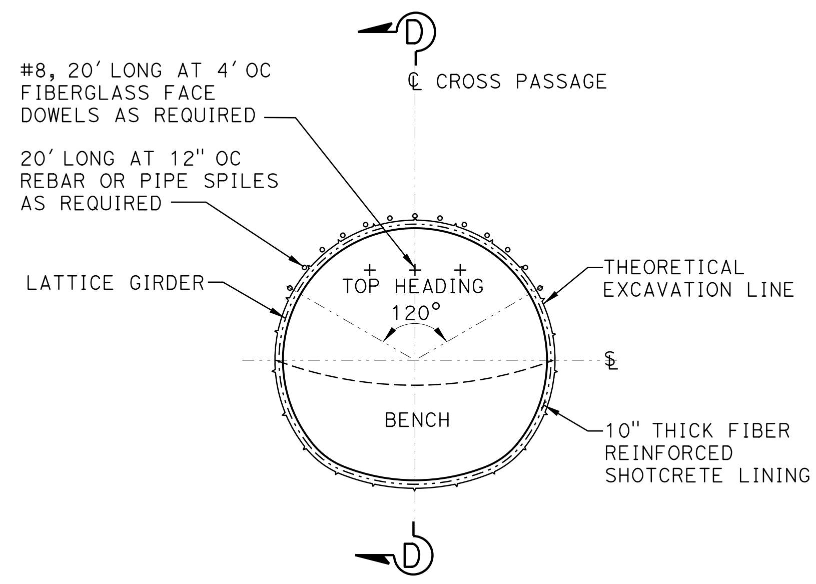
METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



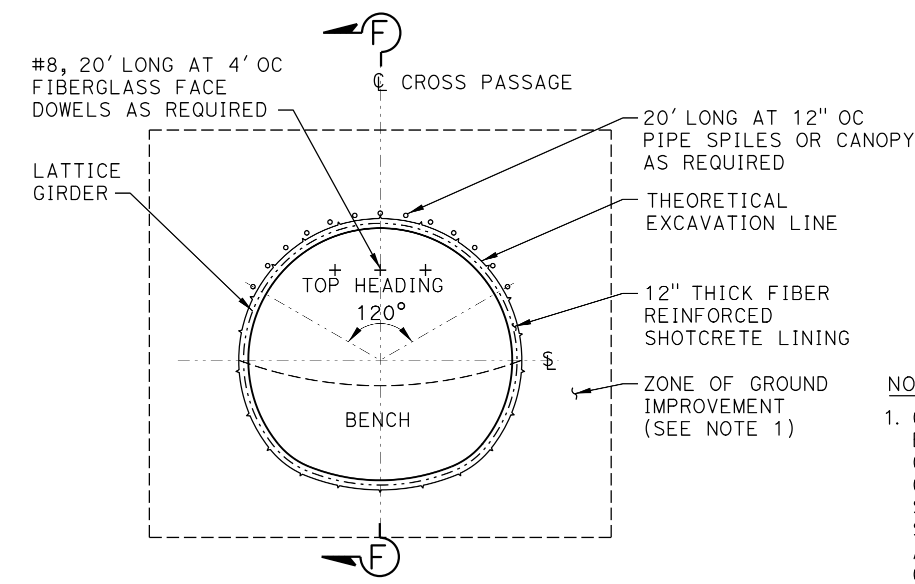
CROSS SECTION (SUPPORT TYPE 1) A/–

NTS



CROSS SECTION (SUPPORT TYPE 2) C/–

NTS

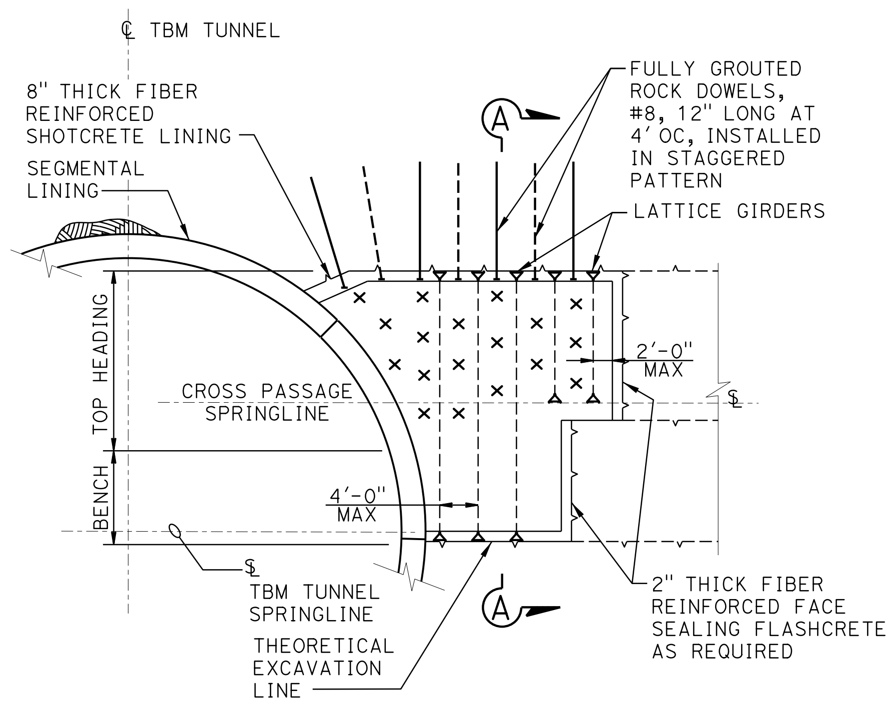


CROSS SECTION (SUPPORT TYPE 3) E/–

NTS

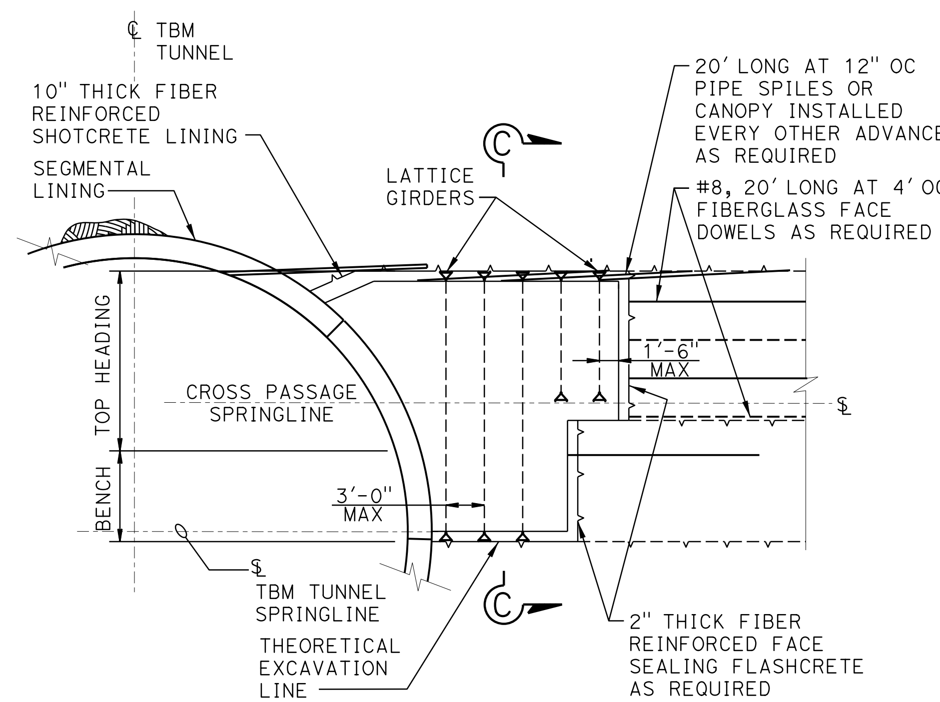
NOTES

1. GROUND IMPROVEMENT METHODS INCLUDE, BUT ARE NOT LIMITED TO, PERMEATION GROUTING CHEMICAL GROUTING, OR GROUND FREEZING TREATED GROUND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 400 PSI AT 28 DAYS. ADDITIONAL IN-TUNNEL GROUNDWATER CONTROL MEASURES SHALL BE IMPLEMENTED AS REQUIRED.
2. CROSS PASSAGE EXCAVATION AND INITIAL SUPPORT REQUIREMENTS SHOWN ALSO APPLY TO LOWER LEVEL CROSS PASSAGE.



LONGITUDINAL SECTION (SUPPORT TYPE 1) B/–

NTS



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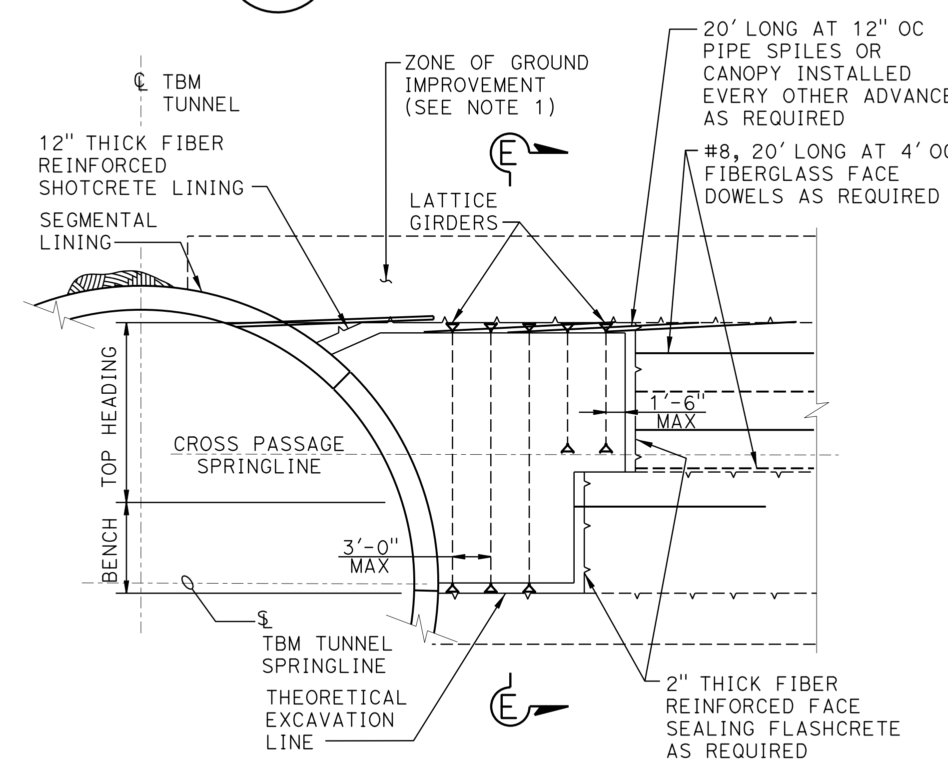

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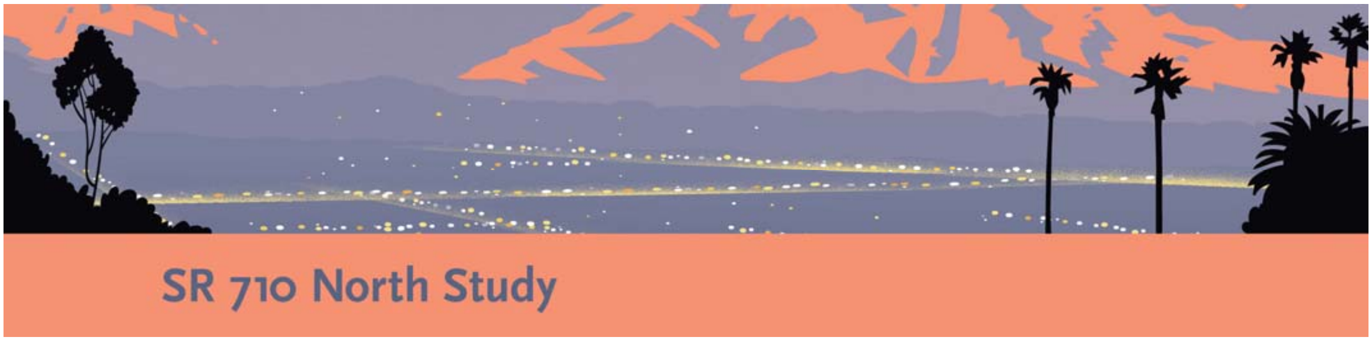
DESIGNED BY Y. SUN	DATE 9-20-13	X PROJECT ENGINEER	SR 710 NORTH STUDY
DRAWN BY J. TOLES	DATE 9-20-13		CROSS PASSAGE 7 oF 7
CHECKED BY S. KLEIN	DATE 9-20-13		BRIDGE NO. TBD / UNIT:
APPROVED S.DUBNEWYCH	DATE 9-20-13		SCALE: AS SHOWN / PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

Appendix E
TM-4B Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings



TECHNICAL MEMORANDUM 4B

Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings

PREPARED FOR: Metro

COPY TO: Caltrans

PREPARED BY: Jacobs Associates/CH2M HILL

DATE: August 22, 2014

PROJECT NUMBER: 428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.

1.2 Task Description and Scope

This technical memorandum (TM) describes the preliminary design concepts developed for bored tunnel and cross passage lining systems for the LRT Alternative. These design concepts were developed from experience with other similar LRT tunnels in the Los Angeles area. The TM provides a general description of the geology along the tunnel alignment, along with discussions of applicable design criteria, general lining requirements, and an overall design methodology that can be applied to the LRT tunnels. Preliminary design concepts for the segmental lining of the bored tunnels, as well as the initial ground support and final lining concepts for the cross passages, are presented on the drawings provided in Attachment A of this TM. This TM does not cover preliminary design concepts for tunnel sections subject to potential fault displacements, which are discussed in a separate TM.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Tunnel Alignment and Geology

2.1 Tunnel Alignment

For most of the alignment the twin-bore tunnels, which have a finished inside diameter of about 19 feet, are spaced approximately one tunnel diameter apart (19 feet), and this distance decreases to about 13 feet at the north and south ends of the underground alignment and as the alignment approaches the stations. The spacing of the tunnel bores will be revisited in subsequent phases of this study based on refined geotechnical parameters.

The tunnel begins at the northern terminus of the existing I-710 freeway, extends in the northwesterly direction, and joins the I-210 freeway near the intersection of the SR-134 freeway in Pasadena. The ground cover depth ranges from about 25 to 90 feet, with an average of approximately 60 feet. The four underground stations along the tunnel portion of the LRT Alternative are expected to be excavated top-down by cut-and-cover methods, and the current plan calls for walking the tunnel boring machines (TBMs) through the station excavations instead of boring through them. However, this sequence could be modified if there were a schedule or cost advantage in doing so, and ultimately would likely be left to the contractor.

Figure 1 shows the tunnel profile of the LRT Alternative. Additional details on the tunnel alignment can be found in *Bored Tunnel Geometry* (JA, 2014a).

2.2 Geologic Conditions

Anticipated geotechnical conditions were evaluated based on data and information described in the Preliminary Geotechnical Report provided by CH2M HILL (2014). Geology in the project area consists of Quaternary-age alluvium, Tertiary-age sedimentary rocks, and crystalline basement complex rocks. Tunnel excavation along the LRT alignment is expected to encounter alluvium, and sedimentary rocks, which include rocks of the Fernando, Puente, and Topanga Formations. The LRT Alternative crosses one active fault (the Raymond fault) and one potentially active fault (the San Rafael fault). Future studies would be performed to evaluate the activity of the San Rafael fault; however, for planning purposes, this fault is treated as an active fault. Figure 1 shows a preliminary geologic profile along the LRT tunnel alignment based on the available geologic data.

Based on the available geologic information, approximately 45 percent and 25 percent of the length of the tunnel excavation is expected to encounter alluvial soil deposits and mixed-face condition, respectively, while the remaining 30 percent is expected to be within sedimentary rocks. The alluvium ranges approximately from 0 to 300 feet thick along the LRT alignment. Along the alignment, a relatively thin layer of artificial fill ranging approximately from 0 to 25 feet thick overlies the alluvium at the southern end of the tunnel alignments.

The alluvium deposits consist of interbedded lenses and/or discontinuous layers of fine-grained soil (clay and silt) and coarse-grained materials (sand and gravel) that include a wide range of soil types. The alluvial soils generally increase in strength with depth. The consistency of the fine-grained soil encountered in the borings ranged from soft to hard; while the density of the coarse-grained materials encountered ranged from loose to very dense. Hard to very hard cobble-size rocks are common locally within the alluvium, and some hard to very hard boulders may be scattered locally throughout the unit. The maximum dimension (i.e., size) of the boulders may be 3 to 5 feet based on descriptions in geotechnical baseline reports (GBRs) from local tunneling projects that include the Regional Connector Transit Corridor, Eastside LRT, and Northeast Interceptor Sewer (NEIS). On a volume basis, the amount of cobbles and boulders baselined in these past reports ranges from significantly less than one percent to a few percent of materials that make up the alluvium. The characteristics and potential amount of the cobbles and boulders that may be encountered during tunneling should be evaluated in later design stage as additional data becomes available.

The Fernando Formation consists primarily of weak, massive, marine claystone, mudstone, and siltstone with some sandstone and conglomerate. The rock mass is slightly to very slightly fractured, moderately to slightly weathered, and extremely weak to very weak, with scattered, hard concretions and very thin to thin hard layers. The Puente Formation that is expected to be encountered along the tunnel alignments consists predominantly of the siltstone unit. This unit consists of thinly bedded to laminated siltstones with medium to thick interbeds to laminations of fine-grained sandstone. The rocks generally are weak with locally hard cemented interbeds and concretions. The Topanga Formation that is expected to be encountered along the tunnel alignment consists predominantly of the siltstone unit south of the Raymond Fault, and the sandstone and conglomerate units north of the Raymond Fault. This siltstone unit consists of thinly bedded to laminated and fissile siltstones and shales, with fine- to coarse-grained sandstone interbeds. The sandstone unit consists of laminated to moderately bedded, medium- to coarse-grained sandstone with thin interbeds and laminations of fine-grained sandstone, siltstone, and/or shale with some conglomerate beds. The conglomerate unit consists of rounded cobbles and fine gravel in a medium- to coarse-grained friable arkosic sand matrix, with sandstone beds present locally. Localized, well-cemented concretions were encountered throughout the formation. The cemented zones, layers, and concretions within the Fernando, Puente and Topanga Formations are generally strong and can be hard to very hard. Additionally, based on local tunneling experience, gas could be encountered in several of these formations expected along the alignments.

The unconsolidated alluvial sediments of the San Gabriel Valley and the Raymond Basins constitute important groundwater basins in Southern California. The groundwater aquifers occur in sand and gravel deposits of the basins and have been actively exploited by local communities for the last few decades as a source of groundwater. These deep aquifers are overlain locally by perched groundwater bodies. The estimated groundwater levels along the LRT alignment are shown in Figure 1. Based on the map of groundwater basins in the project area (CH2M HILL, 2014), both the freeway and LRT tunnel alignments would likely traverse through the aquifers where tunnel excavations are expected to encounter alluvial soils below the groundwater table. The depth to groundwater ranges from less than 10 feet to approximately 160 feet, with groundwater levels up to approximately 70 feet above the tunnel crown.

Refer to *Tunnel Ground Characterization* (JA, 2014b) for additional details on the anticipated geologic conditions and potential ground behaviors along the LRT alignment.

3 Description of Underground Structures

3.1 Bored Running Tunnels

The LRT bored tunnels would be excavated using a pressurized Tunnel Boring Machine (TBM) and would have a finished inside diameter of 18 feet 10 inches. The tunnel excavation is expected to be supported with a one-pass bolted and gasketed precast concrete segmental lining. This lining would provide both initial support of the tunnel excavation and the final lining for the bored tunnels. Refer to the drawings in Attachment A for details regarding the concrete segmental lining.

3.2 Cross Passages

Twenty-six cross passages would be constructed along the LRT twin-bore running tunnels. The spacing between adjacent cross passages is approximately 750 feet. Figure 1 shows the approximate locations of cross passages along the alignment. These cross passages would be designed and constructed for riders and personnel to move between the tunnels in the event of an emergency or planned maintenance activities.

Based on the expected ground conditions and cross passage lengths, the excavation and support of the cross passages would be performed using the Sequential Excavation Method (SEM). This method, also known as the New Austrian Tunneling Method (NATM), provides flexibility in the geometry of the opening. The typical tunnel cross sections for SEM/NATM include elliptical or modified horseshoe-shaped configurations to promote smooth stress redistribution in the ground around the opening. Refer to the drawings in Attachment A for cross passage details.

4 Design Criteria and General Requirements

4.1 Codes, Standards, and Guidelines

The codes, standards, and guidelines generally applicable to lining design include, but are not limited to the following:

- Los Angeles Metropolitan Transportation Authority, Metro Rail Design Criteria. Revised April 16, 2013.
- State of California Department of Transportation (Caltrans) Bridge Design Specification (BDS) referenced "LRFD Bridge Design Specifications", 4th Edition, 2007, (Including 2008 and 2009 Interims), by the American Association of State Highway and Transportation Officials (AASHTO).
- American Concrete Institute, "Building Code Requirements for Structural Concrete and Commentary," ACI 318-11 & ACI 318R-11, 2011.
- American Institute of Steel Construction, "Code of Standard Practice for Steel Buildings and Bridges," 13th Edition, (LRFD), 2005.
- US Department of Transportation, Federal Highway Administration, National Highway Institute, "Technical Manual for Design and Construction of Road Tunnels – Civil Elements", FHWA-NHI-10-034, 2009.
- ASCE Technical Council on Research, Technical Committee on Tunnel Lining Design, "Guidelines for Tunnel Lining Design," edited by T. O'Rourke, American Society of Civil Engineers, Geotechnical Engineering Division, 1984.
- The Concrete Society, Technical Report No. 63: Guidance for the Design of Steel-Fibre-Reinforced Concrete, CCIP-017. 2007.

4.2 Service Life and Durability

The design life for these permanent underground structures is 100 years in accordance with Section 5 of the Metro Rail Design Criteria (2013). All portions of the lining need to be designed to provide the required design life. Key portions of the design impacted by design life include required concrete cover for conventional rebar; gasket design of segmental lining; design for the effects of potential fires in the tunnels; and design for durability.

For fire resistance, the tunnel lining would need to be able to withstand the heat of Metro-specified fire intensity and period of time without loss of structural integrity per Section 5 of the Metro Rail Design Criteria. Metro's Fire/Life Safety Criteria also state that when line sections are to be constructed by a tunneling method through earth, unprotected steel liners, reinforced concrete, shotcrete, or equivalent shall be used, except rock tunnels may utilize steel bents with a concrete lining, if lining is required. Special liner requirements may be imposed to assist control of natural gas intrusion and, where utilized on the tunnel interior, shall be of noncombustible construction.

Design for durability consists of selecting appropriate materials to ensure structure integrity over the project design life. Factors that enhance the density and prevent long-term degradation of concrete, including investigation and testing of sources of metal corrosion such as soil and groundwater, dissimilar metals, and stray currents, should be evaluated and considered in determining performance of the tunnel lining.

4.3 Tunnel and Cross Passage Dimensions

The size of the LRT bored tunnels is governed by the Rail Vehicle clearance requirements specified in Section 5 of the Metro Rail Design Criteria. The Rail Vehicle clearance is defined as the distance from the track centerline, measured to the face of obstruction and includes the Vehicle Dynamic Envelope (VDE), Pantograph Dynamic Envelope (PDE), construction tolerances, running clearances, evacuation walkway, safety space and operating envelopes. Clearance diagrams for bored tunnels based on Metro criteria are shown in Figure 2.

According to Metro's fire/life safety criteria, the cross passage shall have a minimum clear, unobstructed width of 6 feet 6 inches, and shall have a desirable height of 8 feet and a minimum height of 7 feet. Additionally, the distance between cross passages shall be 750 feet nominally, and shall not exceed 800 feet (unless authorized by Fire/Life Safety Committee).

4.4 Structural Design

Section 5 of the Metro Rail Design Criteria includes design criteria of tunnel linings, and references the latest edition of FHWA-NHI-09-010 "Technical Manual for Design and Construction of Road Tunnels – Civil Elements", and AASHTO LRFD Bridge Design Specifications.

4.4.1 Design Loads. The design of the tunnel lining must consider the following loads specified in Section 5 of the Metro Rail Design Criteria.

- Dead Loads (DC, DW): Weight of structures and other permanent elements, including internal structures/facilities such as invert slab, walkway, and tunnel utilities.
- Ground Loads (EH, EV): Vertical and horizontal soil or rock loads.
- Hydrostatic (Water) Loads (WA): Groundwater pressure.
- Earth Surcharge Forces (ES): Lateral stresses due to surcharge loads.
- Seismic Loads (EQ): Effects of design earthquake ground motions.
- Live Loads (LL): Weight of light rail vehicle, maintenance car, people and/or other live loads.
- Live Load Surcharge (LS): Load from moving vehicles above the tunnel.
- Dynamic Load Allowance (IMV, IMH): Statically equivalent dynamic effect resulting from vertical and horizontal acceleration of LL.
- Centrifugal Force (CE): Horizontal radial force at curves.
- Longitudinal Force (LF/BR): Forces due to acceleration and deceleration.
- Temperature Loads (TU, TG): Effects due to temperature.
- Shrinkage Load (SH): Effects of shrinkage (applicable only for cast-in-place concrete lining).

Lateral pressures due to surface surcharge loads are only applicable when the tunnel is at shallow depths and are practically non-existent for deeper portion of the tunnel. Shrinkage of cast-in-place concrete linings are typically addressed by providing adequate reinforcing that meets or exceeds the minimum amount specified in ACI 318, and with properly planned construction sequences. The effects of friction and temperature changes for tunnel linings (e.g., temperature changes of the ground) are typically negligible, except under unusual condition such as a fire event inside the tunnel. The effect of fire should be evaluated in future studies based on Metro-specified fire event. The effects of centrifugal force and train acceleration and deceleration on the tunnel lining are typically negligible for tunnel structures, but should be verified in future design phases if the LRT alternative is selected.

In addition to the loads listed above, construction loads such as TBM jacking loads, segment stacking and handling, and contact grouting need to be considered.

4.4.2 Load Factors and Load Combinations

Segmental Lining and Cross Passage Final Lining

The load factors and typical load combinations to be used for the LRT segmental lining and cross passage final lining shall be based on Section 5 of the Metro Rail Design Criteria and the discussion of applicable loads above. It should be noted that load factors for construction load such as TBM jacking loads, segment stacking and handling, contact grouting, etc., are not covered by codes and should be determined on a project-specific basis. The abovementioned analyses will be performed during future design phases if the LRT Alternative moves forward. Any additional load combinations that may be identified as critical for the project will also be considered.

Cross Passage Initial Support

Load factors for initial support (e.g., shotcrete, rock dowels, lattice girders) are not specified in the Metro Rail Design Criteria. The load factors and load combinations for the LRT cross passage initial support could be based on

Max and Mattle (2003) and the Concrete Society (2007). Typically, the design of the initial support does not include a design earthquake event during construction.

4.4.3 Seismic Design. Seismic design of the tunnel linings will be performed in accordance with the Metro Supplemental Seismic Design Criteria, appendix of Section 5 of the Metro Rail Design Criteria (Metro, 2013). Metro Rail uses a two-level approach to seismic design, which includes the Maximum Design Earthquake (MDE) and the Operating Design Earthquake (ODE). The MDE is defined as a seismic event that has a 4 percent probability of exceedance in 100 years (i.e., a return period of about 2,500 years). The tunnel should be capable of surviving the MDE event without collapse, and the structures are allowed to behave in an inelastic manner. The ODE is a lower severity earthquake and is defined as a seismic event that has a 50 percent probability of exceedance in 100 years (i.e., a return period of about 150 years). The tunnel must remain serviceable with no interruption in rail service during or after an ODE, and the structures are to behave in an essentially elastic manner.

4.5 Water/Gas Proofing

In tunnel sections below the water table, the lining must be capable of being made watertight and, if necessary, gastight, by means of sealing gaskets and/or caulking, based on Section 5 of the Metro Design Criteria (Metro, 2013).

5 Design Methodology Summary

Detailed structural design of the tunnel lining should consider the anticipated ground conditions and behaviors summarized in *Tunnel Ground Characterization* (JA, 2014b), the various loading conditions described in Section 4 of this TM, as well as the maximum allowable settlement due to tunnel excavation discussed in *Evaluation and Control of Ground Movements* (JA, 2014d). The design and analysis of the tunnel lining are typically performed based on analytical methods and past experience with similar tunnels under similar ground conditions. This section provides a summary of methods that are generally used for the design of tunnels similar to the LRT tunnels.

5.1 Static Design

Analytical methods for tunnel lining design include closed-form solutions, beam-spring models, and numerical methods. The ground may be defined as an elastic medium or other nonlinear material model. Numerical modeling programs can analyze two- or three-dimensional models and provide analysis results for each stage of the excavation and support installation sequence specified by the user.

One or more analytical methods can be used to evaluate the following aspects of lining design:

- Demand versus structural capacity of the lining under various loading combinations.
- Sensitivity studies to evaluate the effect of varying key design parameters such as in-situ stress (horizontal-to-vertical stress ratios) and rock mass strength and deformation properties.
- With numerical methods, face stability, tunnel convergence/deformations, and performance of ground support elements in providing tunnel stability can be evaluated at each stage of the excavation and support sequence. This is particularly useful for tunnels excavated using the Sequential Excavation Method (SEM).
- With numerical methods, effects due to the excavation of the adjacent tunnel can be evaluated.

The selection of the appropriate design methodology depends on a number of factors such as tunnel construction method and sequence (e.g., bored or sequentially excavated tunnel), tunnel dimensions and geometry, geologic conditions (e.g., uniform or layered geologic profile), and stage of the design.

5.2 Seismic Design

When seismic (shear) waves propagate perpendicular to the tunnel axis, the tunnel would experience shear distortions, resulting in transverse ovaling deformations. While ovaling may be caused by waves propagating horizontally or obliquely, vertically propagating shear waves are the predominant cause of ovaling (Hashash et al., 2001). The seismic effect of ovaling deformations on the tunnel lining system may be evaluated using closed-form elastic solutions for circular tunnels or numerical modeling.

Closed-form solutions for estimating ground-structure interaction are based on the assumptions that the tunnel lining is an elastic, thin-walled tube located in ground consisting of an infinite, homogeneous, and isotropic medium (Wang, 1993). Closed-form solutions summarized in FHWA (2009) can be used to develop initial estimates of the forces and deformations induced in the tunnel lining due to seismic shear waves. Key inputs for these solutions include free-field shear strain of the ground (i.e., ratio of PGV to effective shear wave velocity), tunnel radius, and relative stiffness of the ground and the lining.

In addition to closed-form solutions, the seismic effect of ovaling deformations on the tunnel lining can be evaluated using numerical modeling software program such as FLAC (Itasca, 2005) and PLAXIS (PLAXIS 2D, 2010). In the analysis, ground deformations due to the free-field shear strain are applied to the model in order to the estimate the resulting structural forces and deformations. Key assumptions for numerical analysis include the following:

- Effect of seismic shear wave is simulated by applying displacement at the far external boundaries of the model. Displacements are estimated based on the free-field shear strain of the ground (i.e., ratio of PGV to effective shear wave velocity).
- Ground is modeled as linear-elastic material with effective dynamic Young’s modulus and Poisson’s ratio.
- Unlike the closed-form solutions, the interface between the lining and the ground may be modeled as in between full-slip and no-slip, which is the condition for most tunnels.

The forces from the seismic analysis are then combined with the forces obtained from static analysis to determine if the structural design criteria are satisfied.

6 Preliminary Design Concepts for Bored Tunnels

The bored tunnels would have a one-pass lining system consisting of a concrete segmental lining with gasketed joints. Typical concepts for the lining of the LRT tunnels are shown on the drawings in Attachment A. For the design of tunnel sections subject to potential fault displacements, refer to *Preliminary Design Concepts for Fault Crossings* (JA, 2014e).

At this stage of the design, the lining design concepts for the LRT tunnels is based primarily on relevant past experience, which includes the Regional Connector Transit Corridor and Metro Gold Line Eastside Extension projects in Los Angeles, as well as the University-Link Rail in Seattle, Washington. These light rail projects also involve twin-bore tunnels with similar diameters as the LRT tunnels, and in some of the same geologic formations and seismic settings (the proposed Regional Connector and Metro Gold Line tunnels). More detailed structural analysis following the methodology outlined above will have to be performed for the next stage of design, if this alternative is to be advanced to the next level of completion.

6.1 Segmental Lining Geometry

Although the ring geometry is usually determined by the contractor, there are some general requirements associated with the geometry, including ring tapers to negotiate curves along the alignment, avoidance of cruciform joints between segments, and selection of tolerances for both fabrication and installation to minimize potential leakage into the tunnel.

Various inserts are required in the segmental linings to facilitate the ring build process, including:

- Lifting sockets in the center of the segments. These sockets can also double as grouting sockets for proof grouting.
- Bolts and sockets on the longitudinal joints to compress the gaskets and to support segments temporarily during construction.

- Dowels and sockets on the circumferential joints to maintain the compression of the gasket, maintain ring alignment, and support segments temporarily during construction.

Another design item is the location and size of the packers or contact surfaces between adjacent ring segments. The joint layout affects the bursting capacity of the lining and the induced moments in the segments because of the eccentricity of the compressive load. Similarly, the size, location, and jacking forces of the TBM thrust rams need to be accounted for in the design of the circumferential joints.

6.2 Gaskets

In the one-pass tunnel lining system, the primary water control elements are the continuous gaskets installed along the full perimeter of each segment. The gaskets are typically made from ethylene propylene diene monomer rubber (EPDM), which has excellent weather, UV, and ozone resistance. Gaskets on adjacent segments bear against each other, and the resulting compression seals the joints between segments against groundwater inflows. The gaskets are also a key part of the gas exclusion system.

According to the Preliminary Geotechnical Report (CH2M HILL, 2014), there is a low-to-moderate potential of encountering naturally occurring oil and/or gas within the Puente Formation or fault zones along the subterranean portion of the LRT Alternative. Therefore, the segmental lining is shown with double gaskets and cross-gaskets for the purposes of controlling potential gas infiltration and to enhance seismic performance. Similarly, the cross passage final linings and the connection between the cross passage ring beam and the segmental lining would be fully enclosed with a membrane for providing an essentially impermeable water and gas barrier. The gaskets and membranes used should be non-degradable (by hydrocarbons for example) in the expected chemical environment and should not leak during static and dynamic loading conditions.

Metro has undertaken studies to test and develop a gasket system with gas sealing properties. The results of those studies lead to the use of double-gasketed segments for the tunnels on the Metro Gold Line Eastside Extension project, which was constructed in an area where methane gas was known to be present, and had a “Gassy” Cal/OHSA underground classification (Choueiry et al, 2007). Additional levels of redundancy can be added to prevent water and gas leakage into the tunnel as well as to have the ability to repair the lining should leakage occur by adopting the following:

- The system should have double gaskets with the use of additional gasket bulkheads or cross-gaskets to further reduce the potential for “ring-to-ring” transmission of water.
- The tunnel ventilation system must be designed to dilute gases to safe levels and to exhaust smoke.
- Segments should have the ability to be easily repaired (using grout) should leakage occur between gaskets. The double-gasket and cross-gasket systems provide backing for confinement of grout.

The performance of the gaskets depends upon the amount of compression (and associated contact pressures) and the contact width between adjacent seals. The gaskets are compressed by either the advance of the TBM pushing on the circumferential joints, or by the segment erector for the longitudinal joints—with bolts, dowels, and external hoop thrusts acting to maintain the compression. The long-term performance of the gasket is primarily affected by the relaxation of the EPDM material, but also by the overall durability of the gasket material and its susceptibility to fires within the tunnels. Appropriate materials and lining systems should be chosen for the segmental lining gaskets and the cross passage final lining membranes; a testing program should be implemented to prove that these would perform adequately throughout the design life of the tunnel structures.

The above mentioned gaskets and membranes essentially render the tunnel structure watertight. However, a perfectly dry tunnel is not possible and some water infiltration is inevitable. The generally accepted standards for limiting water ingress in tunnels include:

- 1 gallon per minute per 1,000 feet of tunnel
- Local infiltration limited to 0.25 gallon per day for a 10 square foot area, and one drip per minute
- No water ingress that causes entry of soil particles

6.3 Recommendations for Segmental Lining

The preliminary design concepts for the bored tunnel lining are presented in the drawings of Attachment A. Each of the LRT Tunnels would have an inside diameter of 18 feet 10 inches and a one-pass double-gasketed segmental concrete lining with cross-gaskets in-between the double gaskets with an approximate thickness of 10 inches. The lining is composed of six segments plus a key segment, resulting in seven radial joints per ring. Based on preliminary analysis, 6,000 psi strength concrete with approximately 1% steel rebar reinforcement of the gross concrete area is required for the segmental lining. Note the amount of steel reinforcement indicated is for hoop reinforcing of the segments. Other reinforcing details, such as the longitudinal reinforcing in the segments, segment joints, and stirrup reinforcing, should be addressed in future design phases. The gasket system provides waterproofing and is a critical part of the gas exclusion system.

7 Preliminary Design Concepts for Cross Passages

Cross passages would have a two-pass lining system consisting of the initial shotcrete lining and the cast-in-place concrete final lining. A water/gas proof membrane would be installed in between the initial and final linings. Typical cross sections for the LRT cross passages are shown on drawings in Attachment A.

7.1 Initial Support Requirements

Cross passages would be excavated using the Sequential Excavation Method (refer to *Tunnel Excavation Methods* (JA, 2014c) for additional details). Hence, the design concepts for initial support for the cross passages is based on the principle of the SEM. Three support types have been developed to account for the range of anticipated ground conditions.

The key initial support and pre-support elements considered for the cross passage excavations include, but are not limited to, cement-grouted rock dowels (RD), fiber-reinforced shotcrete lining (FRS), spiles (SP), and fiberglass face dowels (FD). Ground improvement measures (as discussed in JA, 2014c) using chemical/permeation grouting or ground freezing for cross passages located in alluvium is required to potentially stabilize the ground and limit ground movements.

7.1.1 Ground Classes and Support Types

The three initial support types developed correspond to the anticipated ground conditions associated with the three ground classes defined in *Tunnel Ground Characterization* (JA, 2014b). Each ground class consists of rock mass types (RMTs) that are expected to exhibit similar potential ground behaviors upon excavation and are therefore expected to require similar support systems. Table 1 summarizes the three ground classes defined, associated RMTs, and anticipated general ground conditions. Each support type has a unique excavation sequence and initial support requirements to address the anticipated ground conditions.

7.1.2 Evaluation of Excavation Sequence and Initial Support Measures

Cross passage excavation sequence and initial support concepts are based on the anticipated ground class at each cross passage. In addition, the project-specific requirements in terms of limits of surface settlements induced by tunnel excavations, the control of groundwater inflows (e.g., limits of impact to regional groundwater resources), etc., are also considered in the evaluation of these measures.

For this stage of design, applicable initial support concepts for the cross passages are based primarily on relevant past experience with cross passages of similar size in similar ground conditions and seismic settings for projects such as the Regional Connector Transit Corridor and Metro Gold Line Eastside Extension projects. The Metro Gold Line Eastside Extension cross passages have been constructed, and detailed designs have been completed for the Regional Connector cross passages. As such, the cross passage designs from these two projects are specifically applicable to the cross passages of the LRT Alternative.

7.1.3 Recommendations for Excavation Sequence and Initial Support

For the cross passage excavations, three support types have been identified to address the anticipated ground conditions associated with three of the ground classes. Approximate locations of the cross passages are indicated in Figure 1 and Table 1. The excavation sequence and initial support requirements for each of the three support types are described below. Details of the three support types are shown on the drawings in Attachment A.

Support Type 1

Support Type 1 is required for excavation in Ground Class 1 (RMT Tt-1) ground conditions, and is expected to be applicable to four of the cross passages. The cross passage would be excavated full face with a maximum round length of 4.0 feet. The support type consists of 8-foot-long fully grouted rock dowels with a spacing of 4.0 feet on center and 8-inch-thick fiber-reinforced shotcrete. Face support consisting of 2-inch-thick face sealing shotcrete would be installed, if required based on the ground conditions encountered, to prevent instability of the advancing face or support potential unstable wedges encountered in the face.

Support Type 2

Support Type 2 is required for excavation in Ground Class 2 (RMTs Tp-2, Tt-2, and mixed face conditions) ground conditions, and is expected to be applicable to six of the cross passages. Excavation consists of a top heading and bench configuration. The top heading would be excavated with a maximum round length of 3.5 feet. The minimum lag between the top heading face and the bench face is 7.0 feet. The bench/invert excavation would be excavated with a design maximum round length of 7.0 feet. The support type consists of 10-inch-thick fiber-reinforced shotcrete. Pre-support consisting of 15-foot-long rebar or pipe spiles with a spacing of 12 inches on center would be installed above the tunnel crown for every other advance as required. Face support consisting of 15-foot-long face fiberglass dowels with a spacing of 4 feet on center and/or 2-inch-thick face sealing shotcrete would also be installed as required to maintain face stability.

Support Type 3

Support Type 3 is required for excavation in Ground Class 3 (Alluvium) ground conditions, and is expected to be applicable to 16 of the cross passages. Excavation consists of a top heading and bench configuration. Ground improvement using permeation/chemical grouting or ground freezing would be required prior to cross passage excavation. After the treated ground reaches a minimum compressive strength of 400 psi, the top heading would be excavated with a maximum round length of 3.5 feet. The minimum lag between the top heading face and the bench face is 7.0 feet. The bench/invert excavation would be excavated with a design maximum round length of 7.0 feet. The support type consists of 10-inch-thick fiber-reinforced shotcrete. Pre-support consisting of 15-foot-long pipe spiles or canopy with a spacing of 12 inches on center would be installed above the tunnel crown for every other advance as required. Face support consisting of 12-foot-long face fiberglass dowels with a spacing of 4 feet on center and/or 2-inch-thick face sealing shotcrete would also be installed as required to maintain face stability.

Temporary Support at Breakout

In order to construct the cross passages after the twin-bore tunnels are constructed, it would be necessary to break out of the segmental lining at the cross passage locations. Prior to breaking out of the segmental lining, the lining has to be temporarily supported in order to allow the thrust in the lining of the running tunnel to be redistributed around the opening cut into the segments. Several feasible methods may be considered for the temporary support system at the breakout, and for the purposes of these conceptual evaluations, the approach presented on the drawings in Attachment A is recommended. Prior to saw-cutting the segments for the breakout, shear keys consisting of precast concrete blocks are installed as shown on the drawings. A notch, which is slightly larger than the shear key, is cut into the segments at the joints so that the shear key straddles the joint between two segment rings. The key is then grouted in place with high-strength, non-shrink grout. These keys, together with the longitudinal connectors of the segment joint, transfer the thrust load from the segment that is cut into the adjacent segments. Steel struts would be used as shown on the drawing in order to enable the adjacent segments to carry the additional loads. The shear keys and the segments may be designed to permanently transfer the load from the cut segment into the adjacent segments. Alternatively, the keys may be temporary, and the collar beam (see Section 7.2) can be used to permanently support the thrust in the cut segments. This system with the use of shear keys and steel strut support was used on the Port of Miami project. The advantage of this support system for the breakout is that it can be installed (and removed for temporary system) relatively easily within a short period of time.

7.2 Design Concepts for Final Lining

Cross passage final lining includes two different sections. One is the standard section which covers the majority of the cross passage, and the other is the collar section (ring beam), which is provided at the intersection between the cross passage and the bored tunnel. In both sections, the cross passage structure consists of the initial shotcrete lining and cast-in-place final concrete lining. The standard and collar cross sections for the LRT alternative cross passages are shown on the drawings in Attachment A.

Similarly to the initial support design concepts, final lining requirements for the LRT cross passages are based on relevant experience on similar projects including the Regional Connector Transit Corridor and Metro Gold Line Eastside Extension projects.

The cross passage final lining includes a standard section and a collar section with elliptical-shaped cross sections. The standard section has inside dimensions of approximately 10 feet 8 inches wide and 10 feet 3 inches high, while the collar section is approximately 9 feet 4 inches wide and 9 feet 7 inches high. The standard section final lining consists of the initial shotcrete lining, a water/gas proof membrane, and a 10-inch-thick reinforced cast-in-place concrete lining. The collar section final lining consists of an initial shotcrete lining, a water/gas proofing membrane, and an 18-inch-thick reinforced cast-in-place concrete ring beam. The water/gas proofing member would be a hydrocarbon resistant HDPE membrane as described in Section 6.

8 Fire Resistance

At the preliminary design phase, no analysis has been performed on the tunnel linings with respect to fire resistance. The appropriate level of protection for the linings should be determined in future studies based on Metro-specified fire intensity and period of time without loss of structural integrity. Depending on the results of the studies, the following measures may be considered to improve the performance of the final lining exposed to a fire:

- Increase the concrete cover over the reinforcing steel to restrict the temperature rise in the reinforcing steel.
- Specify the use of aggregate that is thermally stable.
- Use micro polypropylene fibers in the concrete mix to reduce concrete spalling.

Prevention of spalling helps improve the overall structural performance of a concrete section during a fire because of the insulation protection the intact concrete cover offers to the reinforcing. The use of aggregates that are less prone to thermal expansion and splitting at high temperatures has been shown to improve the performance of concrete subject to high temperatures (International Federation for Structural Concrete, *fib*, 2007). The use of micro polypropylene (PP) fibers has been shown to effectively reduce explosive spalling (Tatnall, 2002) by limiting the development of high vapor pressures within the concrete. Appropriate fire resistance measures will have to be developed during the future design phases after the design fire conditions are better understood.

9 Recommendations for Future Design Evaluations

The tunnel lining design concepts presented in this TM for the LRT Alternative are considered preliminary. The concepts were developed with limited geotechnical data along the nearly 4.4-mile-long bored tunnel alignment. Additional geotechnical investigations with more closely spaced borings would be necessary to better characterize the geologic and groundwater conditions along the tunnel alignment, including the fault zones. Since the preliminary design concepts for the LRT Alternative are based primarily on relevant experience (e.g., Regional Connector Transit Corridor and Metro Gold Line Eastside Extension), specific design evaluations will have to be completed in the next design stage, addressing the design criteria, analysis methods, and procedures discussed in this TM, if this alternative is selected.

Other design items requiring further evaluation include, but are not limited to the following:

- Interfaces between the segmental lining and final lining of the station at either end of each station. The detailing for these interfaces is especially important for seismic loading. Additionally, continuity of waterproofing across the joints would require special details and procedures to limit or prevent inflows at the interfaces.
- Evaluation of the method(s) and requirements for ground treatment at cross passage locations based on additional information from future geotechnical investigations.
- Requirements for fire resistance determined based on Metro-specified fire intensity and period of time without loss of structural integrity.
- Requirements for corrosion protection from sources such as the ground, groundwater, and stray currents.

10 Limitations

The preliminary design concepts presented in this TM are based on limited geotechnical data. A significant amount of additional geotechnical investigations and data gathering studies would be required in order to advance these design concepts to a complete preliminary design level.

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12 Revision Log

Revision 0	November 21, 2013	Internal Review
Revision 1	December 16, 2013	Metro/Caltrans Review
Revision 2	May, 27 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Table 1: Locations and Characteristics of Cross Passages for LRT Alternative

CP No.	Stationing	Ground Cover (ft)	Soil Overburden Depth (ft)	Ground Surface/XP Crown Elevation (ft)	Ground Class/Ground Condition	Groundwater Table Elevation/ Head at Crown Elevation (ft)	Support Type
1	181+45.00	55	55	425/370	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	390/20	ST2
2	188+90.43	60	60	440/380	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	400/20	ST2
3	196+15.43	55	85	445/390	Ground Class 3: In Soil; Alluvium	400/10	ST3
4	203+40.72	60	90	460/400	Ground Class 3: In Soil; Alluvium	410/10	ST3
5	210+65.70	60	80	460/400	Ground Class 3: In Soil; Alluvium	410/10	ST3
6	217+90.68	60	80	470/410	Ground Class 3: In Soil; Alluvium	420/10	ST3
7	232+53.99	55	55	475/420	Ground Class 3: In Soil; Alluvium or Mixed Face; Alluvium over Fernando Formation	430/10	ST3
8	240+04.04	60	50	490/430	Ground Class 2: In Rock; Puente Formation (Tp-2)	440*	ST2
9	247+54.08	60	60	500/440	Ground Class 2: In Rock or Mixed Face; Alluvium over Puente Formation (Tp-2)	440*	ST2
10	255+04.13	55	65	515/460	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Puente Formation (Tp-2)	450*	ST3
11	262+54.17	55	60	525/470	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Puente Formation (Tp-2)	470*	ST3
12	270+04.21	60	65	540/480	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Topanga Formation (Tt-2)	480*	ST3
13	277+54.25	60	60	560/500	Ground Class 2: In Rock; Topanga Formation (Tt-2)	500*	ST2
14	285+03.84	50	0	560/510	Ground Class 1: In Rock; Topanga Formation (Tt-1)	560*	ST1
15	292+53.83	75	0	600/525	Ground Class 1: In Rock; Topanga Formation (Tt-1)	600*	ST1
16	308+53.99	60	220	600/540	Ground Class 3: In Soil; Alluvium	490	ST3
17	316+04.03	60	300	610/550	Ground Class 3: In Soil; Alluvium	500	ST3
18	323+54.06	60	220	620/560	Ground Class 3: In Soil; Alluvium	520	ST3
19	331+04.09	55	110	635/580	Ground Class 3: In Soil; Alluvium	550	ST3
20	349+04.06	80	230	690/610	Ground Class 3: In Soil; Alluvium	540	ST3
21	356+54.09	80	50	720/640	Ground Class 2: In Rock; Topanga Formation (Tt-2)	690/50	ST2
22	364+04.37	80	0	750/670	Ground Class 1: In Rock; Topanga Formation (Tt-1)	750*	ST1
23	371+54.40	80	0	760/680	Ground Class 1: In Rock; Topanga Formation (Tt-1)	760*	ST1
24	379+04.94	60	70	750/690	Ground Class 3: In Soil; Alluvium or Mixed face; Alluvium over Topanga Formation (Tt-2)	690*	ST3
25	386+54.44	60	180	760/700	Ground Class 3: In Soil; Alluvium	650	ST3
26	394+04.44	55	180	760/705	Ground Class 3: In Soil; Alluvium	650	ST3

*Approximated groundwater elevation was interpolated

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

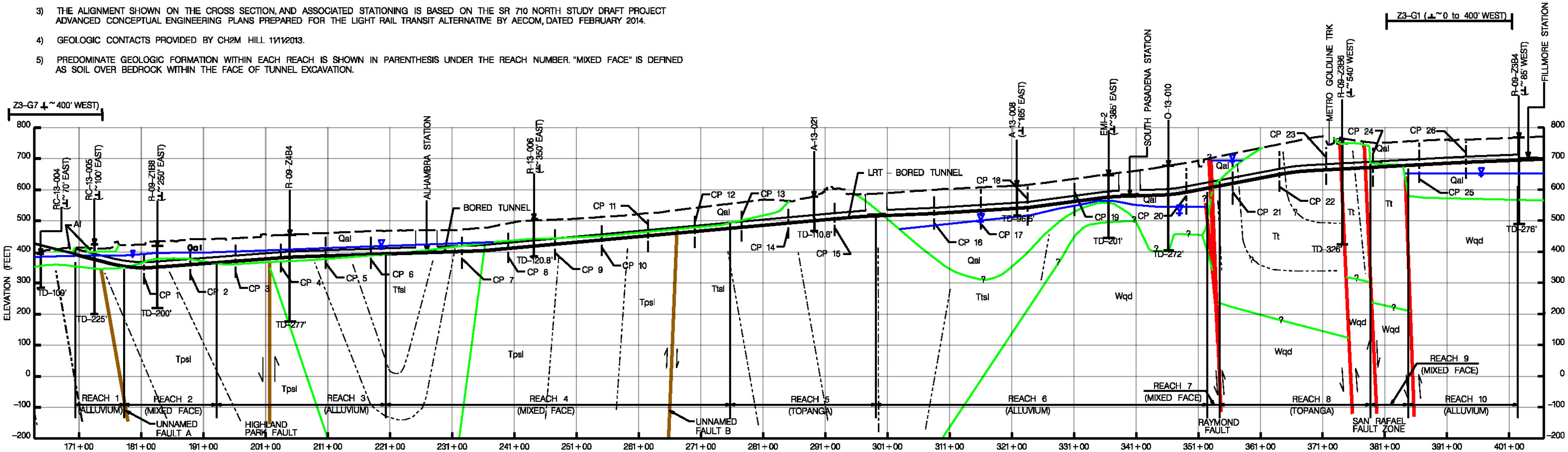
2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT ADVANCED CONCEPTUAL ENGINEERING PLANS PREPARED FOR THE LIGHT RAIL TRANSIT ALTERNATIVE BY AECOM, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

5) PREDOMINATE GEOLOGIC FORMATION WITHIN EACH REACH IS SHOWN IN PARENTHESIS UNDER THE REACH NUMBER. "MIXED FACE" IS DEFINED AS SOIL OVER BEDROCK WITHIN THE FACE OF TUNNEL EXCAVATION.

Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative



LEGEND

UNITS

- Af ARTIFICIAL FILL
- Qal ALLUVIAL SOIL
- Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER
- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
- Tpsl PUENTE FORMATION, SILTSTONE MEMBER
- Tpss PUENTE FORMATION, SANDSTONE MEMBER
- Tt TOPANGA FORMATION, UNDIFFERENTIATED
- Ttss TOPANGA FORMATION, SANDSTONE MEMBER
- Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER
- Ttsl TOPANGA FORMATION, SILTSTONE MEMBER
- Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
- INTRAFORMATIONAL CONTACT
- GENERALIZED BEDDING
- ESTIMATED TOP OF GROUNDWATER TABLE
- Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION
- A-13-020 TD-149.8' GEOTECHNICAL BORHOLE WITH TOTAL DEPTH AND PROJECTION:
  - A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
  - R-09-Z1B8 – CH2M HILL, 2010
  - NM-B3 – NINYO AND MOORE, 1999
  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

LRT GEOLOGIC PROFILE

Figure 1: LRT Tunnel Geologic Profile

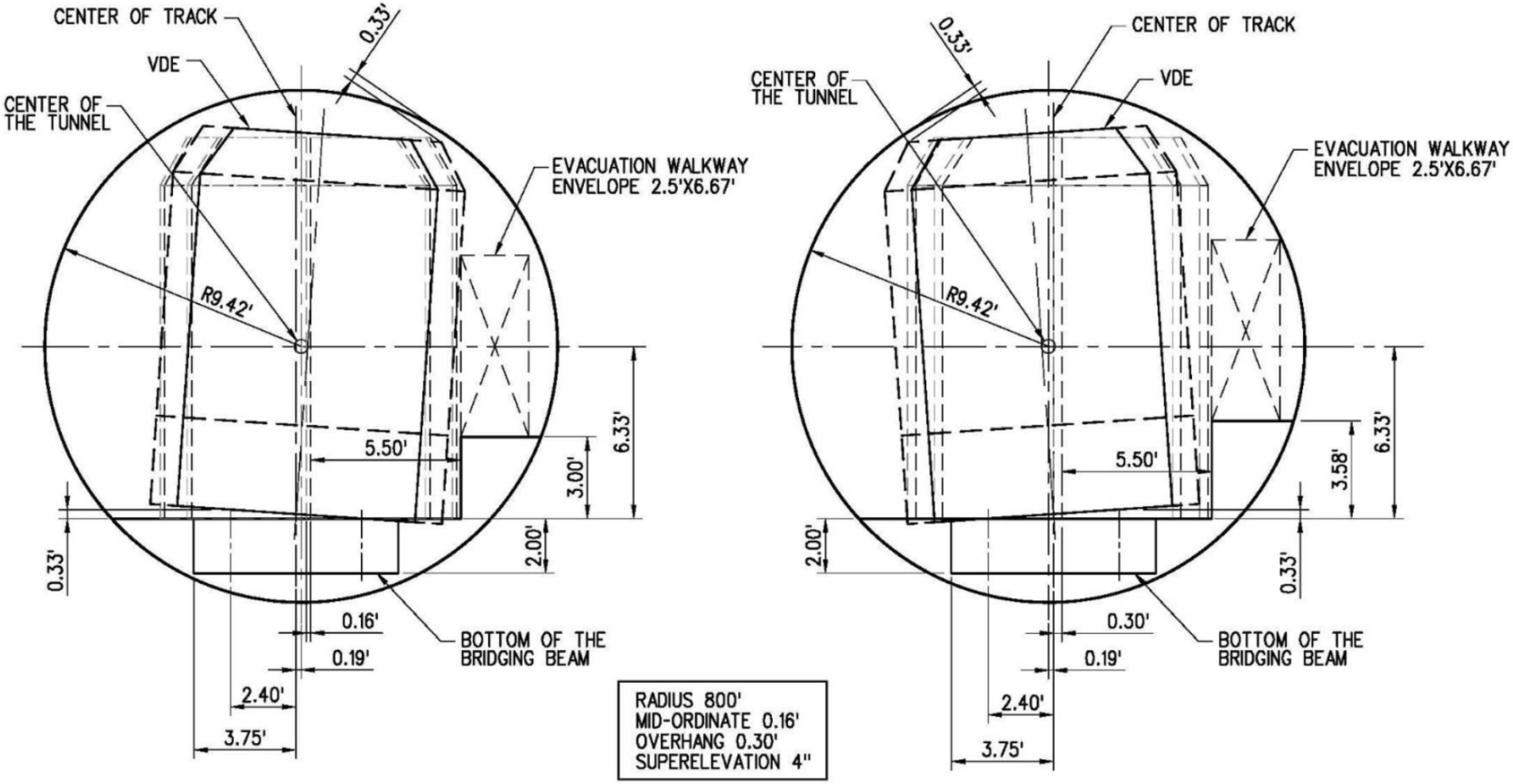
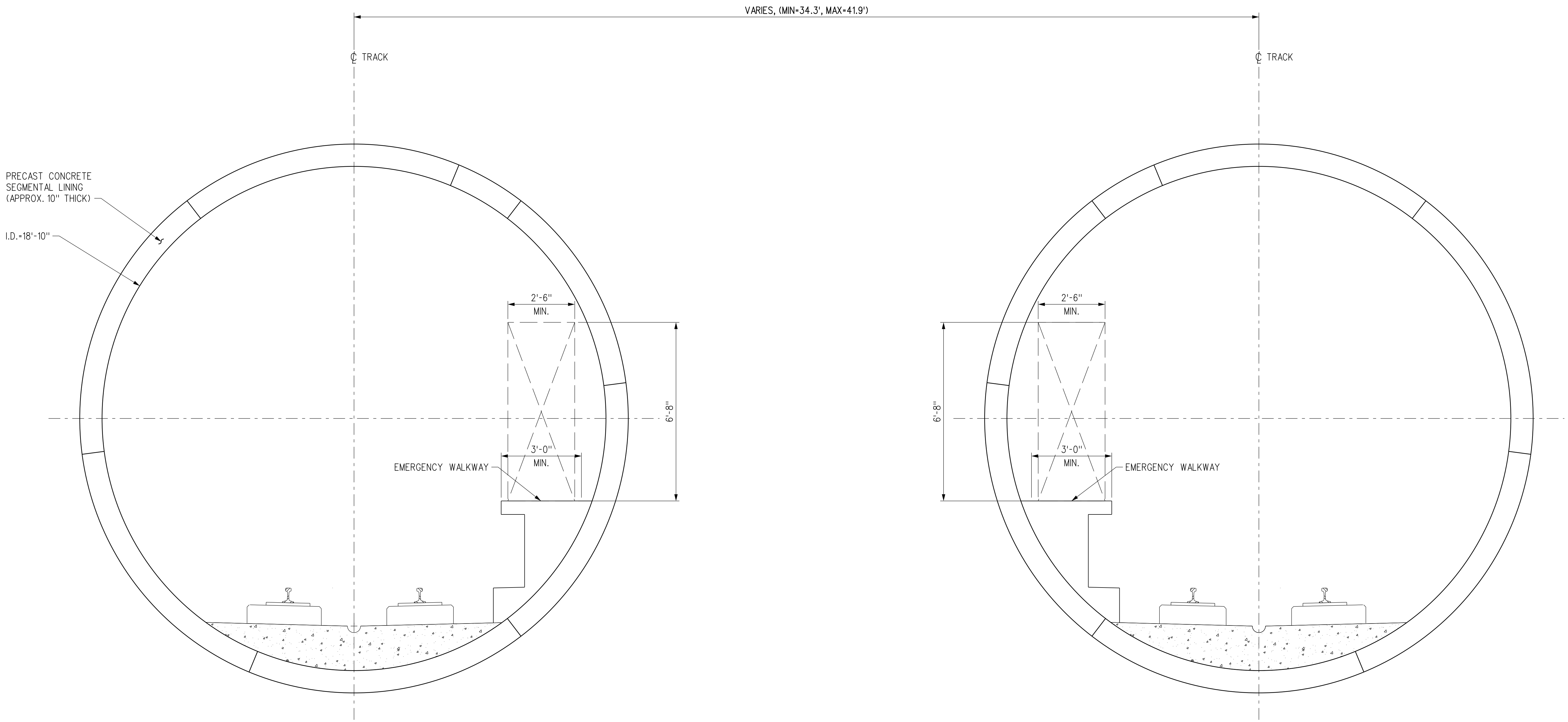


Figure 2: LRT Clearance Diagram (Metro, 2013)

Attachment A

Drawings



TYPICAL SECTION – BORED TUNNELS

NOTES:

1. METRO RAIL DESIGN CRITERIA AND NFPA 130 USED AS REFERENCE FOR TYPICAL CROSS SECTIONS AND CLEARANCES.

2' 1' 0 2' 4'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

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DESIGNED BY M. TORSIELLO
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro

LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TYPICAL BORED TUNNEL SECTION
SHEET 1 OF 1

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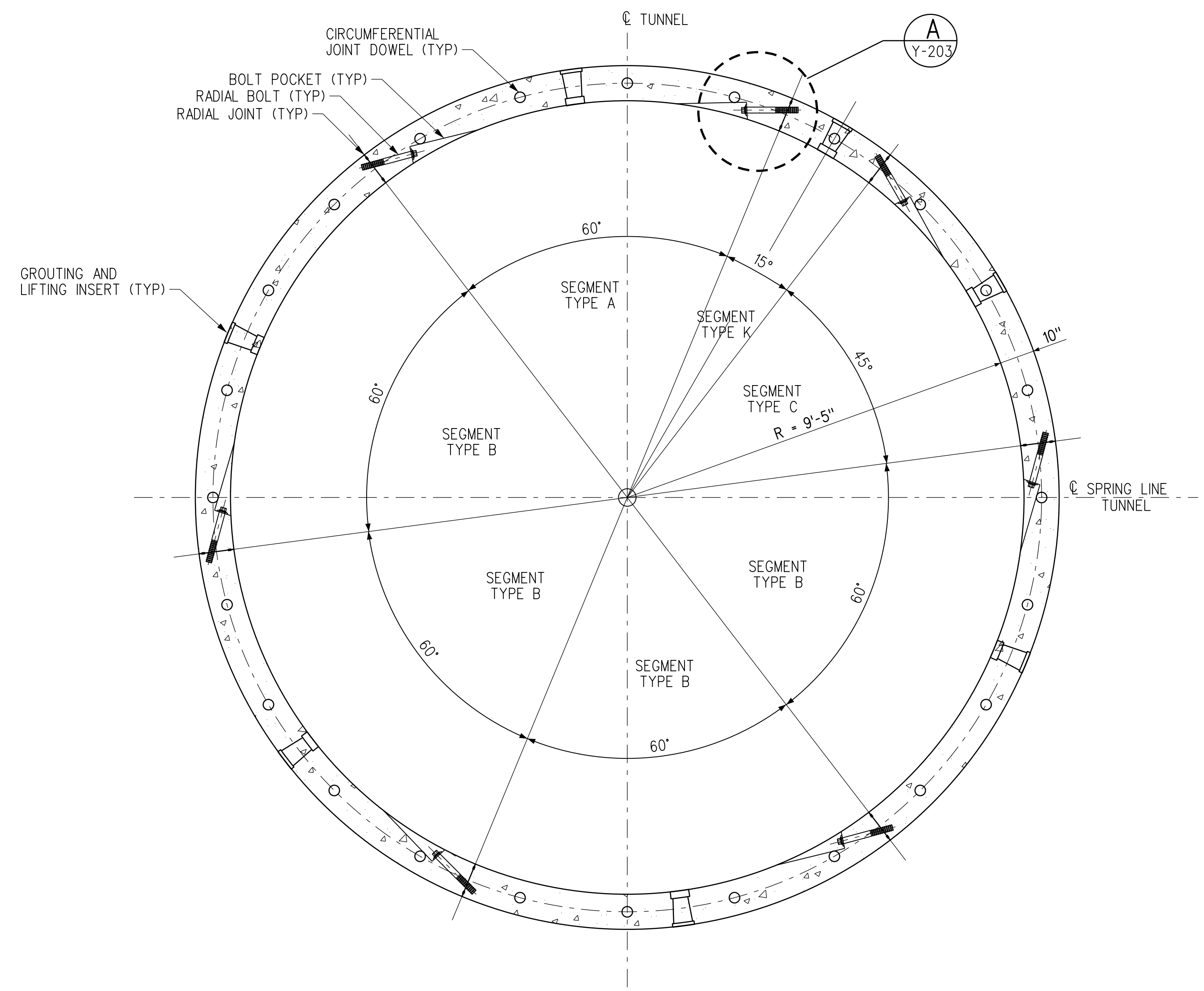

TYPICAL SECTION
SEGMENTAL LINING

A
Y-201

NOTES:

1. TYPICAL SECTION APPLIES TO BOTH NORTHBOUND AND SOUTHBOUND TUNNELS.
2. SEE DWG Y-202 FOR SEGMENT LAYOUT DETAILS.

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PRELIMINARY

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CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TUNNEL SEGMENTAL LINING
TYPICAL SECTION

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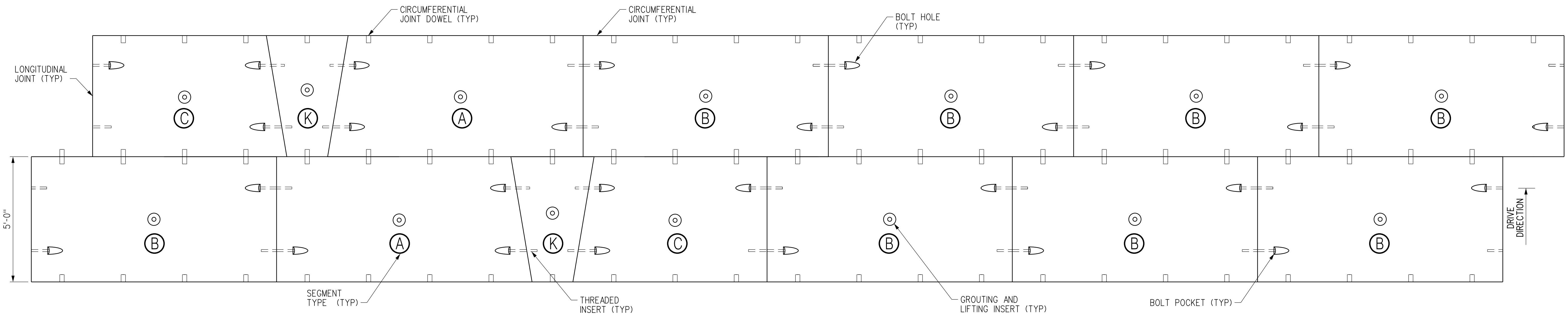

DEVELOPED PLAN ON INTRADOS

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DESIGNED BY M. HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TUNNEL SEGMENTAL LINING
DETAILS

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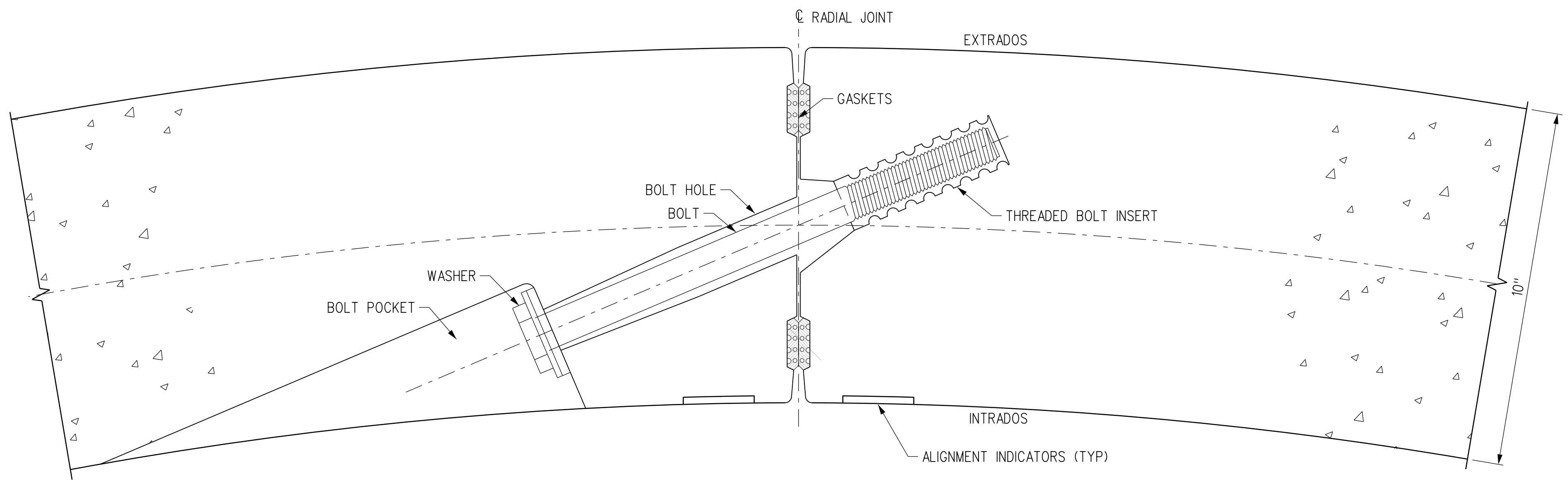

TYPICAL RADIAL JOINT

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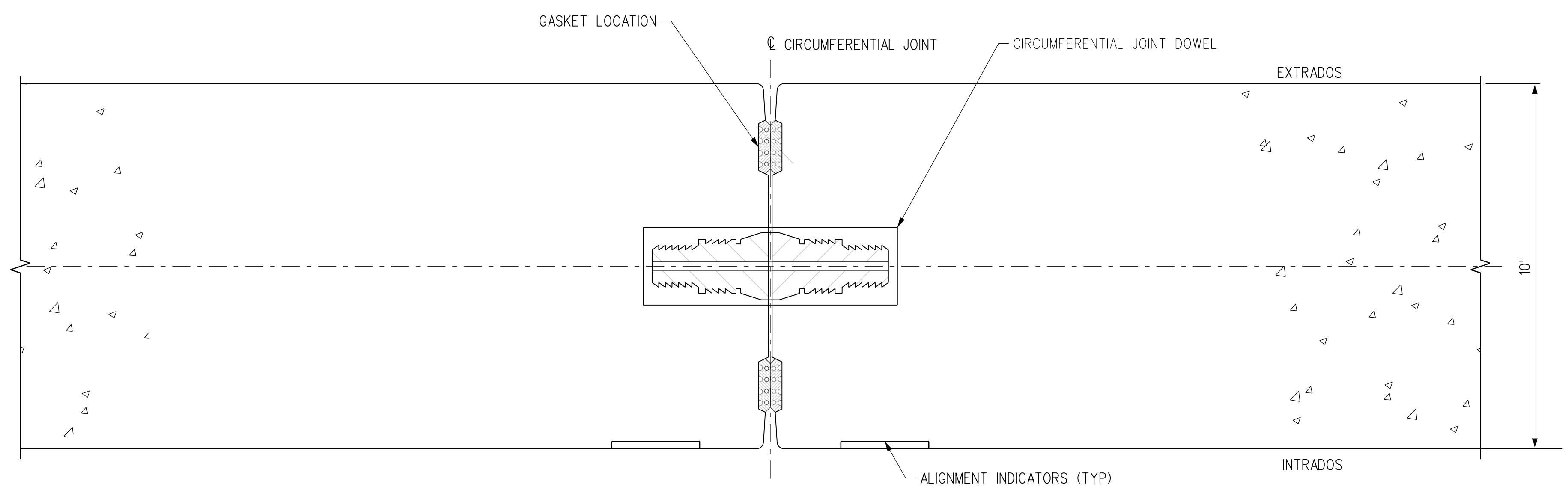
Y-201



TYPICAL CIRCUMFERENTIAL JOINT

B
Y-201

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

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DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TUNNEL SEGMENTAL LINING
DETAILS
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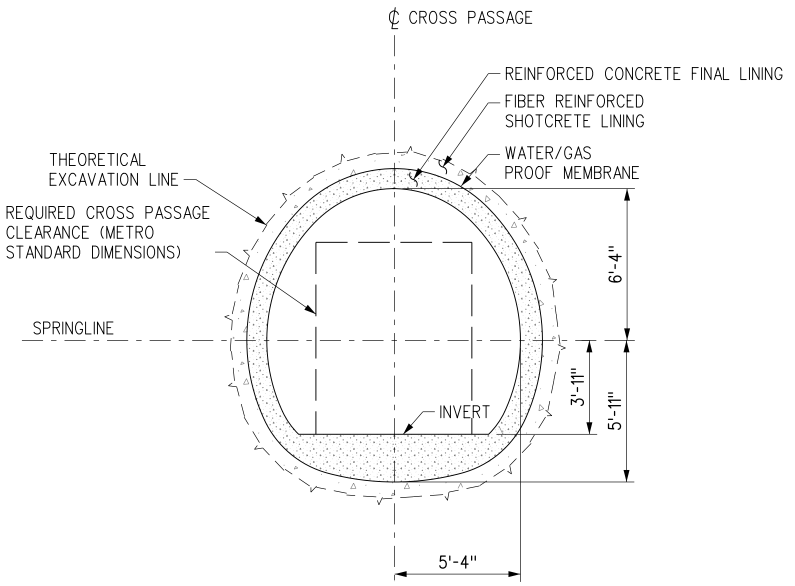

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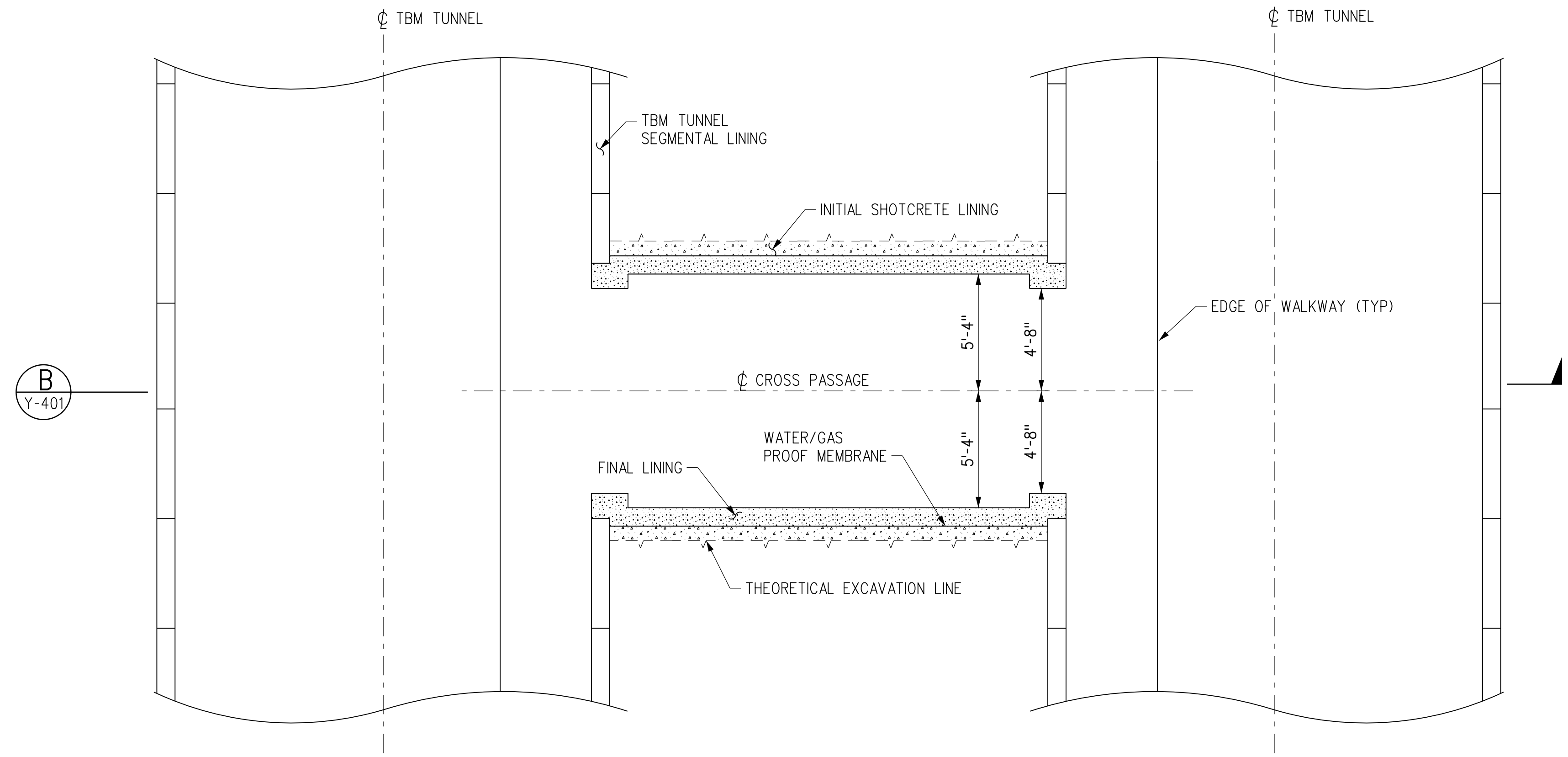

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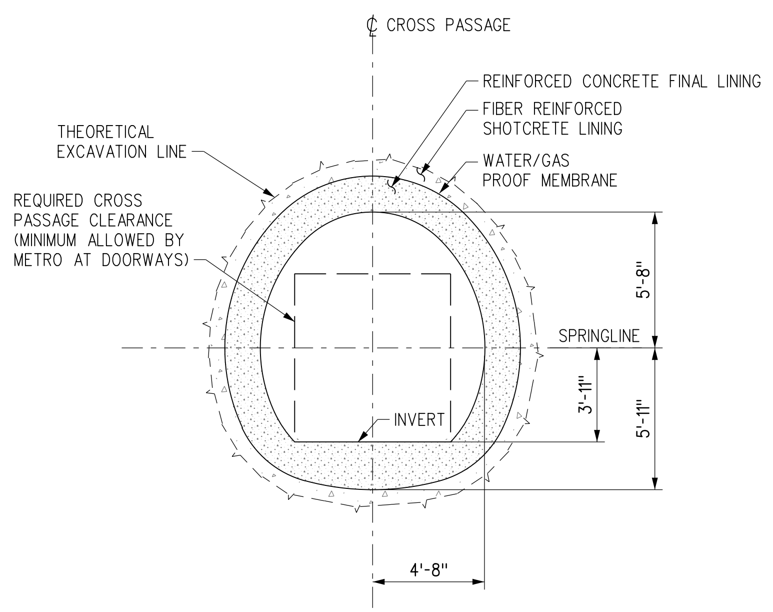

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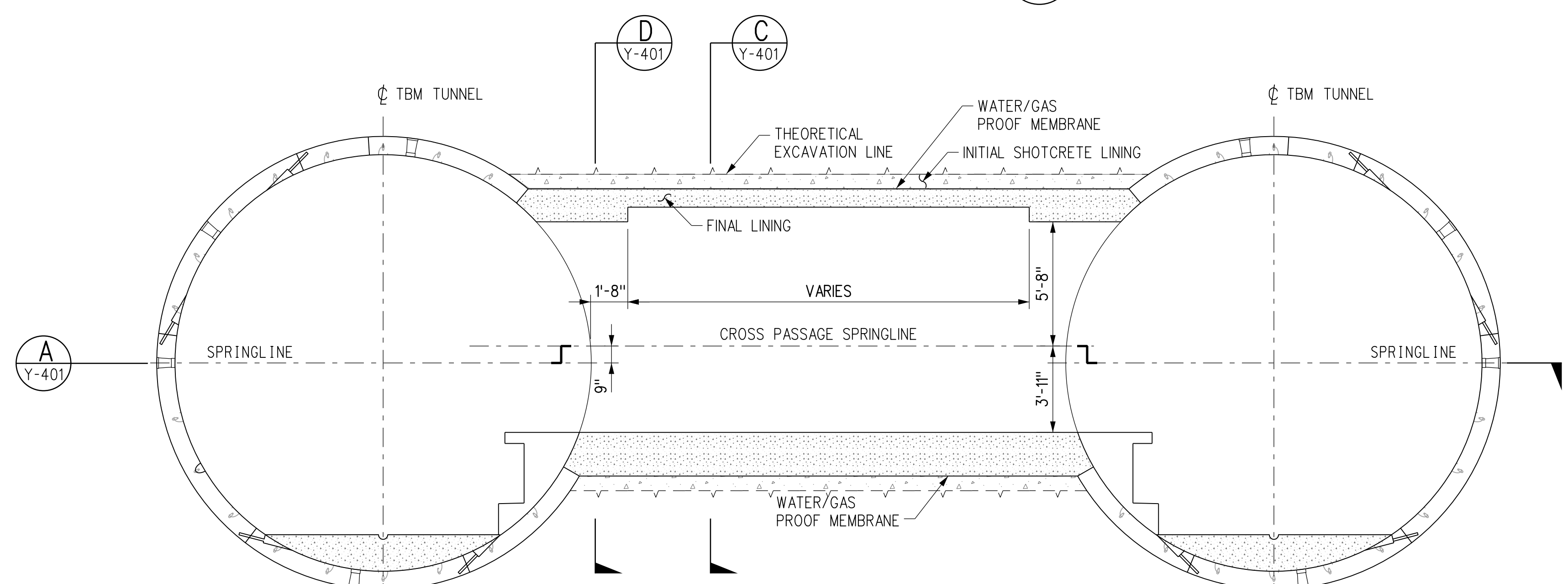

SECTION B Y-401

NOTES:

1. METRO RAIL DESIGN CRITERIA USED AS REFERENCE FOR CROSS PASSAGE CLEARANCE REQUIREMENTS.

4' 2' 0 4' 8'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAKES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION
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DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro

LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE
PLANS AND SECTIONS

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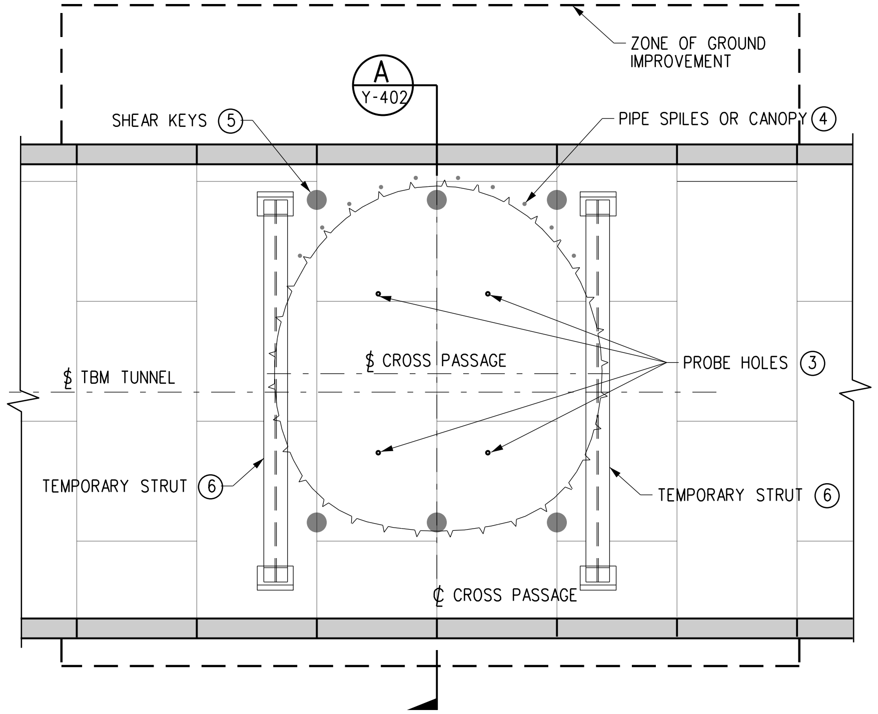
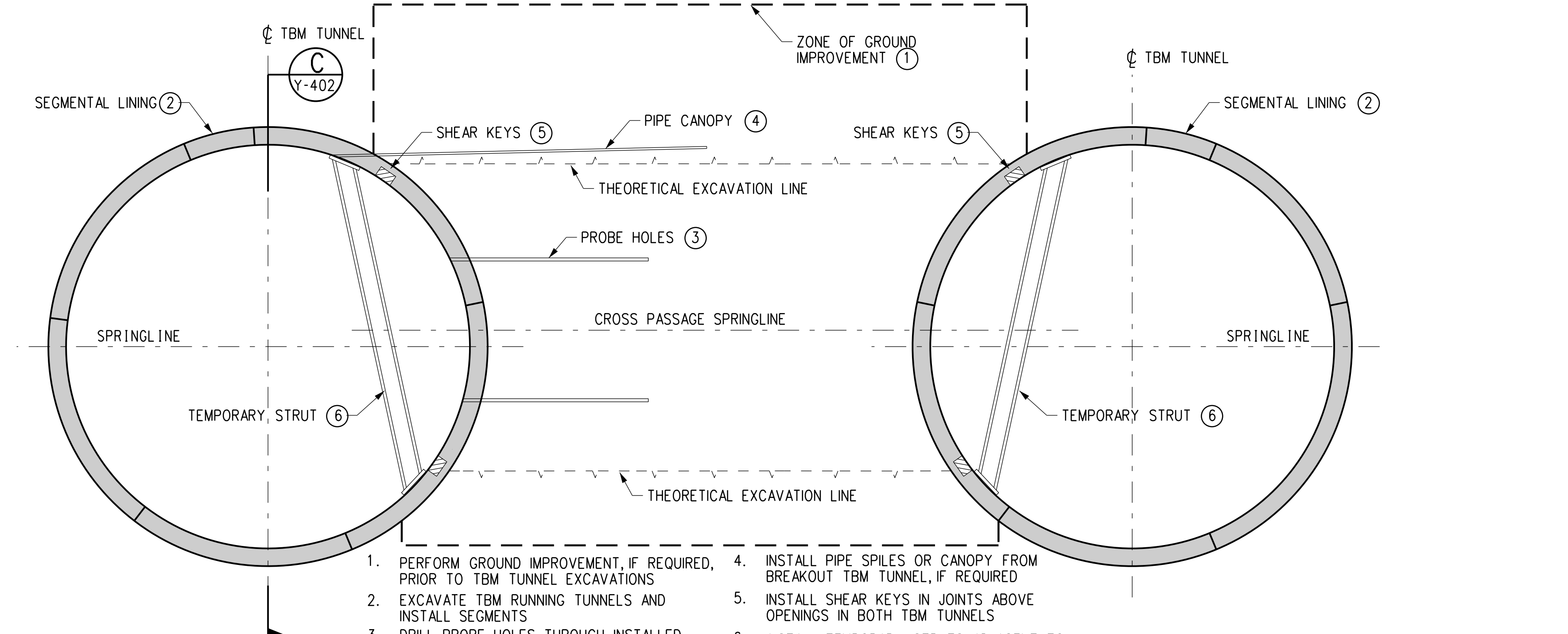


CROSS PASSAGE BREAKOUT ELEVATION

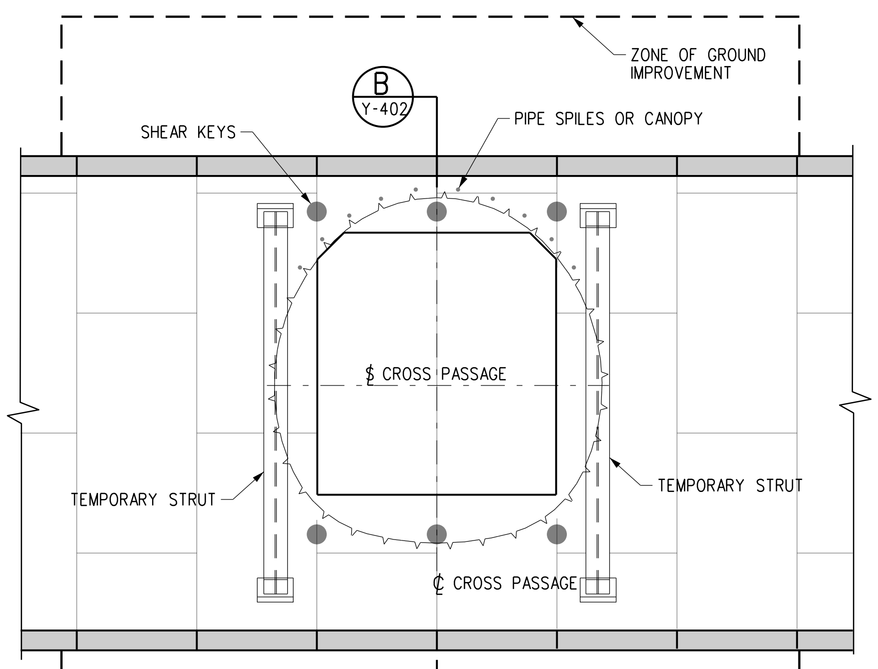
C Y-402



1. PERFORM GROUND IMPROVEMENT, IF REQUIRED, PRIOR TO TBM TUNNEL EXCAVATIONS
2. EXCAVATE TBM RUNNING TUNNELS AND INSTALL SEGMENTS
3. DRILL PROBE HOLES THROUGH INSTALLED SEGMENTS IN BREAKOUT TBM TUNNEL AND DETERMINE/VERIFY CONDITIONS OF TREATED GROUND
4. INSTALL PIPE SPILES OR CANOPY FROM BREAKOUT TBM TUNNEL, IF REQUIRED
5. INSTALL SHEAR KEYS IN JOINTS ABOVE OPENINGS IN BOTH TBM TUNNELS
6. INSTALL TEMPORARY STRUTS ADJACENT TO CROSS PASSAGE OPENINGS IN BOTH TBM TUNNELS AND PRELOAD STRUTS USING HYDRAULIC JACKS (NOT SHOWN)

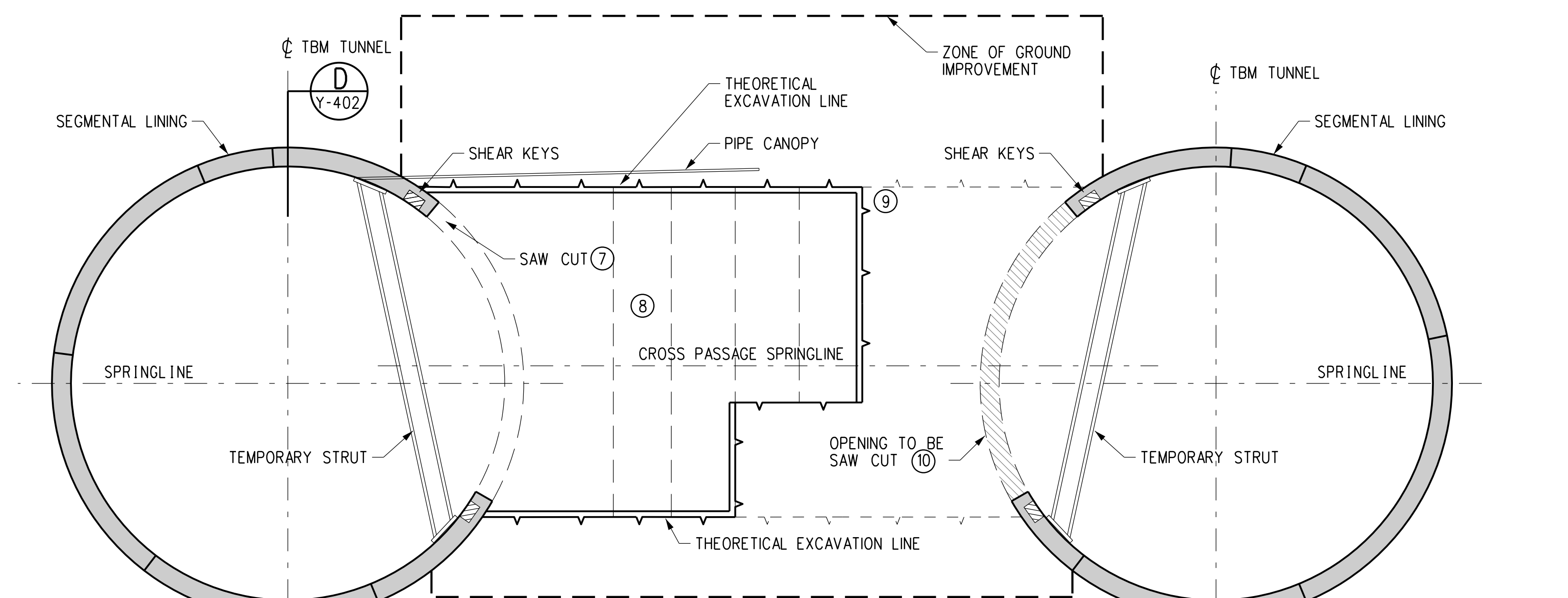
CROSS PASSAGE LONGITUDINAL SECTION

A Y-402



CROSS PASSAGE LONGITUDINAL SECTION

D Y-402



7. SAW CUT AND REMOVE SEGMENTS TO FORM OPENING IN BREAKOUT TBM TUNNEL
8. EXCAVATE CROSS PASSAGE AND INSTALL INITIAL SUPPORT IN STAGES IN ACCORDANCE WITH REQUIREMENTS SHOWN ON DRAWING Y-405
9. COMPLETE CROSS PASSAGE EXCAVATION
10. SAW CUT AND REMOVE SEGMENTS TO FORM OPENING ON OPPOSITE TBM TUNNEL

CROSS PASSAGE BREAKOUT ELEVATION

B Y-402

NOTES:

1. FOR INITIAL SUPPORT OF CROSS PASSAGE SEE DRAWING Y-405
2. FOR FINAL LINING OF CROSS PASSAGE SEE DRAWING Y-401

4' 2' 0 4' 8'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY Y. SUN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE BREAKOUT AND CONSTRUCTION SEQUENCE
SHEET 1 OF 2

CONTRACT NO
DRAWING NO Y-402
REV
SCALE 1/4" = 1'-0"
SHEET NO

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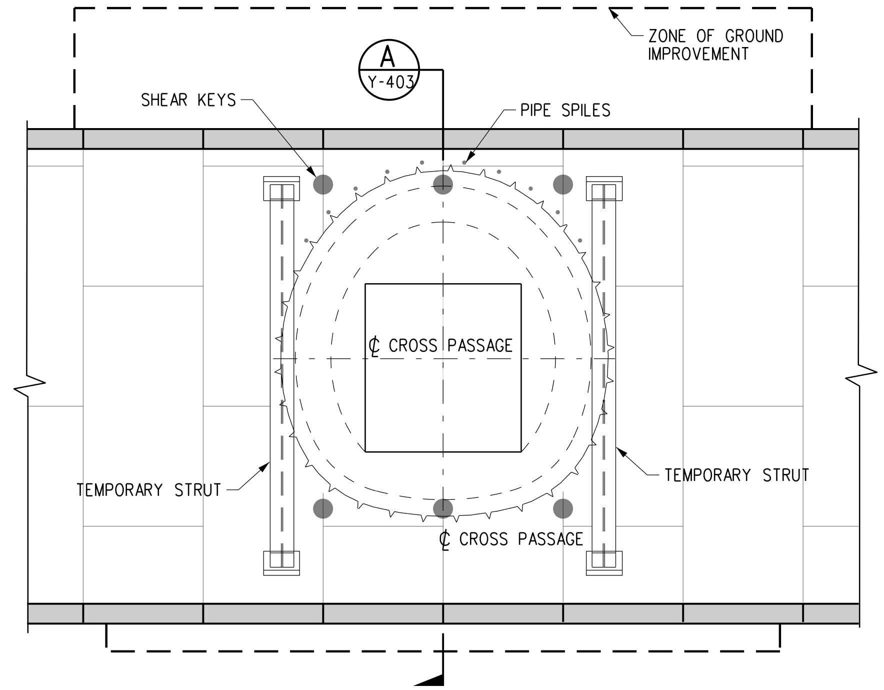
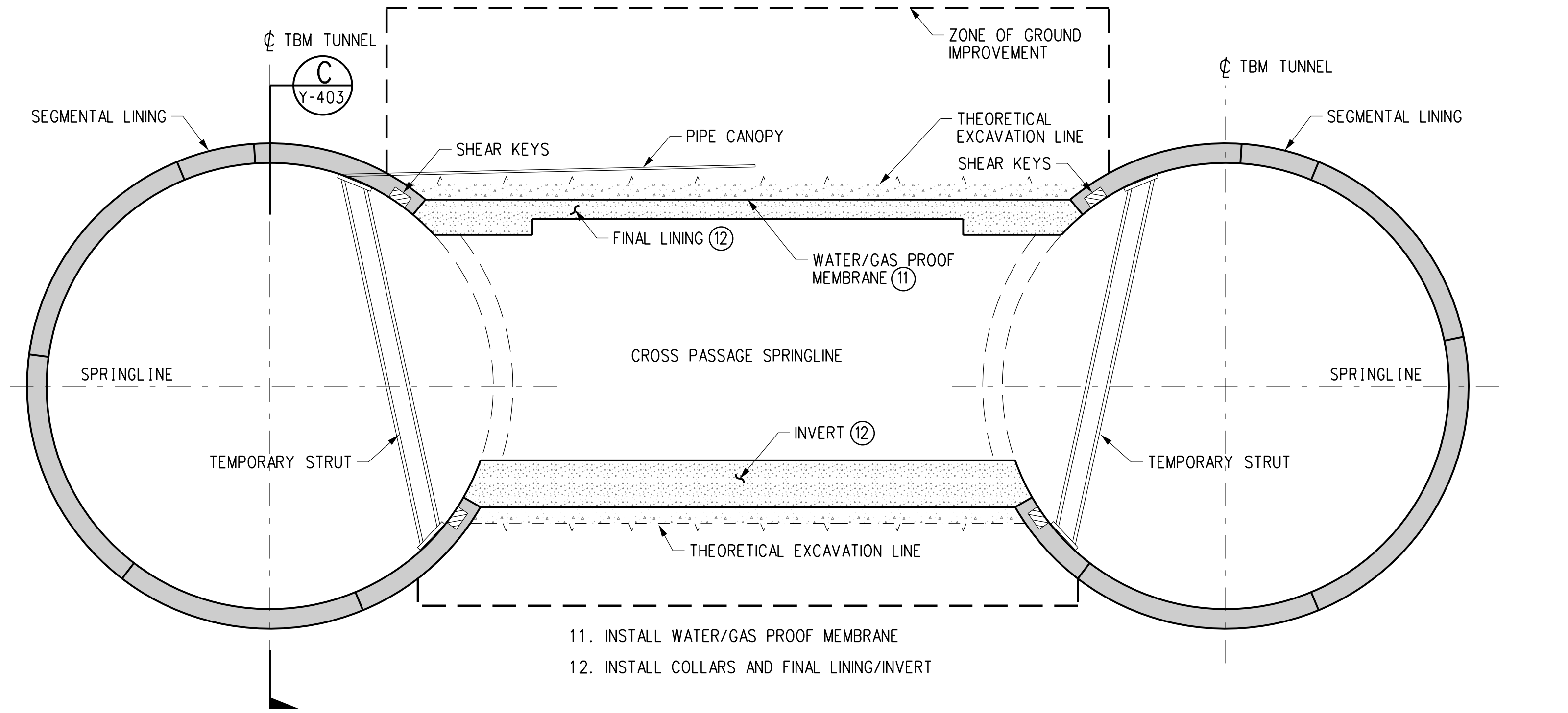
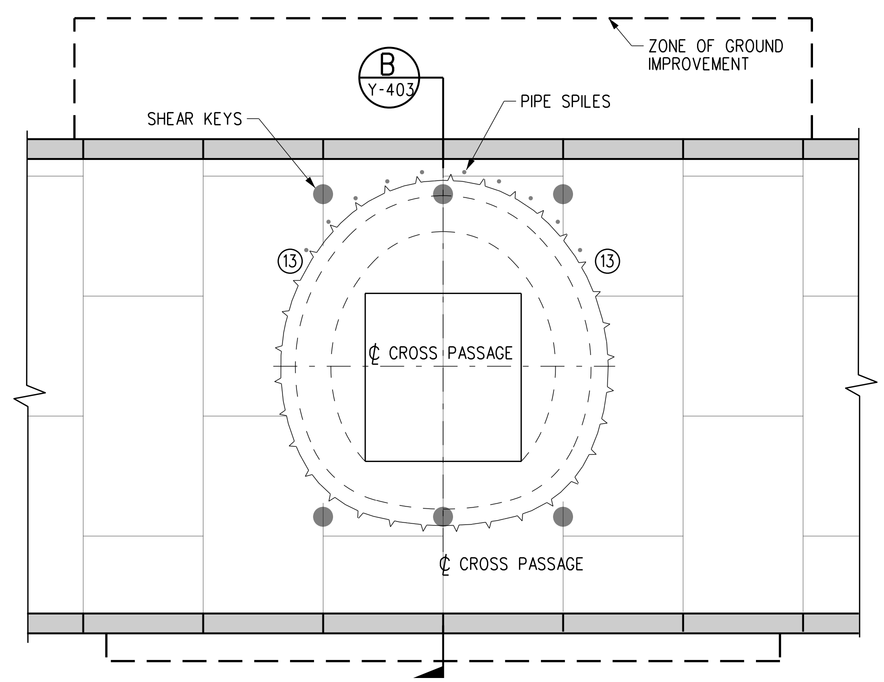
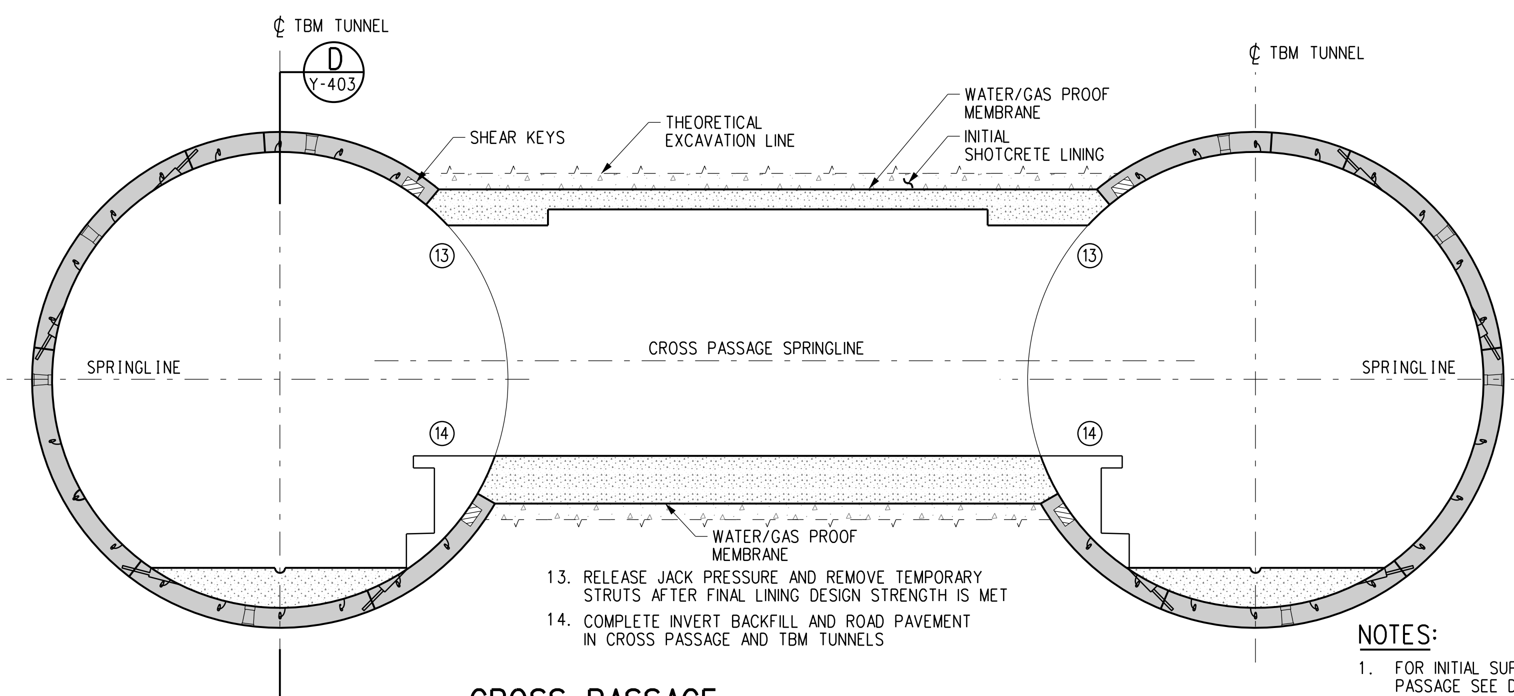




CROSS PASSAGE
OPENING ELEVATION
C
Y-403

CROSS PASSAGE
LONGITUDINAL SECTION
A
Y-403





CROSS PASSAGE
OPENING ELEVATION
D
Y-403

CROSS PASSAGE
LONGITUDINAL SECTION
B
Y-403

NOTES:

1. FOR INITIAL SUPPORT OF CROSS PASSAGE SEE DRAWING Y-405
2. FOR FINAL LINING OF CROSS PASSAGE SEE DRAWING Y-401

4' 2' 0 4' 8'

PRELIMINARY

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REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

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DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE BREAKOUT
AND CONSTRUCTION SEQUENCE
SHEET 2 OF 2

CONTRACT NO
DRAWING NO Y-403
REV
SCALE 1/4" = 1'-0"
SHEET NO

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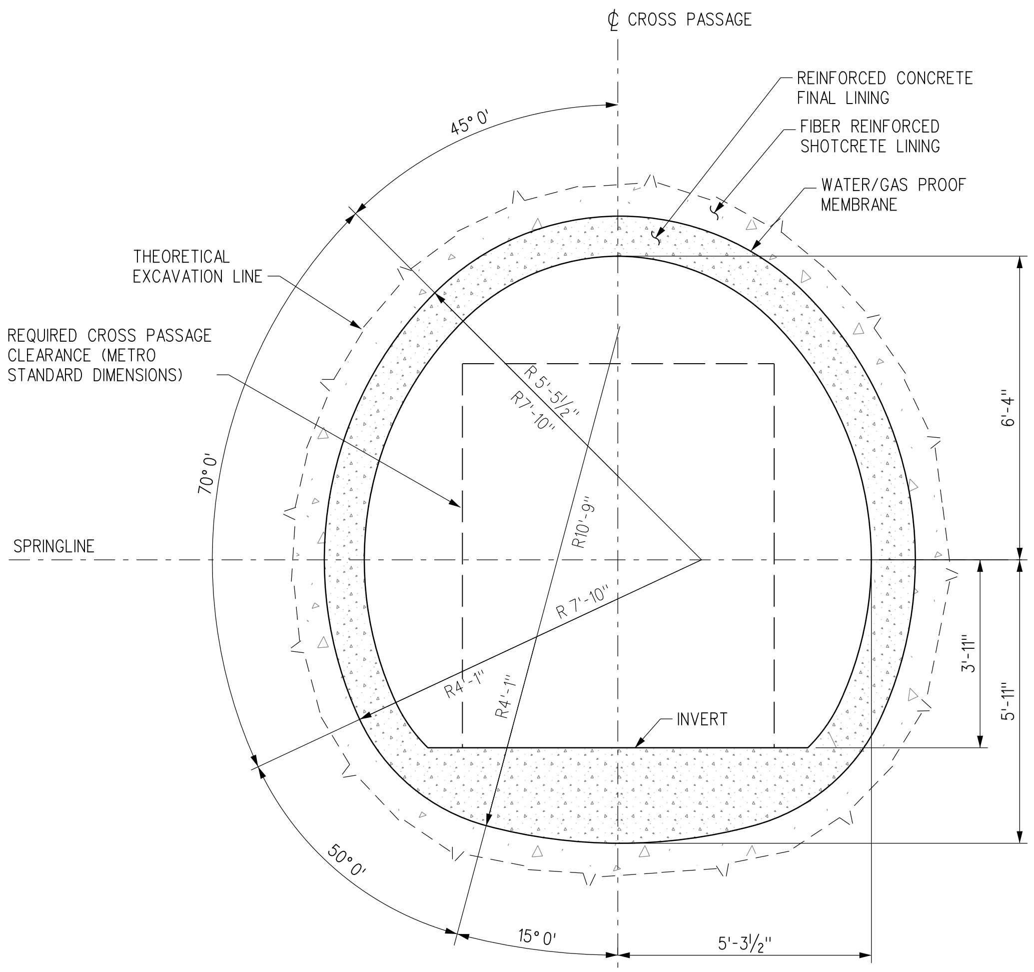

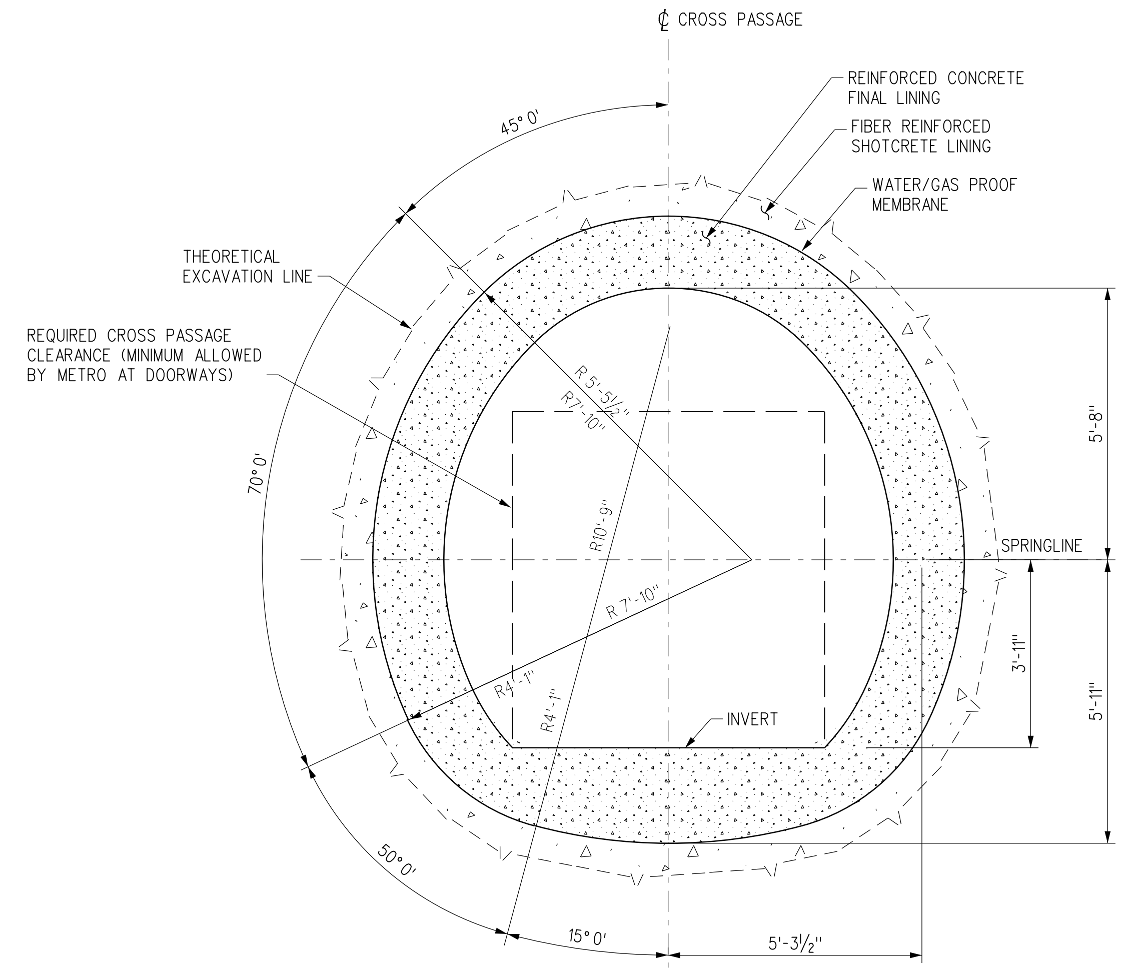

SECTION C / Y-401

NOTES:

1. METRO RAIL DESIGN CRITERIA USED AS REFERENCE FOR CROSS PASSAGE CLEARANCE REQUIREMENTS.

2' 1' 0 2' 4'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY Y. SUN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE SECTIONS

CONTRACT NO
DRAWING NO Y-404
REV
SCALE 1/2"= 1'-0"
SHEET NO

Model Name: Default

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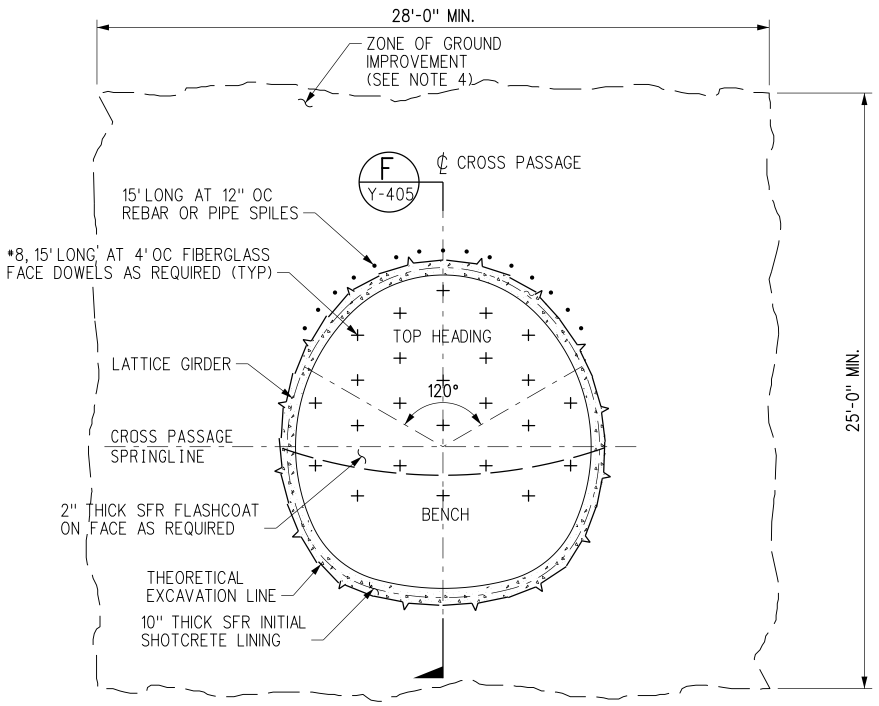
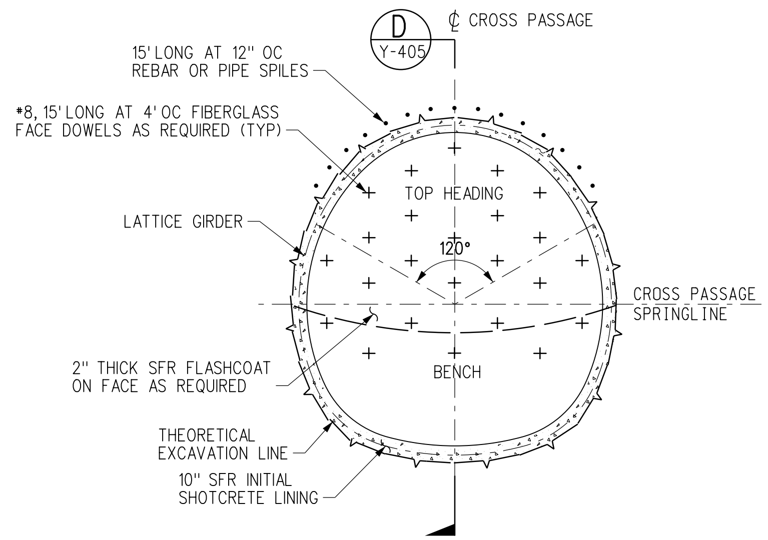


CROSS SECTION
SUPPORT TYPE 3
E Y-405



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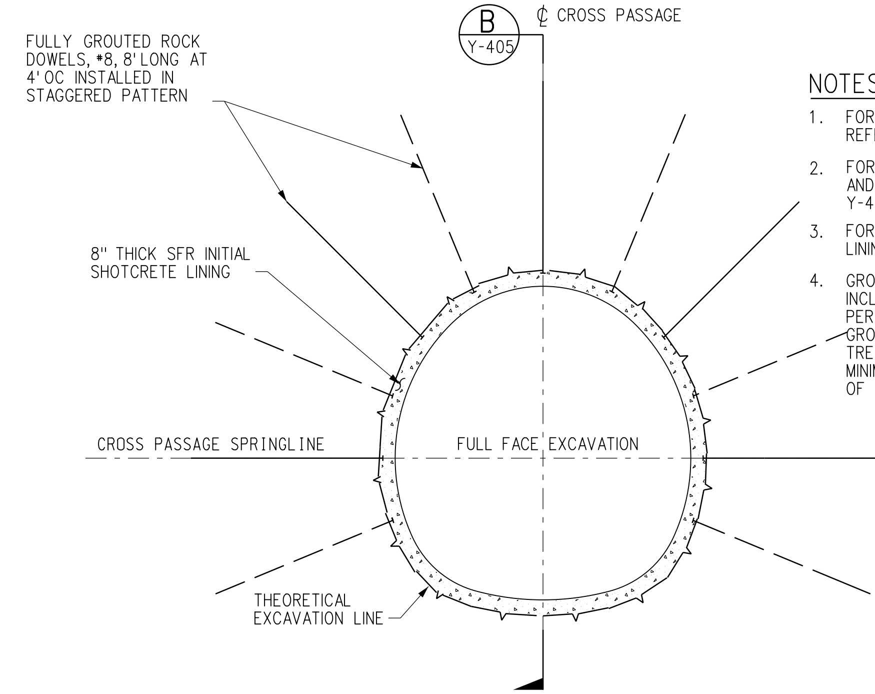
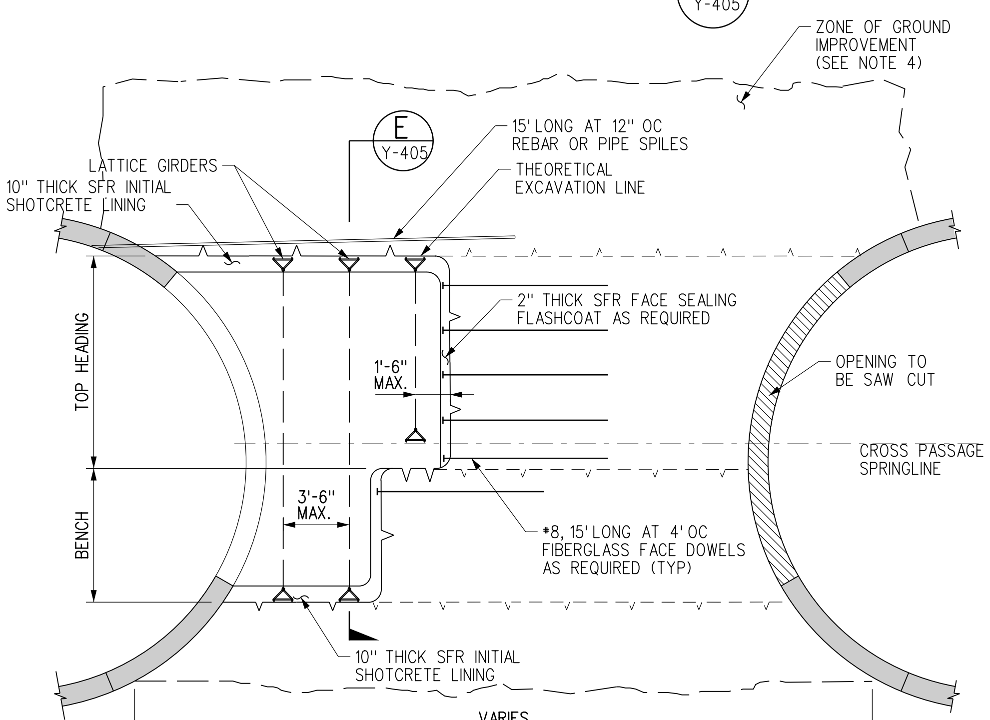
C Y-405



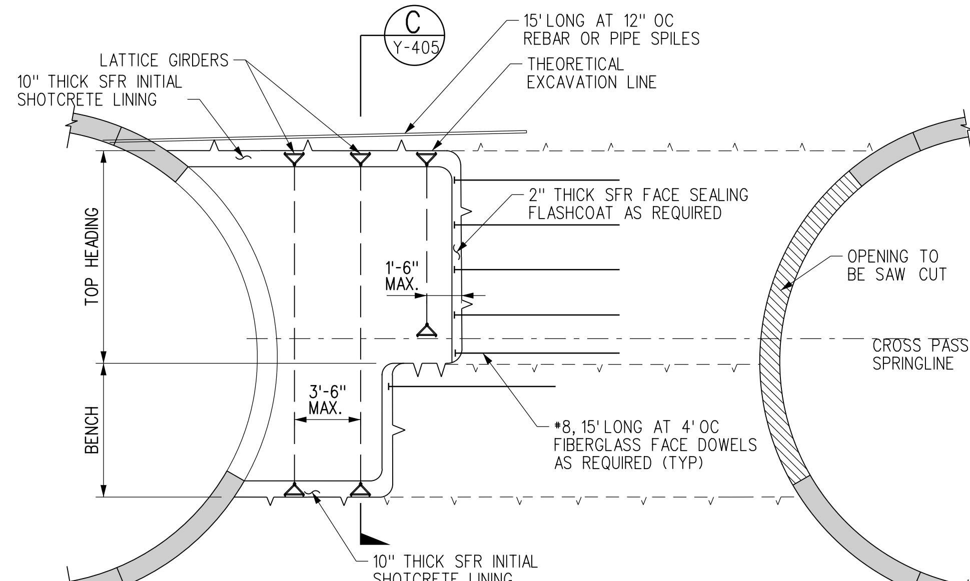
CROSS SECTION
SUPPORT TYPE 1
A Y-405

NOTES:

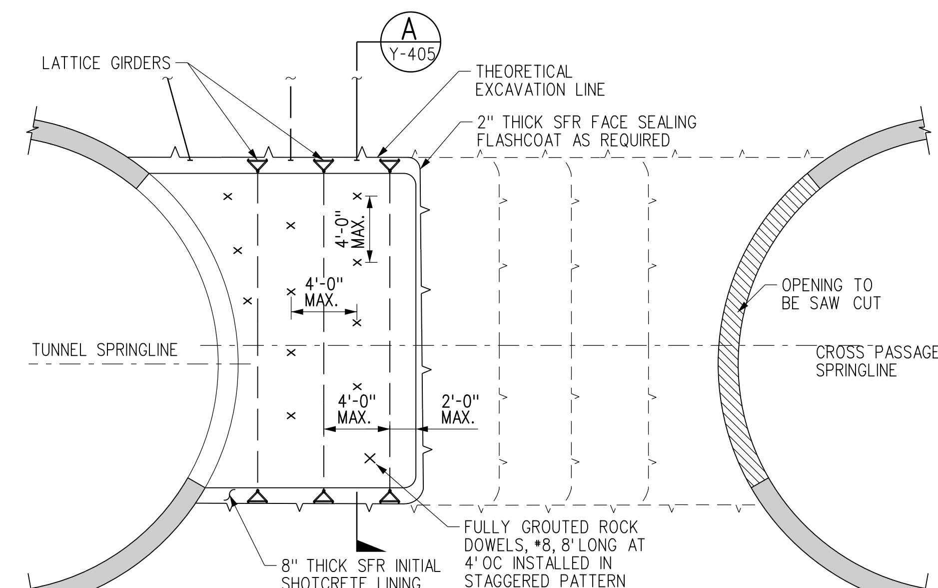
1. FOR CROSS PASSAGE GEOMETRY, REFER TO Y-401
2. FOR CROSS PASSAGE BREAKOUT AND CONSTRUCTION SEQUENCE, SEE Y-402 AND Y-403.
3. FOR CROSS PASSAGE FINAL LINING, REFER TO Y-401
4. GROUND IMPROVEMENT METHODS INCLUDE, BUT ARE NOT LIMITED TO, PERMEATION GROUTING, CHEMICAL GROUTING, OR GROUND FREEZING. TREATED GROUND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 400 PSI AT 28 DAYS.



LONGITUDINAL SECTION
SUPPORT TYPE 3
F Y-405



LONGITUDINAL SECTION
SUPPORT TYPE 2
D Y-405



LONGITUDINAL SECTION
SUPPORT TYPE 1
B Y-405

PRELIMINARY

4' 2' 0 4' 8'

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY Y. SUN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
CROSS PASSAGE
INITIAL SUPPORT

CONTRACT NO
DRAWING NO Y-405
REV
SCALE 1/4" = 1'-0"
SHEET NO

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10/18/2013
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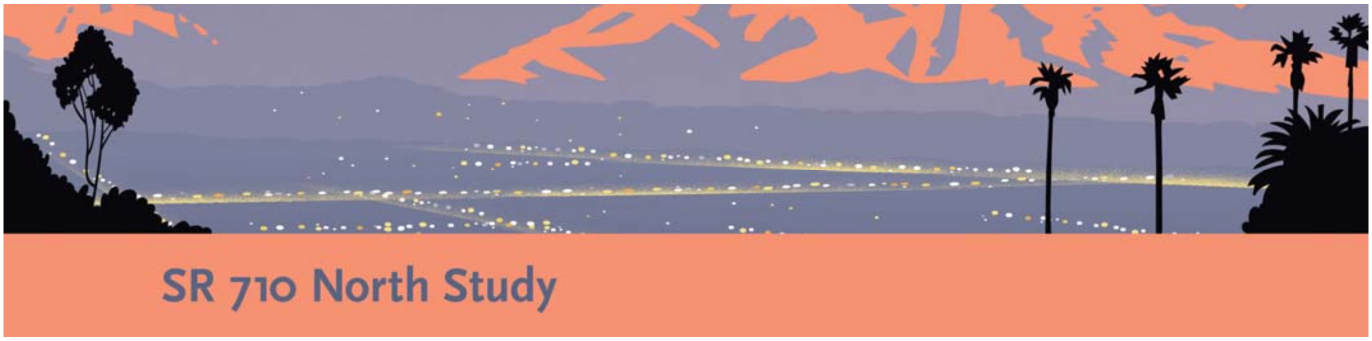
Appendix F
TM-4C Preliminary Design Concepts for the Freeway Tunnel Internal Structure



TECHNICAL MEMORANDUM 4C

Preliminary Design Concepts for the Freeway Tunnel Internal Structure

PREPARED FOR: Metro

COPY TO: Caltrans

PREPARED BY: Jacobs Associates/CH2M HILL

DATE: August 22, 2014

PROJECT NUMBER: 428908

1.0 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This TM describes the preliminary design concepts for the internal structure elements of the twin-bore and single-bore variations of the Freeway Tunnel Alternative. The internal structure consists of the horizontal and vertical elements (mainly slabs and walls) to be constructed within the tunnel after the excavation is performed and the support installed. These elements would eventually make up the roadway decks and the walls separating the travel lanes from the emergency exit walkways. At this stage, the design and support details provided herein are of a conceptual nature, and this TM presents one feasible option for the internal structure. Preliminary drawings for the internal structure concept presented herein are included in Attachment A.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2.0 Internal Structure Description

2.1 Geometry

The preliminary design concept presented at this time for the internal structure consists of a double-deck roadway constructed of cast-in-place (CIP) and precast reinforced concrete. Each deck has two 12-foot-wide travel lanes, a 1-foot shoulder, and a 10-foot shoulder for vehicles. Figure 1 shows a cross section of a Freeway Tunnel bore (refer to *Bored Tunnel Geometry*, JA 2014a, for more details). Each deck also has a walkway, which serves as the emergency egress route in the event of an emergency such as a tunnel fire. Access is available to the walkway at intervals spaced at approximately 656 feet, and the roadway walls and the access doors are two-hour fire rated (per NFPA 502 requirements). A partition slab, which would support various tunnel system components, is provided at the mid-height of the lower wall. The roadway surfaces would have a 2% cross slope for drainage. The upper deck is supported on walls that are in turn supported on corbels that bear on the segmental lining near the invert of the tunnel. The lower deck is shown to be directly supported on elastomeric bearings that rest on the corbels. Tunnel ventilation and systems are located in the enclosures formed between the tunnel lining and the walls and slabs of the internal structure.

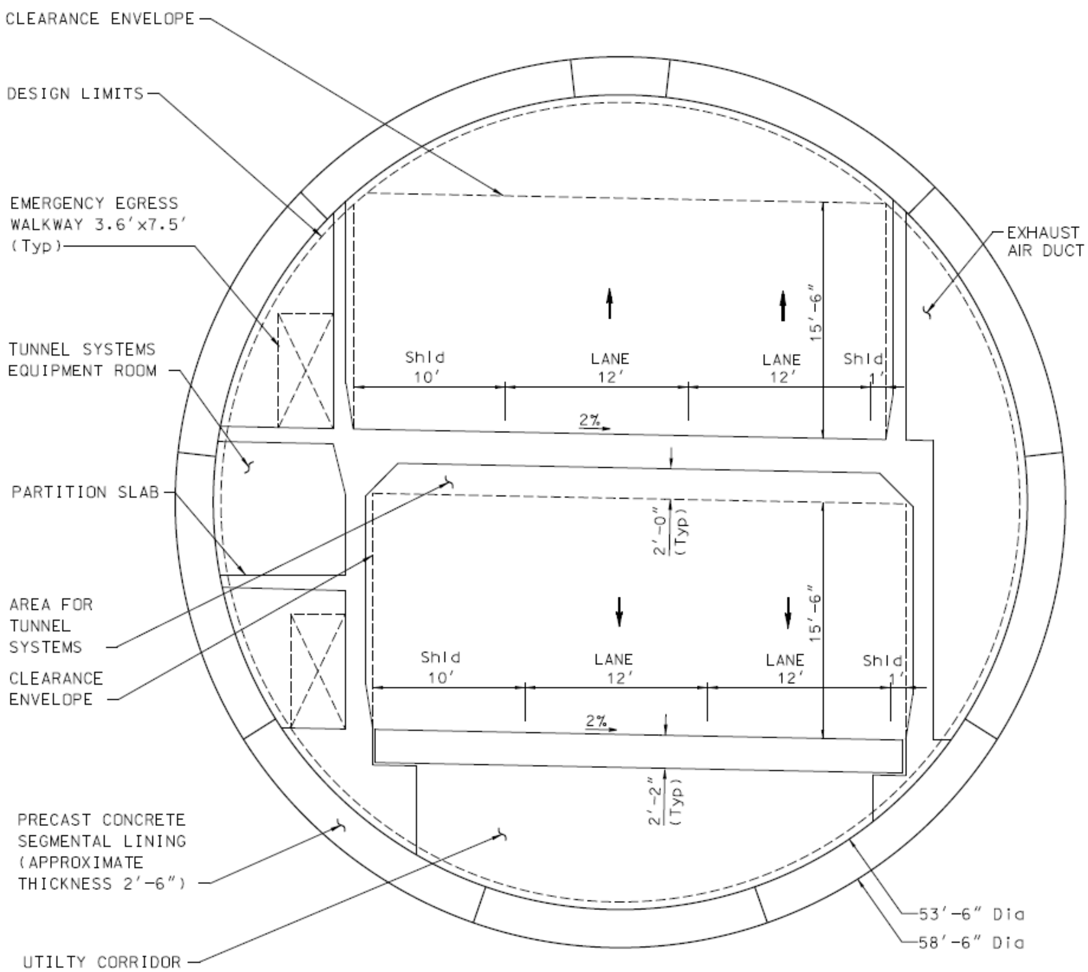


Figure 1. Cross Section of Freeway Tunnel Bore

2.2 Connections

The lower portion of the walls at each deck level is designed to be in the shape of a safety barrier to redirect vehicles impacting the tunnel walls. The internal structure is connected to the lining at the top of the upper walls, at the bottom of the lower walls through the main corbels, and at the upper level roadway slab, as shown in the drawings (Attachment A). The top wall connection is designed with a compressible joint material so that it would allow inward movement of the segmental tunnel lining under external loads without introducing compression into the walls. The connection, however, supports the top of the walls to the extent required for vehicle collision or seismic events. The connection at the upper roadway deck is designed to fully restrain the internal structure against the potentially large inertial sway displacements that could occur during a seismic event. A small amount of sympathetic displacement of the internal structure would occur due to the racking or ovalling of the tunnel lining due to this connection. This displacement is, however, far smaller in magnitude than the potential inertial displacements as discussed in Section 5. The partition slab cantilevers from the lower deck walls, and the connection between this slab and the tunnel lining consist of compressible filler thick enough to accommodate differential displacements during a seismic event. The corbels that support the internal structure are CIP structures doweled into the tunnel lining.

The joint between the top walls and the tunnel lining would need to be sealed to separate the ventilation and walkway spaces from the roadway so that tunnel ventilation can function as designed. An elastomeric (e.g., polyurethane or polysulfide) seal that creates a positive seal between the surfaces of the wall and lining, and can accommodate some movement in extension and compression, would be used to seal the gap. Additional details on the connection can be found in Section 5.

2.3 Roadway Surface

The finished surface of the concrete slabs of the upper and lower decks would provide the pavement surface. The surfaces would be prepared in accordance with Caltrans specifications for concrete roadway surfaces. A drainage collection system would need to be included on the low side of the sloped roadway; the drainage design details will be evaluated in subsequent phases of the design of this alternative. The current conceptual design is based on regularly spaced drainage inlets connecting to a carrier drain outside the main internal structure. The design will need to consider potential spill sizes but currently it is assumed that hazardous tanker trucks are excluded from the any of the variations of the Freeway Tunnel alternatives, which reduces the risk of toxic and flammable material spills in the tunnel.

2.4 Construction Sequence

The construction of the internal structure would lag the tunnel construction by a distance that depends on the construction logistics involved with TBM mining operations, the need to haul segmental lining pieces to the heading and excavated material away from the heading, and TBM abandonment. It is envisioned that construction traffic would travel on the segmentally-lined invert during construction of the internal structure. The lower roadway slab can be either precast or cast-in-place, and it is envisioned that the remainder of the internal structure would be cast-in-place at this time, because of the presence of cantilever slabs and the requirement for full moment connections at all joints. CIP construction can also be adapted much more easily than precast construction to the as-built condition of the tunnel lining. It may, therefore, not be feasible to precast the other internal structure components because of the structural and construction constraints. The construction sequence, and details regarding precast vs. cast-in-situ alternates will be investigated in more detail in subsequent design phases, and ultimately could be left to the contractor to optimize.

Although there are various ways to sequence construction of the internal structure, for preliminary design, it has been assumed that the construction sequence is as follows (and shown in Figure 2):

1. The CIP corbels at the base of the structure would be constructed first.
2. The lower walls would be constructed next.
3. The flared wall tops, the upper roadway deck and the upper walkway slab would then be constructed. The lower partition slab can be constructed during this step or as an additional step.
4. The upper walls would then be cast above the upper deck slab. The walls and the tunnel lining would be connected, as shown on the drawings in Attachment A.
5. Finally, after completion of the TBM drive, the lower deck slab would be cast in place or brought in as a precast slab.

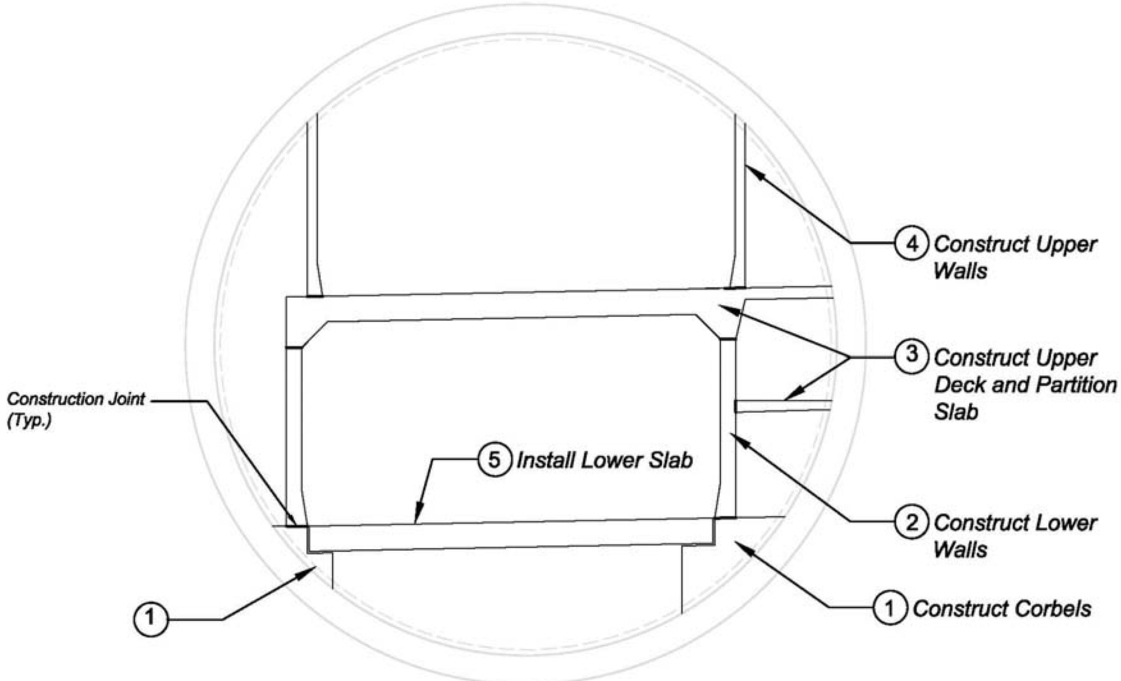


Figure 2. Internal Structure Construction Sequence

2.5 Seismic Vault Section

The internal structure cross section at the vault (enlarged tunnel section at the active fault crossings) is essentially the same as a typical cross section; however, the span of the upper and lower slabs would increase by approximately 20 inches. See *Preliminary Design Concepts for Fault Crossing* (JA, 2014b) for details. The key difference is that additional space is provided to accommodate the fault offset. Therefore, when the fault offset occurs, the vault sections would displace relative to each other horizontally and vertically, and the additional space provided would allow traffic to be maintained through the fault zone after the earthquake.

3.0 Preliminary Design Criteria and General Requirements

3.1 Design Codes and General Requirements

The codes and standards applicable for design include, but are not limited to, the following:

- California Department of Transportation (Caltrans) *Bridge Design Specifications* (BDS) (Caltrans, 2011).
- AASHTO, *LRFD Bridge Design Specifications*, 4th ed., American Associations of State Highway and Transportation Officials, Washington, DC, 2007 (AASHTO, 2007).
- California Department of Transportation *Seismic Design Criteria* (SDC), Version 1.7. April 2013 (Caltrans, 2013).
- *ITA Guidelines for Structural Fire Resistance for Road Tunnels* (ITA, 2004).
- National Fire Protection Association (NFPA). 2011. NFPA 502: Standard for Road Tunnels, Bridges, and Other Limited Access Highways.

The general requirements of the Freeway Tunnel Alternative which dictate the geometry are described in *Bored Tunnel Geometry* (JA 2014a).

3.2 Service Life and Durability

At this stage of the design, Caltrans has not established service life and durability of the various underground structures. Structures designed in general accordance with the Caltrans Bridge Design Specifications (2011) are expected to have a service life of 75 years. The design of the internal structure would provide the required durability by appropriate design and construction quality control of key components—such as concrete quality and density, cover to reinforcing steel, ventilation seals—and by making the internal structure fire resistant.

The roadway would require drainage for water entering the roadway. Potential sources of water are via traffic vehicles, the water used for firefighting, runoff from rain, and cleaning the tunnel. The roadway is drained using a 2% cross slope to the roadway surface, and the collected water on each level would be diverted to a sump structure at a low point or in the lowest cross passage, where it may be pumped out of the tunnel. The vertical carrier pipes which carry drainage from the top level to the bottom level may be embedded in the walls of the internal structure.

For fire resistance, the internal structure would be required to withstand the design fire event without loss of structural integrity. The design would be in accordance with ITA (2004), NFPA (2011), and other standards as applicable. The design fire event has not been determined at this time. Section 6 discusses key considerations for a fire-resistant design of the internal structure.

3.3 Preliminary Static and Seismic Design Criteria

Static design criteria for the internal structure are from Caltrans (2011). The internal structure is designed to support its self-weight, the weight of various tunnel system components (such as Vehicle Messaging Signs and utilities), and vehicular loading. Structure deflection under loads should be within limits specified by Caltrans (2011). Details regarding static loading are covered in Section 4.

Ground motions and parameters for seismic design are summarized in Tables 1 and 2 and were developed based on *Preliminary Earthquake Acceleration Response Spectra, SR 710 North Study, Los Angeles County, California,* (CH2M HILL, 2013). These parameters were derived based on procedures outlined in Caltrans (2013). Two levels of seismic event, consisting of a Safety Evaluation Earthquake (SEE) and a Functional Evaluation Earthquake (FEE), are to be considered for the freeway tunnel internal structure design.

The SEE is a seismic event that has a 5% chance of being exceeded in 50 years, which is equivalent to a 975-year (1,000-year nominal) return period earthquake. The structure is required by Caltrans (2013) to meet several performance criteria. These criteria are described in more detail in Section 5. In summary, the internal structure should be able to survive the seismic event without collapse, and inelastic behavior is permitted in the structure. The FEE is a smaller seismic event, with a 100-year return period. The performance requirement under FEE is that the internal structure remains fully functional with minimal damage and at the location of any seismic displacements some structural repairs may be necessary.

Table 1. Summary of Seismic Ground Motions for Preliminary Design[1,2]

Design Ground Motions		**FEE (100 yr)**	**SEE (1,000 yr)**
PGA (horizontal) (g)	Rock	0.21	0.75
	Soil	0.23	0.84
PGV (horizontal) (ft/s)	Rock	0.83	2.92
	Soil	0.92	3.33

[1] These are parameters associated with horizontal ground motions. Parameters associated with vertical ground motions can be estimated using the vertical-to-horizontal (V/H) ratio of 0.85.

[2] Source: CH2M HILL (2013)

Table 2. Soil and Rock Mass Dynamic Parameters for Preliminary Design

Soil / Rock Formation[1]	Total Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Primary Wave Velocity (ft/sec)	Effective Shear Wave Velocity [2] (ft/sec)	Effective Dynamic Shear Modulus (ksi)	Effective Dynamic Young's Modulus (ksi)
Fill	120	500	816	350	3.2	7.6
Old Alluvium (<50 ft deep)	135	1,080	2,248	756	16.6	44.9
Old Alluvium (>50 ft deep)		1,650	3,435	1,155	38.8	104.9
Fernando Formation	136	1,080	2,248	864	21.9	59.1
Puente Formation, Tp-2	134	1,600	2,993	1,280	47.3	123.1
Puente Formation, Tp-1		2,200	4,116	1,760	89.5	232.7
Topanga Formation, Tt-2	134	1,300	2,432	1,040	31.3	81.3
Topanga Formation, Tt-1		2,900	5,425	2,320	155.5	404.4
Basement Complex Rocks, Wqg-2	158	1,600	2,771	1,280	55.8	139.6
Basement Complex Rocks, Wqg-1		3,500	6,062	2,800	267.1	667.9

3.4 Static Design Loads

The internal structure has been designed for:

- *Dead Loads (DC, DW):* Weight of structures and other permanent elements, including internal structures/facilities such as invert slab, walkway, and tunnel utilities.
- *Live Loads (LL):* Vehicular loading.
- *Dynamic Load Allowance or Impact (IM):* Statically equivalent dynamic effect of vehicle impact on the roadway.

Dead load is primarily the self-weight of the structure. In vehicular loading (Caltrans, 2011), three types of loads are specified on each travel lane. These are the AAHSTO HS20-44 truck loading, a Design Tandem Load consisting of two 25,000-pound axles spaced 4 feet apart, and a Design Lane Loading consisting of a 64 pounds per square feet (psf) uniform load. To account for dynamic traffic loads, the truck or tandem loads are increased by a 33% dynamic impact factor.

The governing loading to be considered is the highest loading from the following cases:

- Design Truck + Lane Loading or
- Design Tandem + Lane Loading

The other normally required loading combination of "90% of two trucks + Lane Loading" is applicable only in longitudinal analysis of multispan bridges and is not applicable to the project internal structure configuration.

The walkways are designed for dead loads and a live load of 75 psf to account for pedestrians. The total superimposed loading on this slab is expected to be approximately of the same order of magnitude as the walkway live loading.

An air extraction duct is located on one side of each tunnel bore as part of the tunnel's ventilation system, and pressure differential is present between duct and the traveled way. For this project, based on the ventilation data, the pressure differential does not present a significant load on the internal structure. Additionally, variable message signs, fire insulation panels, drainage lines, firefighting water lines, and other tunnel system components that would be supported by the internal structure would add load to the structure. These loads, however, would not be significant compared to the structure weight and the vehicular loading, but will be considered as part of future design phases.

3.5 Vehicle Collision Loads

Given the conceptual nature of preliminary design, a comprehensive analysis of vehicle collision loads was not conducted at this stage. During future design phases, an assessment in accordance with relevant Caltrans (2011) specifications will be performed. A safety barrier shape has been included at the bottom portion of the walls, and is expected to satisfy typical vehicle collision loading (see drawings in Attachment A), although some strengthening of the walls (or lower portions of them) may be necessary.

3.6 Seismic Design Loads

At this time the internal structure is designed for:

- *Earthquake Loads (EQ):* Seismic design loads on the internal structure resulting from ground motions

If the internal structure was independent of the tunnel lining, then the ground motions would induce inertial forces in the internal structure; however, because the internal structure is rigidly connected to the tunnel lining at the level of the upper roadway deck, inertial forces are not expected to be induced on the internal structure itself. Inertial forces are expected at the connection of the internal structure to the tunnel lining. See Section 5 for additional details.

The seismic forces in the internal structure members would then be primarily due to the racking effect of the tunnel lining (Wang, 1993; Hashash et al., 2001; Hashash et al., 2005). Under racking, the lining distorts or ovals because of the shearing deformation induced by the ground motions. This in turn induces sympathetic deformation and consequent forces in the internal structure. In future design phases, dynamic analyses should be performed to verify whether the inertial effects are negligible on the internal structure.

At the location of the fault rupture, the tunnel is oversized to accommodate the fault offset, essentially shielding the internal structure. Refer to *Preliminary Design Concepts for Fault Crossing* (JA, 2014b) for details.

3.7 Load Combinations and Load Factors

3.7.1 Static Design

The internal structure design for static loads is for the *STRENGTH I* load combination per Caltrans (2011). The load combination and load factors are as below:

1.25 (DC+ DW) + 1.75 (LL+ IM)

3.7.2 Seismic Design

The internal structure design for seismic loads is for the *EXTREME I* load combination per Caltrans (2011). The load combination and load factors are as below:

1.0 (DC+ DW) + 1.0 (LL) +1.0 (EQ)

The earthquake effects are typically from both horizontal and vertical components of the ground motions. The preliminary analysis primarily focuses on the racking due to vertical shear waves as that typically controls. The effects of the vertical component of the ground motion will be investigated in more detail during future design phases via a full dynamic analysis of the tunnel and internal structure which implicitly takes into account both horizontal and vertical effects.

3.8 Deflection Limits

3.8.1 Static Limits

The vertical deflection limit for the internal structure, per Caltrans (2011), is Span Length/800 for the live vehicular load (including the dynamic load allowance) for service load conditions. The live load deflection should be taken as the larger of (1) that resulting from the design truck alone, or (2) that resulting from 25% of the design truck taken together with the design lane load. Although this deflection limit is optional, it is invoked here as an additional check. The load combination for checking deflection limits is the *SERVICE I* load combination, per Caltrans (2011), as shown below:

1.0 (DC+ DW) + 1.0 (LL+IM)

3.8.2 Seismic Limits

The key deflection limits for seismic loading would be the lateral movements caused by racking deformation. Several lateral movement limits apply, per Caltrans (2013). See Section 5 for details.

4.0 Preliminary Static Design Evaluations

4.1 Analysis Methodology

The static analysis of the internal structure frame for dead and live loads was performed using STAAD (Bentley, 2012) and SAP2000 V15 (CSI, 2011) programs for structural analysis and design. These programs accept geometric and material properties, loads, and support definitions, and provide the axial load, moment, and shear in the internal frame, as well as deflections. The internal frame was modeled as a concrete frame with a 28-day concrete strength of 4,000 psi. The moment of inertia of the frame members was modified to account for cracking, per Caltrans (2011). The slabs were modeled as beams, with a cracked moment of inertia equal to 50% of the gross inertia. The walls were modeled with a cracked moment of inertia equal to 70% of the gross inertia. The frame was modeled as pin-connected to the supporting corbels. The connections between the internal structure and the tunnel lining were modeled as supports with the appropriate stiffness values.

Vehicle live loads are distributed longitudinally (along the structure length) and also transversely. These effects are best captured by a three-dimensional (3D) analysis. Only transverse distribution effects can be captured by a two-dimensional (2D) analysis. For this reason, both 3D and 2D analyses were run for the internal structure. All of the design of the internal structure was based on 2D models, but with the results appropriately scaled based on the effects observed from the 3D model.

Figure 3 shows an example of the 2D frame analysis in STAAD with the static loads.

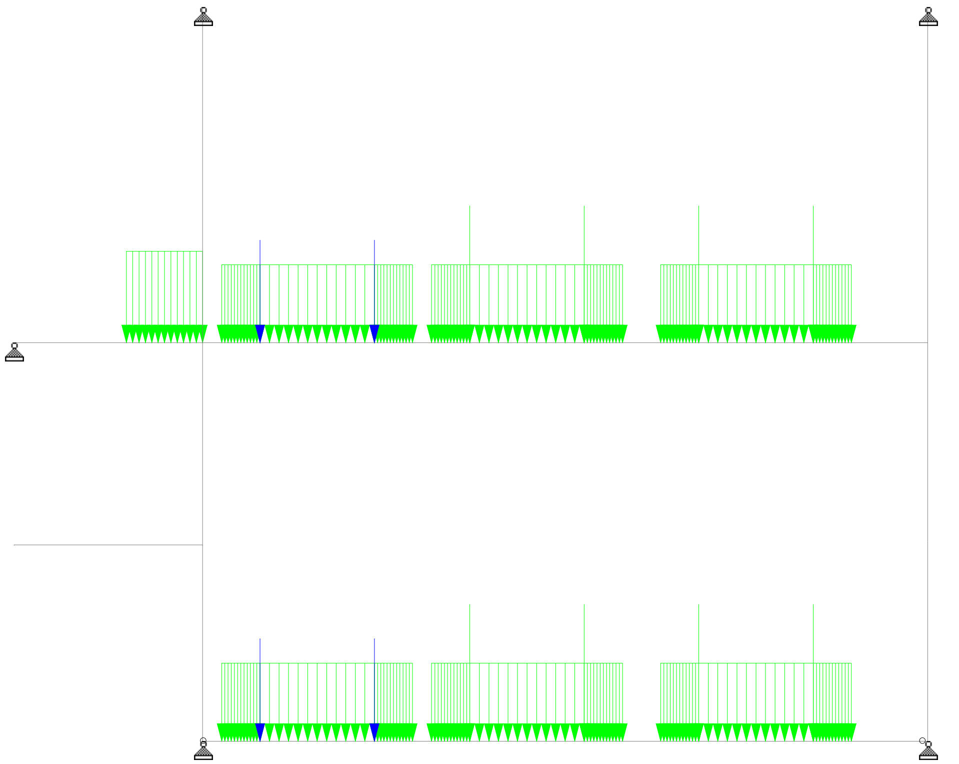


Figure 3. 2D Frame Model of the Internal Structure Showing Vehicular and Live Loading

4.2 Static Design Results

Analysis of the vehicular lane loading indicated that the AASHTO HS 20-44 truck load governed for member forces, and that the design tandem did not. Member forces for static design are based on the 2D model, with live load forces scaled up where appropriate because of the effects observed in the 3D model. Each member of the internal structure is subjected to a moment and thrust. Moments from the analysis typically need to be amplified for side sway and slenderness to obtain the design moments. The need for amplification was checked per Caltrans (2011). The walls were braced against side sway in the static condition because of the connection to the tunnel lining at the walkway level. No moment amplification due to side sway therefore resulted. A check for moment amplification due to slenderness also indicated that moment amplification was not required.

The adequacy of the internal structure members under this moment and thrust was checked using a moment thrust interaction diagram. The program spColumn (Structurepoint, 2010) was used to generate the moment-thrust interaction per the Caltrans (2011) resistance factors.

According to the moment thrust-interaction analyses, the upper slab is required to be 2 feet 2 inches thick with approximately 1% reinforcement. Under live loads, it is subject to 0.17-inch deflection. This deflection is less than the span length/800 limit, which is equal to approximately 0.5 inch for the span length of the roadway shown on Figure 1. The lower slab is also designed to be 2 feet 2 inches thick with approximately 1% reinforcement. Under live loads, it is subject to 0.25-inch deflection, which is also less than the span length/800. This slab has a higher maximum moment and deflection than the upper slab because of its simply supported ends. Both slabs therefore satisfy the strength and serviceability requirements of Caltrans (2011). The pedestrian walkway slab is 12 inches thick with 1% reinforcement. The partition slab is a tapered slab. The thickness varies from 12 inches to 9 inches with 1% reinforcement. There would be other dead loads and equipment loads on this slab; however, the thickness and reinforcing indicated above are expected to be adequate.

A 10-inch thickness is adequate for the upper wall with 0.5% reinforcement. The lower walls are required to be 16 inches thick with 1.6% reinforcement. As discussed above, the walls have to be checked for vehicle collision loads during subsequent design phases. The reinforcing percentages mentioned above are for the main reinforcement. Additional reinforcement would be required for shear ties and for distribution reinforcement.

5.0 Preliminary Seismic Design Evaluations

The seismic design evaluations were preliminarily performed for the SEE event, since that is the larger event and is expected to control design. The FEE evaluations will be performed during future design phases.

5.1 Analysis Methodology

5.1.1 Racking Analysis

A Plaxis (PLAXIS 2D, 2010) model was used to analyze potential ovaling deformations of the tunnel lining structure subjected to an estimated maximum ground racking displacement field based on the maximum earthquake-induced ground free field shear strain during the SEE event. As these analyses are "pseudo static," the effects of inertial loading on the lining and internal structures are not considered. Two geologic sections were modeled—one where the tunnel is in rock, and another where the tunnel is in soil. See *Preliminary Design Concepts for the Freeway Lining and Cross Passages* (JA, 2014c) for details of the geologic sections, analysis parameters, and key assumptions. For each section, the estimated free-field seismic displacement was applied to the ground in order to evaluate the resulting forces on the tunnel lining and internal structure. Figure 4 shows the applied displacement. The slabs of the internal structure and the tunnel lining were modeled as pinned with tied-in connections, while the connections between the top walls and the lining were modeled as relatively flexible connections that provide limited resistance in tension and compression before yielding. The bracket connections at the top of the wall would be bolted to the tunnel lining at a regular spacing along the wall (see drawings in Attachment A) designed to produce this behavior.

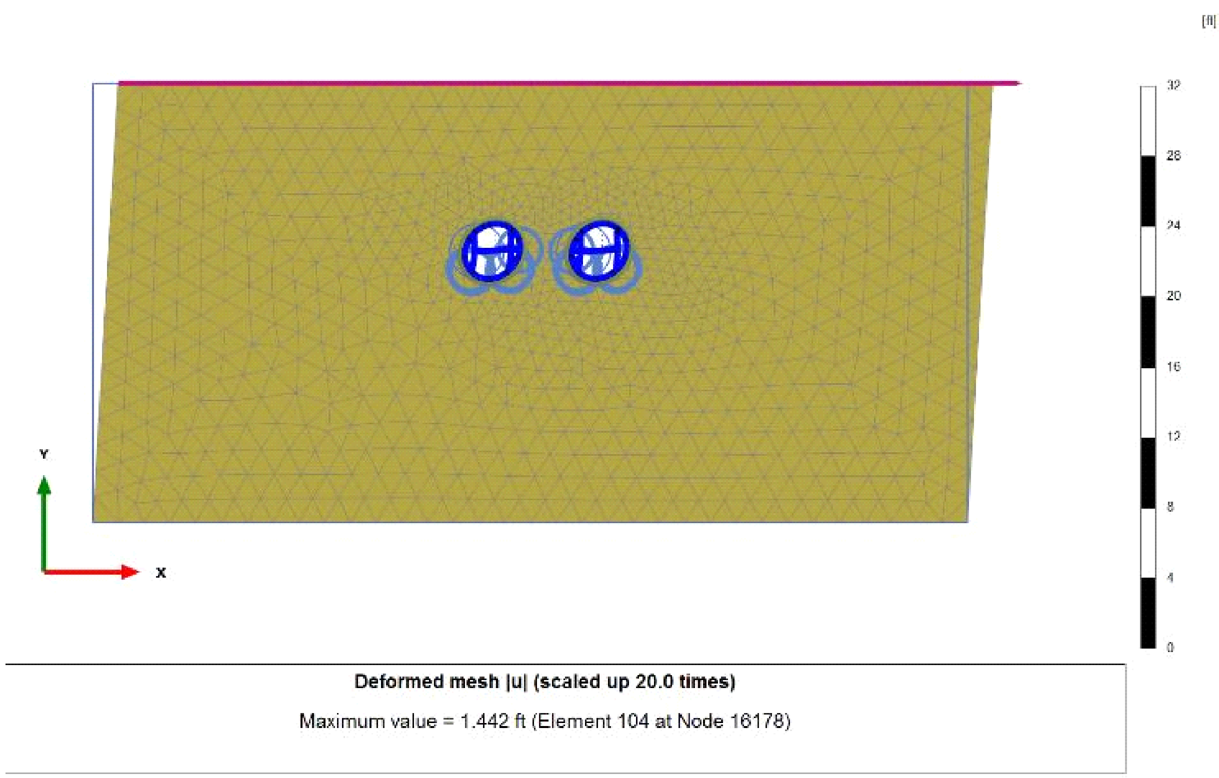


Figure 4. Racking Analysis in Plaxis Depicting Applied Free Field Deformations

Figure 5 shows the tunnels after free-field seismic displacement is applied to the model. The tunnels distort into an oval shape. Note that the deformations shown in the figure are scaled up by 10 for illustrative purposes. The results from the numerical modeling indicated that the tunnel lining and internal structure in alluvial soil would generally experience greater displacements, and thus larger forces, than the tunnel section in rock.

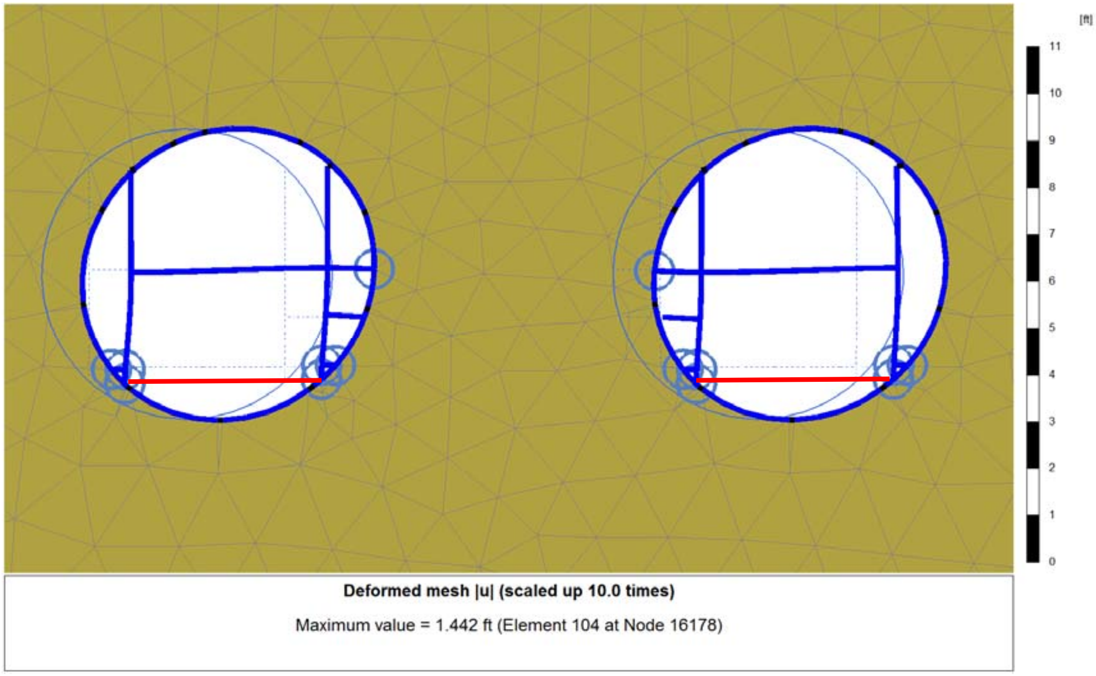


Figure 5: Plaxis Model of the Deformed Tunnels after Application of Seismic Displacements

(Note: Deformations are scaled up 10 times)

5.1.2 Inertial Analysis

As mentioned previously, the internal structure has a rigid connection at the upper level deck to the tunnel lining. Inertial forces (which are typical of a building frame, for example) would not be induced in the internal structure. The connection would, however, attract an appreciable amount of lateral force from the inertial effect. Shear would also be induced in the connection due to both horizontal and vertical components of the ground motions. The connection also sets the period of vibration of the internal structure to virtually zero since the internal structure cannot sway because of inertia. With this vibration period, the spectral acceleration of the internal structure effectively reduces to the peak ground acceleration (PGA). The lateral force in the connection due to the inertial effect was estimated and accounted for in the design by summing up the masses of the internal structure and the vehicles supported by it and multiplying the mass by the PGA. This estimate as well as the inertial effect on the internal structure will be verified using dynamic analyses in future design phases.

5.2 Preliminary Design Procedure

The forces and displacements generated from the racking and static analyses were used to perform a design check of the structure per Caltrans (2013). This is a broad check on the demand versus capacity of the structure in terms of displacement and force. The imposed demand must be less than the available capacity.

Displacement demand is described in terms of ductility and is both global and local. The global ductility demand is defined as the ratio of the total lateral displacement induced by the earthquake forces in an elastic analysis of the structure to the displacement required to produce first yield in the structure members. Per Caltrans (2013), the typically available ductility capacity ranges from 1 to 5 for various types of structures, from pier walls to multicolumn bents. The internal tunnel structure most closely resembles the multicolumn bent (Caltrans, 2013), for which available ductility capacity is in the range of 4 to 5. The local displacement ductility demand is defined as the ratio of the combined yield and plastic displacement of the member to its yield displacement. A local displacement capacity of 3 or more is required for the structure elements.

Force demand on the structure is taken as the sum total of the forces from the static and seismic analyses. These forces are moments, shears, and axial loads. The design intent (Caltrans, 2013) is to ensure that plastic hinges, if any, form in the walls and not in the beam-type elements. A plastic hinge forms at a point at which the induced moment is larger than the plastic moment capacity of the section. The plastic moment capacity can be determined from a Moment-Curvature analysis. Additionally, the induced curvature at the plastic hinge should be less than the ultimate curvature capacity of the section. With this intent, the following can occur:

- The induced moment in the wall can be larger than the plastic moment capacity at the given axial load level. However, the corresponding curvature should be less than the ultimate curvature capacity of the wall. The ultimate curvature capacity is the point at which the concrete crushes or the reinforcement fractures.
- The beam-type elements in the internal structure, which are the roadway decks, should remain essentially elastic for the induced seismic forces. The induced moments in these elements should be smaller than the plastic moment capacity.

With respect to shear, essentially elastic behavior is required of both walls and columns. The shear capacity of the walls should be greater than the imposed shear and should also be greater than the overstrength shear, which is defined as the maximum shear that can be induced in the walls under seismic forces. The shear capacity of the beams should be larger than the imposed shear. Beam shear capacity should also be larger than the shear that can be induced in the beams if the wall under them reaches overstrength shear.

Caltrans (2013) allows the use of expected material strengths instead of the minimum specified strengths for concrete and steel for seismic design. For concrete, this expected material strength is 30% greater than the minimum specified. For Gr. 60 reinforcing steel, this expected material strength is approximately 13% larger than the minimum specified. Other specifications pertaining to inelastic seismic design include (1) maximum strain limits for concrete and steel, (2) reduced ultimate strains for steel reinforcing depending on bar sizes, and (3) resistance factors for bending and shear in concrete that are different from the usual Caltrans (2011) factors. Caltrans (2013) also idealizes the plastic moment-curvature capacity of concrete sections by not allowing moment increases beyond the idealized yield curvature of the section.

5.3 Design Results for the Internal Structure

5.3.1 Displacement Demand

An elastic pushover analysis of the frame indicates that the joint between the wall and the upper roadway deck reaches first yield at a displacement of approximately 2.4 inches. The inertial displacement is zero, and the racking analysis produces a displacement of 2.2 inches. The global ductility demand is then approximately 0.9. The multicolumn bent is expected to have a capacity of between 4 and 5. The capacity is therefore significantly greater than demand.

An analysis of the 16-inch and the 10-inch walls for local displacement capacity indicates that the ratio of the yield plus plastic displacement to the yield displacement is approximately 5 for both walls. A minimum ratio of 3 is required. Capacity is therefore greater than demand.

5.3.2 Force Demand

A moment curvature analysis of the 16-inch wall indicates that the plastic moment capacity of the wall, expected to be approximately 116 kip-ft, exceeds the imposed moment of 82 kip-ft. The nominal shear capacity of the wall with, for example, #4 shear links spaced 8 inches vertically and 12 inches horizontally is approximately 12 kips. This is larger than the induced shear of 5 kips and also the overstrength shear, estimated to be 7 kips. These results indicate that essentially elastic behavior of the walls can be expected during the seismic event, although this is not required by Caltrans (2013). No plastic hinge formation is expected.

A moment curvature analysis of the 27-inch deck indicates that the plastic moment capacity, expected to be approximately 175 kip-ft, exceeds the imposed moment of 140 kip-ft. The nominal shear capacity of the deck with, for example, #4 shear links spaced on a 12-inch x 12-inch grid is approximately 25 kips. This is larger than the induced shear of 16 kips. If the walls were to reach their overstrength limits and induce the corresponding moment demand of 166 kip-ft and shear demand of 14 kips on the beam, the beam capacity estimated above would still exceed demand. The beams, therefore, remain essentially elastic, as required by Caltrans (2013).

5.4 Preliminary Design Provisions for Connections

5.4.1 Top Wall Connection

From the analyses, it has been determined that the forces in the top wall connection are minimal. This is expected since the very flexible bent plate connection, combined with the relatively flexible wall, does not restrain the internal structure to any great extent. The connection is expected to yield/deform in the seismic event but would not fail. The key result for this connection is the relative deformation that takes place between the top wall and the tunnel lining and its effect on the elastomeric ventilation sealant described earlier. Although the sealant can accommodate some movement in extension and compression, it is expected that the sealant could be damaged during the SEE event because of the relatively large difference in the displacements of the lining and the top of the wall. Therefore, it is anticipated that some ventilation leaks would take place after the SEE event, and a repair of the joint sealant would be required. The seals are expected to survive the FEE event because of the reduced shaking level. This will be confirmed during future design phases.

5.4.2 Upper Slab Connection

From the analyses, it has been determined that the total force on the upper roadway slab is on the order of 35 kips per foot. This force can be either tensile or compressive in nature. A connection as conceptually shown on the drawings can be used to resist this force. In compression, the connection simply resists the force via bearing on the segmental lining. In tension, reinforcement is provided in the slab that is lapped with the main reinforcing bars of the walkway slab. This reinforcement is threaded into couplers that are welded to a 1.5- to 2-inch-thick steel base plate. The plate is anchored into the segmental lining via adhesive or mechanical anchors. The force developed in the walkway slab is transferred to the tension reinforcement, then to the base plate via the couplers, and finally into the segmental lining via the anchors.

The base plate can be fabricated in 4- to 6-foot-long pieces and anchored into the lining at the appropriate location. This work is expected to be performed ahead of the internal structure construction. The couplers would then be welded to the plate, or they can be pre-welded to the plate prior to installation. The rebar shown on the drawings in Attachment A would be threaded into the couplers. During internal structure construction, the walkway slab main reinforcing would be lapped with the rebar in the couplers, and the walkway slab and upper deck slab would be cast integral with one another and with the connection. This preliminary concept is one method of achieving the connection. Other feasible methods will be further explored during subsequent design phases.

5.4.3 Partition Slab Connection

The partition slab is designed to cantilever off the lower wall of the frame. The differential movement between the end of the slab and the tunnel is approximately 3 to 4 inches. A connection as conceptually shown on the drawings can be used to accommodate this movement. In the case where the slab moves towards the tunnel lining, the filler material would compress. In the case where the slab moves away from the tunnel lining, the slab would simply slide along its support.

6.0 Fire Resistance Provisions

At this stage in the study, no analysis has been performed for the internal structure with respect to fire resistance. The appropriate level of protection for the internal structure should be determined in future studies based on the design fire event combined with the effects of the ventilation system and/or fire suppression system. The fire protection measures adopted must ensure that the internal structure can withstand the design fire event without loss of structural integrity. Currently, a 100MW fire was assumed for this study and is subject to considerable evaluation in future phases.

The results of the above mentioned studies will determine the extent of fire protection measures that need to be adopted. However, based on research and previous experience, the following measures are considered appropriate to ensure the satisfactory performance of the internal structure when exposed to a 100MW fire:

- Install fire-resistant tunnel cladding, such as aluminum silicate insulation boards or a vermiculite cement coating on the underside of the upper deck slab.
- Increase the concrete cover over the reinforcing steel to restrict the temperature rise in the reinforcing steel.
- Use polypropylene fibers in the concrete mix to mitigate concrete spalling.
- Use aggregates that are thermally stable in the concrete mix.

Fire-resistant cladding directly protects the structure from a fire by isolating the structure from the flames. However, the presence of fire protection panels could significantly influence the assumptions on which the design of the ventilation system is based because the panels reduce the heat absorbed by the structure. The use of fire protection panels and the associated increase in heat load could therefore have a major impact on both the type and cost of the ventilation system to be used. Conversely, the ventilation system has a significant impact on air temperatures and therefore the gradient of temperature penetration into concrete sections. Studies will be required to determine the effectiveness of the ventilation system in limiting the increase in the wall temperature of the internal structure surfaces during the design fire to comply with NFPA (2011) criteria.

Prevention of spalling, or preserving the concrete cover, helps improve the overall structural performance of a concrete section during a fire because of the insulation protection the intact concrete cover offers to the reinforcing. The intact concrete limits the internal temperature rise and heat penetration into the concrete. Caner, et al. (2005) illustrated this in their analysis of a 300-millimeter-thick (11.8-inch) concrete lining subjected to a surface temperature of 1100°C (2000°F). The data show that the temperature at a depth of 75 millimeters (3 inches) stays below 300°C (570°F), even when the surface temperature is maintained at 1100°C for 2 hours.

The use of micro-polypropylene (PP) fibers has been shown to effectively reduce explosive spalling (Tatnall, 2002) by limiting the development of high vapor pressures within the concrete. The melting point for the PP fibers ranges from 150° to 170°C (300–340°F), which is below the range when spalling occurs. The melted PP fibers leave a network of small voids that allows the vapor pressure to escape from the concrete and limits spalling of concrete within the fire-affected area. The addition of PP fibers does have the potential to cause some problems during the mixing process and pumping of concrete. The fibers can reduce the slump and have a tendency to clump when they are introduced into the mix. The procedures outlined in ASTM C1116 address the proper mixing techniques that would help to avoid clumping.

The use of aggregates that are less prone to thermal expansion and splitting at high temperatures has been shown to improve the performance of concrete subject to high temperatures (International Federation for Structural Concrete, 2007). Other aggregate characteristics that improve the performance of concrete subjected to high temperatures are: small size, rough surface, and angular shape. Aggregates listed in order of decreasing thermal stability are: basalt, limestone, and siliceous. High-strength, dense, and low-permeability concretes are also more prone to explosive spalling. Therefore, the use of very low water-cement ratios or the addition of silica fume is not recommended for a concrete that may be subject to high temperatures.

Key elements of the internal structure requiring fire protection would be the upper roadway deck and the supporting walls. The underside of the upper roadway deck could incur the full impact of a fire. The flat slabs of the roadway deck are subjected to high bending forces, and the integrity of the reinforcing is thus critical to the structure's performance. If the reinforcing steel were subjected to elevated temperatures that resulted in a loss of strength, a catastrophic collapse could occur. The above fire protection measures are recommended for the internal structures. However, for the roadway deck, adding fire cladding on the underside may be required to protect this critical component.

The roadway walls, which mostly transfer axial loads, are subjected to lower fire demand than the roadway ceiling. Reinforcing integrity is not critical to fire performance of the roadway walls. PP fibers, thermally stable aggregate, and a proper concrete cover should provide adequate protection for these elements. The lower roadway deck has the bending reinforcing on the bottom of the slab, and as such, the reinforcing would not be directly exposed to a tunnel fire. The same fire protection proposed for the walls should be adequate for the lower roadway deck.

7.0 Summary and Future Design Recommendations

In summary, the preliminary design concept for the internal structure—with 16-inch main supporting walls, 10-inch upper walls, and 26-inch-thick deck slabs—is expected to meet the static and seismic design requirements. Connections, as conceptually shown in the drawings in Attachment A, would be required between the internal structure and the segmental lining. Additional concepts or modifications to this concept may also meet the design requirements and can be evaluated in subsequent phases of this study.

Currently, the analyses of the racking and inertial effects on the internal structure were performed separately. The forces resulting from these analyses have been combined to estimate the worst-case effect. A full dynamic analysis of the tunnel and internal structure in a single model, with input earthquake motions, would be beneficial in developing further insight into the behavior of these two components and the connections between them. Such an analysis would combine the effects of racking and inertia. This type of analysis should be considered during future design phases.

The current analyses indicate that for the level of displacements in the tunnel lining and internal structure anticipated in an SEE event, the ventilation seals between the internal structure and the lining (see drawings in Attachment A) would most likely fail. Some post-earthquake repairs of the seal should be anticipated for the SEE event. Additional analysis of the type mentioned above should be undertaken to further refine this prediction for the SEE event. The impact of a leak in the ventilation system post-earthquake should be further investigated during future design phases. The seals are expected to survive the FEE event because of the lower level of shaking.

As design progresses, other loadings—such as dead weight of tunnel system components that the internal structure would support and vehicle collision loads—would become better defined and should be included in the internal structure analysis. The performance of the internal structures should also be evaluated when the design fire event is selected. The types of fire-resistant elements/provisions required and their impact on the ventilation systems should be evaluated further.

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9.0 Revision Log

Revision 0	January 24, 2014	Internal Review
Revision 1	February 20, 2014	Metro/Caltrans Review
Revision 2	June 3, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

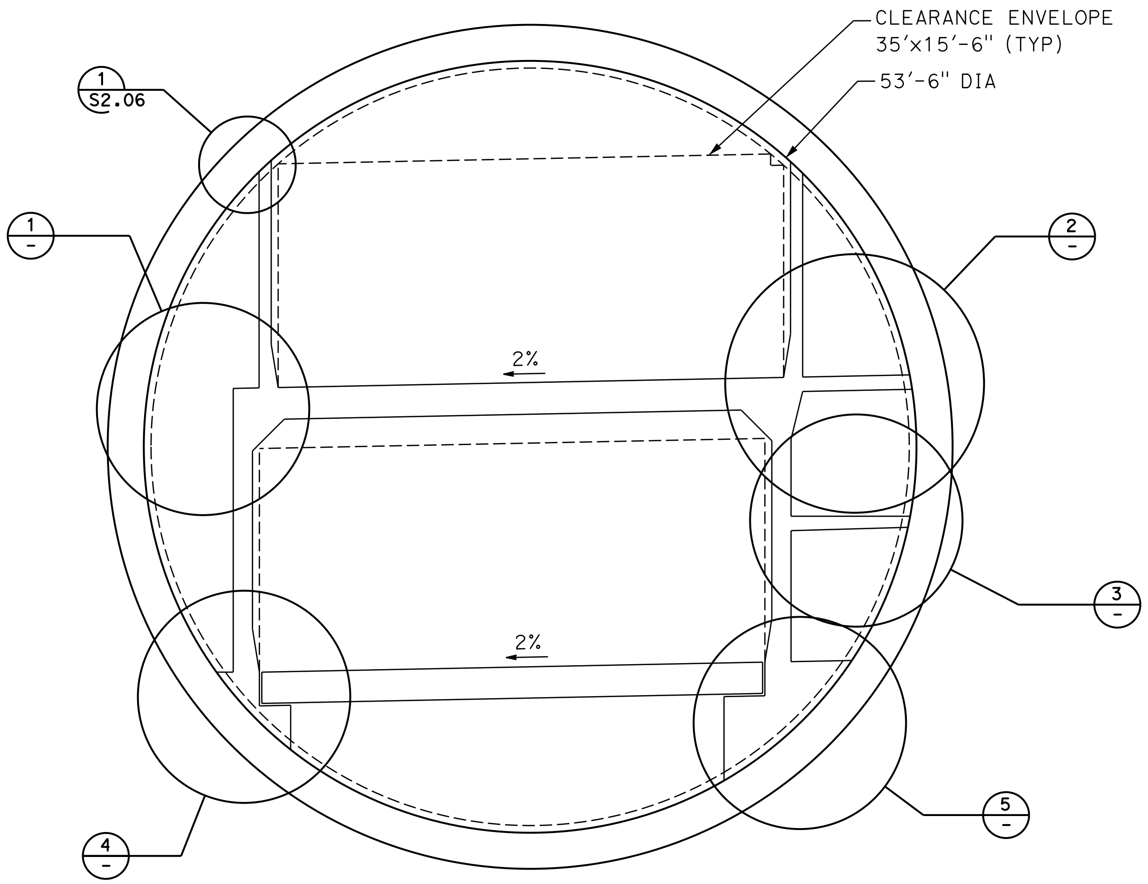
Internal Structure Drawings

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

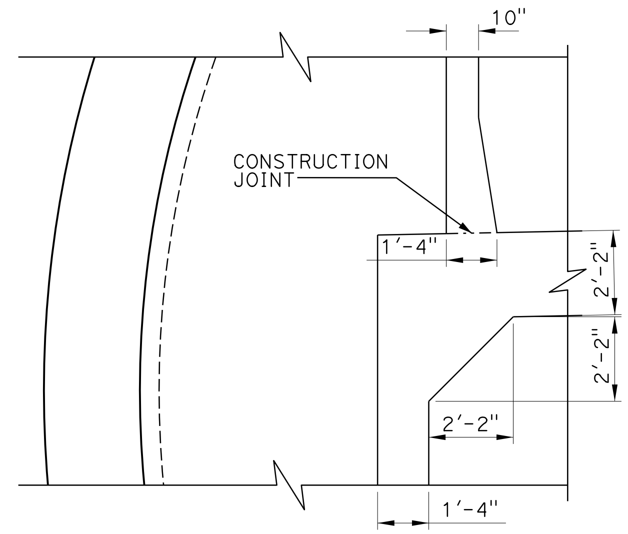
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

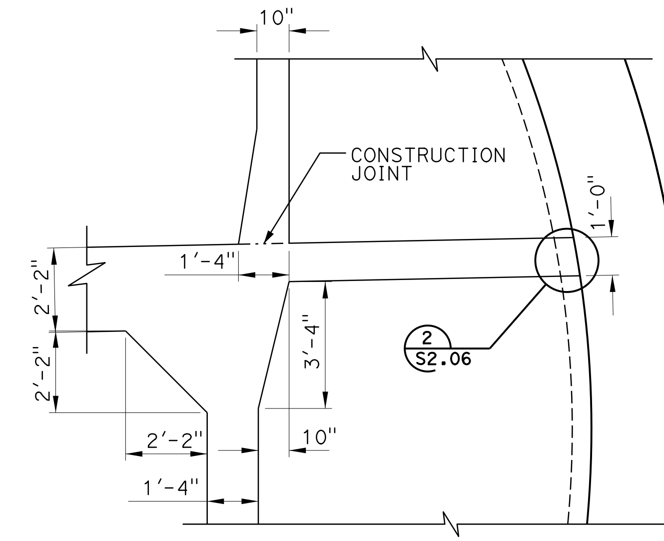
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



TUNNEL SECTION
NTS



DETAIL 1/–
NTS



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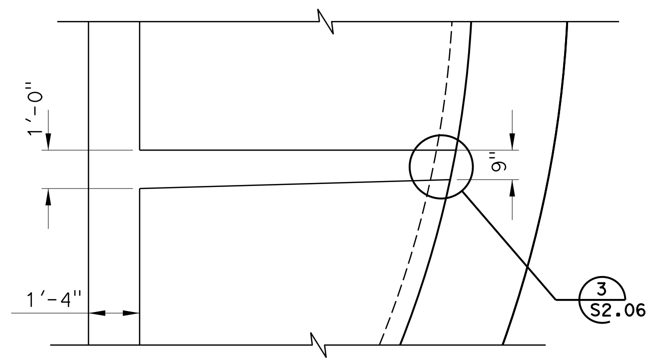

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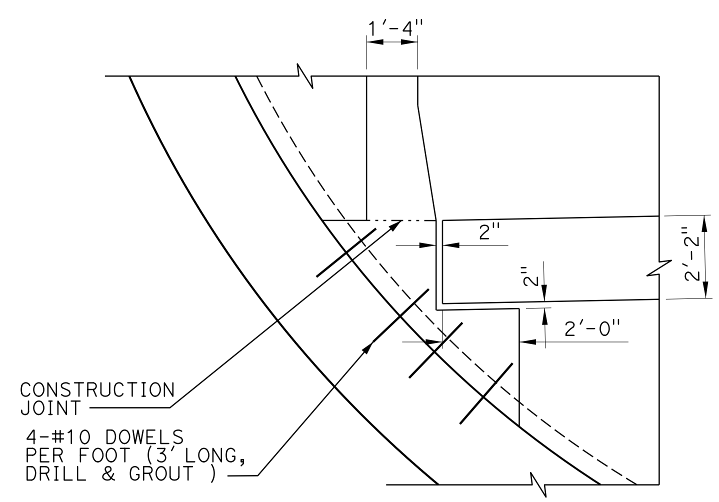

DETAIL 4/–
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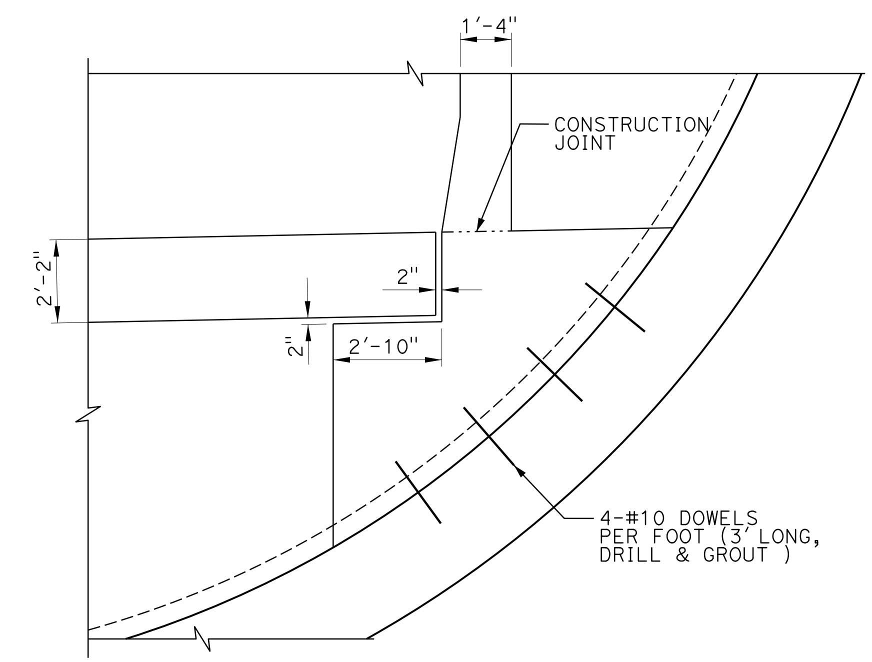


DETAIL 5/–
NTS

NOTE:

1. REFER TO S2.01 FOR DETAILS ON TYPICAL TUNNEL SECTION.

S2.05

DESIGNED BY	J. VANGREUNEN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	S. KLEIN	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

INTERNAL STRUCTURE DETAILS

BRIDGE NO. TBD | UNIT:

SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

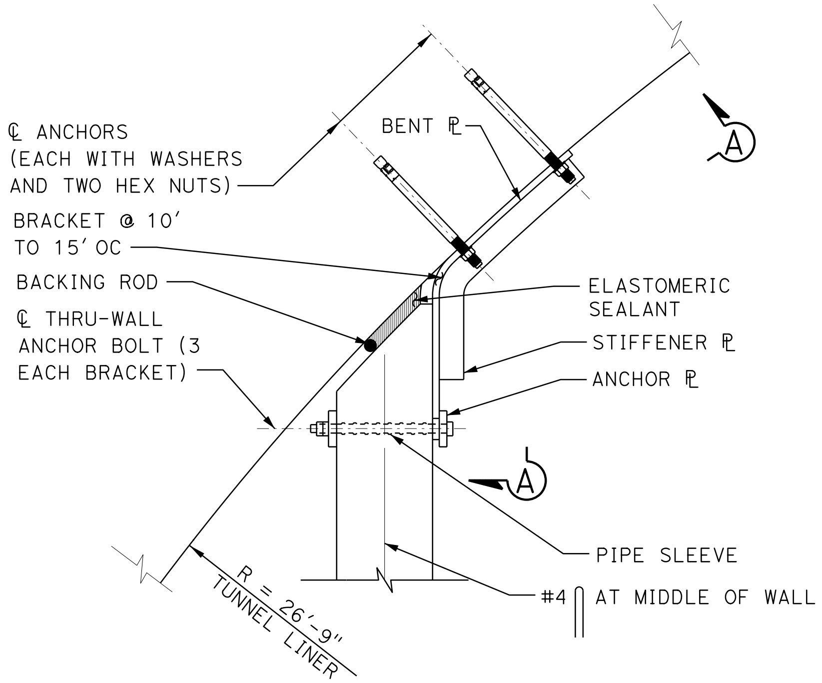
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

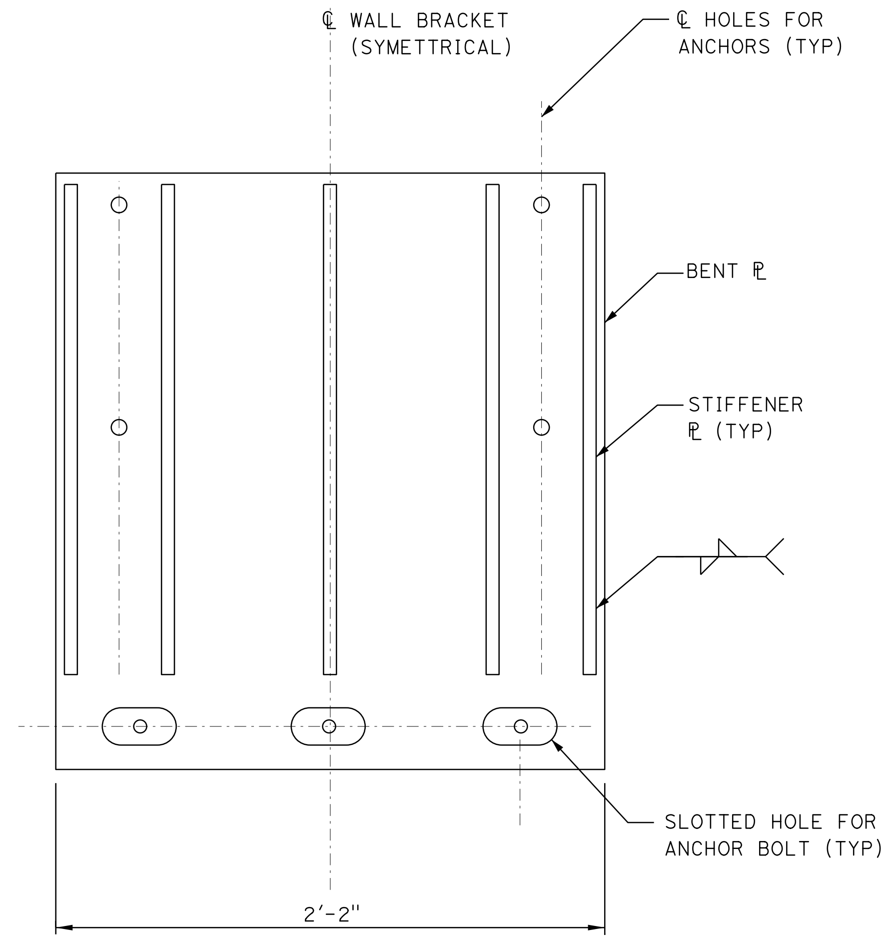
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

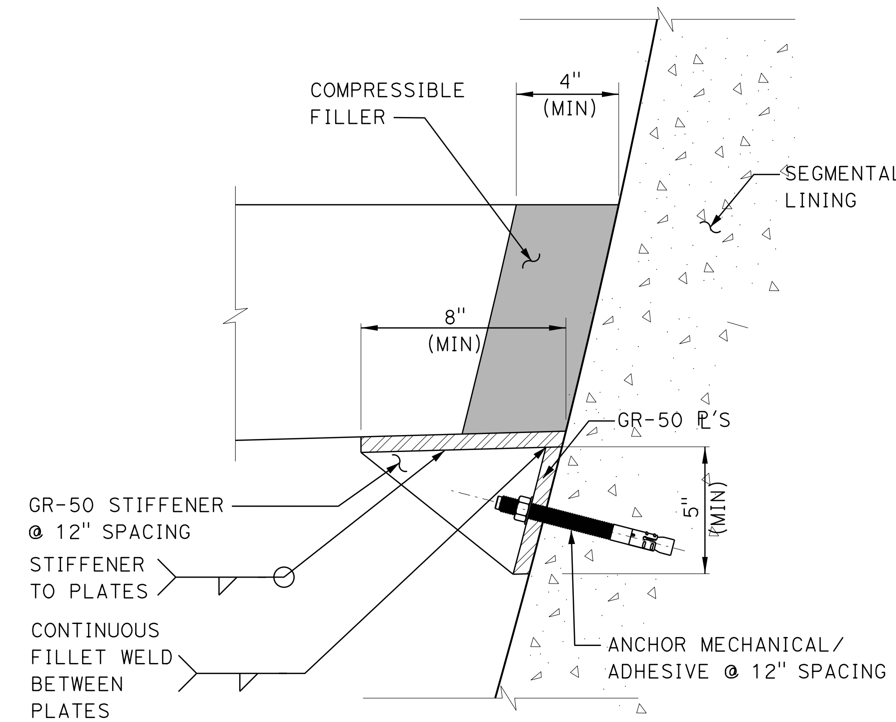
JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



DETAIL 1 / – S2.05
NTS



SECTION A / –
NTS



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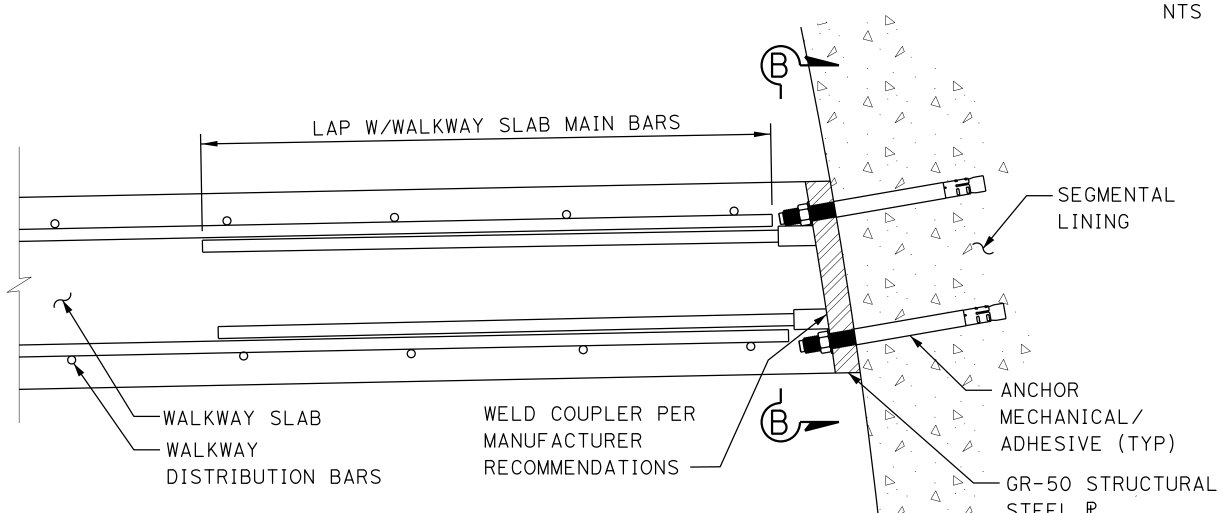

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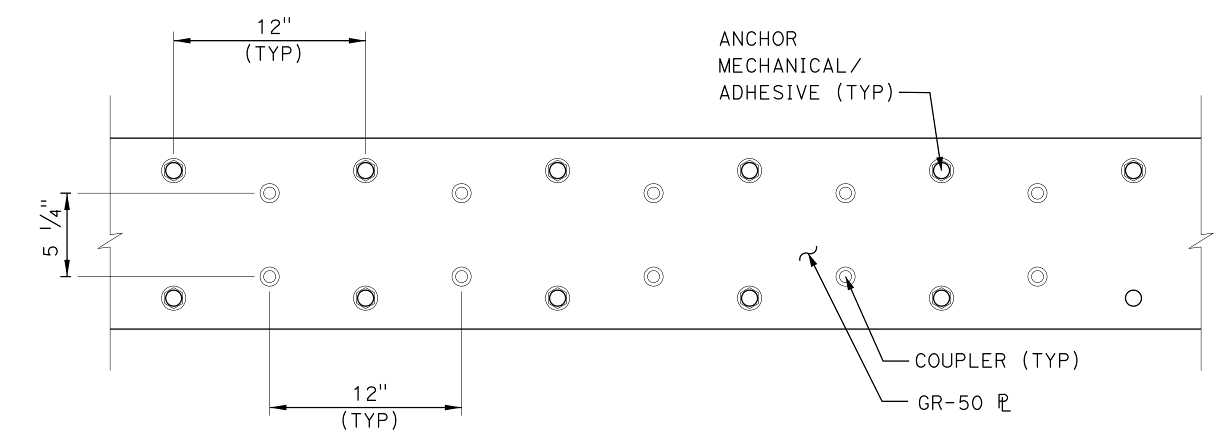


SECTION B
NTS

S2.06

DESIGNED BY	J. YAO/M. HARIHARAN	DATE 9-20-13	X PROJECT ENGINEER	SR 710 NORTH STUDY	
DRAWN BY	J. TOLES	DATE 9-20-13		INTERNAL STRUCTURE DETAILS	
CHECKED BY	J. VANGREUNEN	DATE 9-20-13		BRIDGE NO. TBD	UNIT:
APPROVED	S. DUBNEWYCH	DATE 9-20-13		SCALE: AS SHOWN	PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

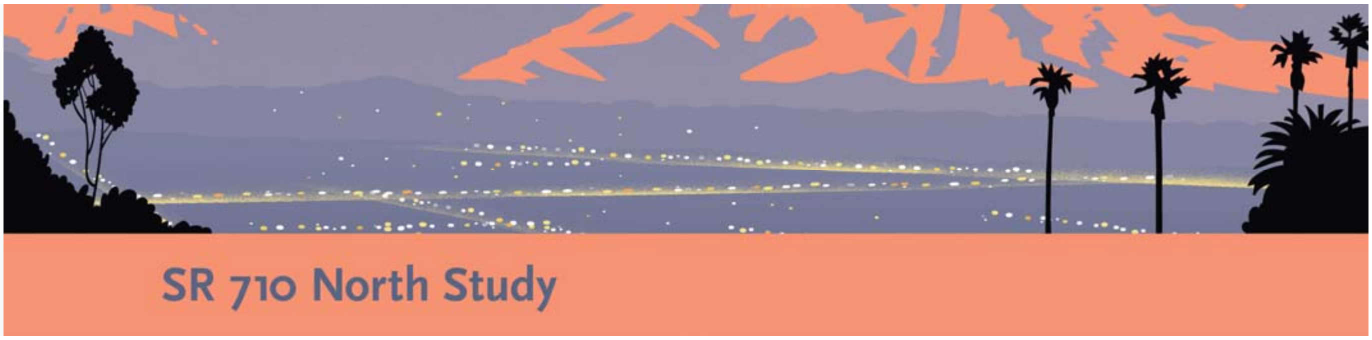
ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S206.dgn

CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:35

Appendix G
TM-5 Evaluation and Control of Ground Movements



TECHNICAL MEMORANDUM 5

Evaluation and Control of Ground Movements

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Scope

This technical memorandum (TM) summarizes methods of determining construction-induced surface ground movements and presents a methodology to determine their potential effects along the proposed Freeway Tunnel and LRT Alternatives. The tunnels are expected be excavated with tunnel boring machines (TBMs) and the portals, from which the TBMs would be launched, are expected to be open excavations. Additionally, along the LRT alignment, cut-and-cover stations would be excavated and for the twin-bore alternatives, cross passages would be spaced along the alignment, which connect the two bored tunnels.

A preliminary analysis on excavation-induced ground movements has been performed using empirical methods to determine the extents of the zone of potential excavation-induced ground movement influence to support the environmental documentation for this study. This analysis uses conservative assumptions to determine the ground movements, and it is expected ongoing improvements in tunneling technology should result in smaller movements than predicted in this report. Additionally, ground control and mitigation measures that can be used to further control and monitor ground movement during excavation are explained.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Project Elements

2.1 Freeway Tunnel Alternative

The Freeway Tunnel Alternative would consist of the following major elements which could cause excavation-induced ground movements.

2.1.1 Bored Tunnels. The Freeway Tunnel Alternative consists of either a single- or twin-bore tunnel 22,340 feet in length. The excavated diameter of the tunnel, which is slightly larger than the outside diameter of the tunnel lining, is anticipated to be approximately 60 feet (JA, 2014d). Refer to *Tunnel Excavation Methods* (JA, 2014a) for more information about the excavation of the bored tunnels.

2.1.2 Emergency Vehicle Cross Passages. In addition to the bored tunnels of the Freeway Tunnel Alternative, six pairs of emergency vehicle cross passages (twelve total) would be included along the twin-bore variation to connect each tunnel level to the adjacent tunnel. Vehicle cross passages would be used in the event of an emergency to allow for first responders to cross from one tunnel bore to the other. The two tunnels are assumed to have a clear distance between them of approximately one tunnel diameter (i.e., approximately 60 feet). These cross passages would be roughly circular in shape and approximately 29 feet in diameter. They would be excavated using the Sequential Excavation Method (SEM) after the bored tunnels are excavated. Refer to *Tunnel Excavation Methods* (JA, 2014a) for more information about the excavation of the cross passages.

2.1.3 Construction Portals. Construction portals at the north and south ends of the tunneled portion of the Freeway Tunnel Alternative would be excavated prior to the initiation of tunneling operations. These portals would be used to launch the TBM(s) and support construction activities. The roadway ramps down from the ground surface within the portal to gain cover for launching the TBM. The north portal is expected to be approximately 100 feet deep, measured at the headwall (where the tunnel starts), 240 feet wide, and 500 feet long. The south portal is expected to be approximately 130 feet deep, measured at the headwall, 230 feet wide and 500 feet long. The portal excavations gradually increase in depth from the ground surface to the headwall. The portals for the single-bore variation are similar in shape, but the width is smaller – approximately 110 feet less. Although the portal excavations are longer than 500 feet, the area beyond this length of the excavation will be designed by CH2M HILL in their assessments for the design of the permanent works. Refer to *Preliminary Design Concepts for the Freeway Portal Excavation Support Systems* (JA, 2014b) for more information about the excavation and support of the portals.

2.2 LRT Alternative

The LRT Alternative would consist of the following major elements which could cause excavation-induced ground movements.

2.2.1 Bored Tunnels. The LRT Alternative includes approximately 21,180 feet of twin-bore tunnel that is expected to be excavated with two TBMs. The alternative also includes four underground stations that would be excavated using cut-and-cover techniques in advance of the TBM arrival at each location. Along the alignment, the TBMs are expected to break into the south end of each station, be walked through the station excavation, and recommence tunneling at the north end of the station.

The excavated diameter of the LRT bored tunnels, which is slightly larger than the outside diameter of the tunnel lining, is expected to be just over 21.5 feet (JA, 2014d). Two TBMs would be used to excavate this alternative; one for each tunnel bore. They would be launched from a portal on the south end of the alignment and terminate at a station at the north end. Refer to *Tunnel Excavation Methods* (JA, 2014a) for more information about the excavation of the bored tunnels.

2.2.2 Pedestrian Cross Passages. In addition to the bored tunnels, twenty-six pedestrian cross passages would be excavated along the LRT tunnel alignment to connect the two tunnels for emergency egress. The two tunnels are assumed to have a clear distance between them of approximately one tunnel diameter. These cross passages, which are oval- shaped and have an insider diameter of approximately 12 feet wide by 14 feet high would be excavated using the Sequential Excavation Method (SEM) and would be used for emergency egress only and are spaced at roughly 750- to 800-foot intervals along the bored tunnels. Refer to *Tunnel Excavation Methods* (JA, 2014a) for more information about the excavation of the cross passages.

2.2.3 Construction Portal. A construction portal at the south end of the tunneled portion of the LRT alternative would be excavated in advance of tunneling operations. This portal would be used to launch the two TBMs and support construction activities. The portal is expected to be approximately 50 feet deep, measured at the headwall, approximately 70 feet wide, and 350 feet long. The portal is deepest at the headwall and becomes shallower with distance away from the headwall as the base of the portal slopes upwards to meet the existing grade. Refer to *Preliminary Design Concepts for the LRT Portal and Station Excavation Support Systems* (JA, 2014c) for more information about the excavation and support of the portal.

2.2.4 Stations. Four cut-and-cover stations are included in the LRT alignment. From south to north, these are the Alhambra Station, Huntington Station, South Pasadena Station, and Fillmore Station, where the alignment terminates and meets the existing Gold Line.

- The Alhambra Station excavation is approximately 410 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet.
- The Huntington Station excavation is approximately 825 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet. The long length of this station allows space for a track crossover.
- The South Pasadena Station is approximately 410 feet long and 60 feet wide in plan. The total excavation depth to the bottom would be approximately 80 to 90 feet.
- The Fillmore Station is approximately 1300 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet. The long length of this station allows for a track crossover and tail tracks. It is assumed that the entire excavation would be performed by cut-and-cover methods, and that the TBMs would be retrieved from this excavation.

Refer to *Preliminary Design Concepts for the LRT Portal and Station Excavation Support Systems* (JA, 2014c) for more information about the excavation and support of the stations.

3 Anticipated Geologic Conditions

The ground movement assessments were based on the Preliminary Geotechnical Report provided by CH2M HILL (2014). A preliminary geologic profile along both alignments is provided in Attachment A.

The subsurface conditions along the alignments generally consist of alluvium; weak rocks of the Puente, Topanga, and Fernando Formations; and Basement Complex Rocks. The alluvium consists of interbedded lenses and/or discontinuous layers of fine-grained soil (clay and silt) and coarse grained materials (sand and gravel) that include a wide range of soil types. The Topanga Formation includes a lower siltstone member, a middle sandstone member, and an upper conglomerate/breccia member. The Puente Formation is comprised predominantly of sandstone, shale, diatomaceous siltstone/shale, and siltstone members. The Fernando Formation consists primarily of weak, dark gray to black, massive (unbedded), claystone and siltstone. The Basement Complex Rocks comprise a wide suite of lithologies, including diorite, monzonite, quartz diorite, quartz monzonite, and gneissic diorite.

Groundwater depths along the alignments range from as shallow as 10 feet to as deep as 175 feet below the ground surface. Water inflows into the tunnels could occur while excavating below the groundwater table in the saturated alluvium. The rock formations along the alignment are generally considered non-water-bearing, however, seepage may occur within sandstone beds and faulted and fractured zones. The faults along these alignments, especially the Raymond Fault, are known groundwater barriers. At the Raymond Fault, groundwater levels on the north side are significantly higher than the levels on the south side. Refer to the geologic profiles in Attachment A for more information on the expected groundwater levels.

4 Methodology

4.1 Bored Tunnels

The bored tunnel sections are expected to be excavated with pressurized closed-face TBMs such as an earth-pressure balance (EPB) tunnel boring machine or a slurry pressure balance tunnel boring machine (Slurry TBM) based on the geologic and groundwater conditions along the tunnel alignments. Over the past 10 to 15 years in the U.S., pressurized-face machines have been used to reduce the risk of uncontrollable ground loss during excavation, as well as to minimize overall loss of ground, and subsequent ground movements due to tunneling, as compared to the use of open-face tunnel excavation methods. These TBMs provide an immediate support of the excavated ground and allow the timely response of any potential loss of ground. The required use of a pressurized face tunnel boring machine is one way to help limit surface ground movements during excavation; ground control measures are discussed in greater detail later in this TM.

The ground movements associated with tunnel excavation can be estimated using either semi-empirical methods or numerical modeling methods that use software programs such as PLAXIS (PLAXIS BV, 2010) or FLAC (Fast Lagrangian Analysis of Continua). Generally, semi-empirical methods are simpler, faster, require less-detailed understanding of the physical properties of the ground, and provide direct estimates of slope and curvature of the settlement trough. The numerical modeling methods are more sophisticated, provide more rigorous analysis for complex problems, and allow more in-depth understanding of soil-structure interaction. For these preliminary design evaluations, semi-empirical methods were used for ground movement estimates.

4.1.1 Ground Movements. Ground movements may occur in three directions:

- x-direction: lateral direction perpendicular to the tunnel alignment. Ground movements (lateral) in this direction are designated S_x and the offset distance from the tunnel centerline is given as x.
- y-direction: parallel direction to the tunnel alignment. Ground movements in this direction are designated S_y and the offset distance from the tunnel is given as y.

- z-direction: direction perpendicular to the ground surface. Ground movements in this direction are designated S_z and the vertical distance from the tunnel springline to any point above the tunnel is given as z.

The focus of these evaluations is on the ground movements in the z-direction, which is the vertical ground movement (also referred to as settlement). The induced ground movements transverse to the proposed tunnels are estimated using the semi-empirical method that was originally proposed by Peck and Schmidt (1969), and subsequently updated by O'Reilly and New (1982). This method assumes that the shape of the settlement trough above a single tunnel follows a Gaussian distribution and that the volume of the settlement trough is equal to the total volume of lost ground during tunneling. The total vertical ground movements caused by two tunnels are the sum of the ground movements caused by each individual tunnel, assuming the ground movements associated with each bore are independent of each other and can be superposed to estimate the combined trough due to both tunnels. Figure 1 shows a typical surface settlement trough above two tunnels.

The shape of the settlement trough over a single tunnel is characterized by three main parameters: depth to the tunnel springline (*z*), the ground loss (V_l), and horizontal distance from the tunnel centerline to the point of inflection of the settlement profile curve (*i*). In this report, the depth z is the vertical distance from the building or structure's foundation bottom, or utility springline, to the proposed tunnel springline at the location of the structure under consideration. Ground loss is defined as the volume of all ground movements taking place around a tunnel and is usually characterized as a percentage of the excavated area. The settlements caused by a single tunnel excavation are predicted using the following equations:

$$S_{z(x)} = S_{z,\max} . e^{\left(-\frac{x^2}{2i^2}\right)}$$

$$S_{z,\max} = 0.313(V_l)\left(\frac{D^2}{i}\right)$$

$$i = K.z$$

where:

$S_{z(x)}$	=	settlement at location x from tunnel centerline
x	=	horizontal distance from the tunnel centerline
z	=	vertical distance from the tunnel springline to the point of analysis
i	=	horizontal distance from the tunnel centerline to the point of inflection on the settlement profile curve
D	=	excavated tunnel diameter
V_l	=	average ground loss, usually presented as percentage of excavated area
K	=	trough width factor

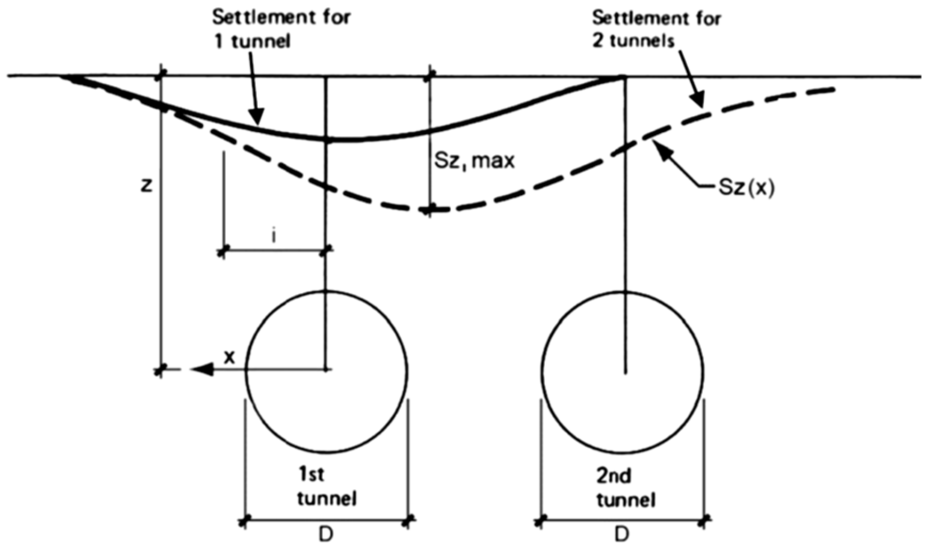


Figure 1: Typical Surface Settlement above Two Tunnels

Lateral ground movements in the x-direction can result in tensile strain in structures and utilities; therefore, predictions of these lateral ground movements would also be performed. According to O'Reilly and New (1982), the lateral ground movements can be calculated using the equation below, assuming that the resultant vectors of ground movements are directed towards the tunnel axis.

$$S_{x(x)} = S_{z(x)} * (x/z)$$

where x = horizontal distance from the tunnel centerline
z = vertical distance from the tunnel springline to the point of analysis

Figure 2 shows the appropriate shape and key parameters for the lateral ground movement profile.

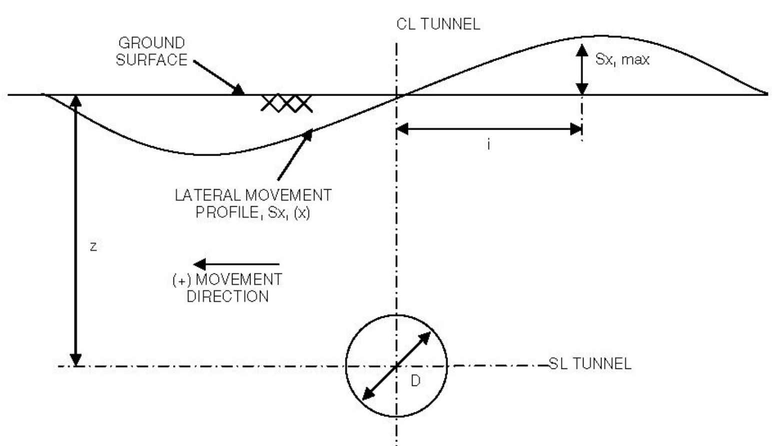


Figure 2: Lateral Ground Movement Profile Above a Tunnel

The ground movements parallel to the tunnel excavation (y-direction) are generally on the same order (and usually less than) settlement slopes in the x-direction, but considered less critical to the buildings and structures than those transverse to the tunnel excavation, because the impact of longitudinal settlement is typically transitory, leveling off as the tunnel passes. Ground movement in this direction should be considered in further phases of this study.

4.1.2 Estimation of Ground Loss. The ground loss that occurs in soft ground formations is a function of several factors, including expected ground conditions, presence of groundwater, construction means and methods, and overall workmanship. Ground loss is often reported in the percentage of ground lost in the excavation, or the volume loss. Ground loss during excavation is typically caused by a combination of three general sources: face losses, shield losses, and tail losses; settlement can also continue after excavation is complete.

- Face Losses: ground loss at the heading of the tunnel, often caused stress changes in the ground and over-excavation of material due to the presence of boulders or hard inclusions. Face losses can be controlled and mitigated by pressurized-face TBM technology.
- Shield Losses: ground loss at the shield of the TBM, often caused by the overcut of the cutterhead and intrusion of surrounding ground into the overcut annulus. Steering adjustments to either excavate curves or to make steering corrections can increase the overcut of the machine and the volume of this annular space. Shield losses can be reduced by limiting the overcut of the TBM and also by injecting bentonite along the shield and maintaining a positive pressure in the annulus or similar measures.
- Tail Void Losses: ground loss which occurs as the shield passes, often caused by intrusion of surrounding material into the annulus between the outside skin of the shield and the outside surface of the primary support. Tail void losses can be controlled by requiring grouting through the tail shield concurrent with TBM advance, which would limit the potential for ground intrusion into the gap. The annular gap for the freeway tunnels is expected to be large due shield geometry on the large-diameter TBM.
- Post Excavation Settlement: when the segmental linings leave the shield, they deform due to their self-weight and the external loads imposed by the ground (or overburden pressure) and grouting pressures. Deflection of the lining may result in additional vertical ground settlement. Additionally, consolidation settlements could occur after the excavation phase has passed, which are caused by the changes in pore pressures over time. These are not typically as significant as the other ground movement components.

Ground Loss - Freeway Tunnels

To obtain a reliable estimate of volume losses for such large-diameter (>30 feet) EPB/slurry TBM tunnels, several case studies of excavation by large-diameter TBM were examined to determine the degree of settlement control that was achieved given the typical large annular gap. Volume losses and ground convergences as directly reported or back-calculated from these case histories were used as the basis for the expected volume losses for the freeway tunnels. The case histories and volume loss values are shown on Table 1.

It can be seen from Table 1 that volume losses larger than about 0.5% were in portal zones (i.e, in the areas where the TBM crew is on a learning curve and where ground cover is typically shallow) or where TBM operational procedures were not particularly suited to the ground encountered. Once out of the portal zones and with properly controlled machine operations, volume losses were lower and typically ranged from range from 0.25% to 0.5%. Based on this, a volume loss of 0.5% was adopted for the alluvium and 0.25% for the weak sedimentary rock formations for this study as an average along the freeway alignments.

Further, volume losses were also assumed to be 0.5% for mixed face conditions, which is when the face of the tunnel excavation is expected to be in both soil and rock. Also, when there is less than one-half diameter of rock cover over the tunnel crown, the volume loss is assumed to be that of soil (or 0.5%). Since the geologic profile currently available does not distinguish between fresh and weathered rock, the purpose of this half-diameter limit is to provide estimates that are on the conservative side. It is not unusual for mixed face conditions to results in larger amounts of loss of ground.

Ground Loss - LRT Tunnels

The LRT tunnels are of significantly smaller diameter than the freeway tunnels, and it is not expected that the volume losses for the LRT tunnels would be any greater than the freeway tunnels, using shield bentonite injection and tail void grouting from an EPB or slurry TBM. Two recent case histories with similarly-sized TBMs include the Sound Transit U230 contract in Seattle, Washington, and more locally, the Los Angeles Metro’s Gold Line Eastside Extension (MGLEE).

On the U230 contract for Sound Transit, two EPBMs were used to excavate through glacially overconsolicated materials and they achieved back-calculated ground losses of less than 0.2% (Swartz et al., 2013). Additionally, volume losses of less than 0.3% were achieved with the MGLEE’s EPBMs through a mix of both granular and cohesive materials above and below the groundwater table, based on back-calculations (Choueiry et al., 2007 and PB, 2011). Based on these case histories, the above volume losses for the freeway tunnels are also appropriate for use on the LRT tunnels.

Table 1: Large-Diameter TBM Volume Loss Case Histories

Project	Location	Year(s) Built	Machine Type	Machine Dia.		# of Tunnels	Gap		Geology	Cover			Max. Settlement		Vol Loss	Remarks
				m	ft		mm	inch		m	ft	diameters	mm	inch	%	
Alaskan Way - SR 99	Seattle, USA	2013-2015?	EPB	17.48	57.33	1	205.6	8.5	Glacial Sands, Silts , Clays, Stiff, Overconsolidated. High groundwater table	30 to 46	100 to 150	2 to 3	-	-	-	1) Currently under construction
Shanghai Yangtze River Tunnel	Shanghai, China	2009	Slurry/Mixshield	15.43	50.61	2	215	8.5	Soft clay and silty clay, liquified soil and quicksand. High groundwater table.	6.3 to 24	20 to 79	0.5 to 1.5	40 to 60	1.6 to 2.4	0.7	1) Highest recorded settlement occurred near portals with shallow ground cover.
SMART	Kuala Lumpur, Malaysia	2003-2007	Slurry/Mixshield	13.21	43.33	1	200	7.9	Mix: Karstic limestone, quarternary alluvium. High groundwater table	12 to 20	39 to 66	1 to 1.5	-	-	-	1) Ground settlement was generally minimal. In extreme cases the bentonite cake could not develop due to low-strength, unconsolidated loose soil, and the face could not be supported. This phenomena caused face or crown failure and excessive local settlement on the surface.
Hubertus Tunnel	The Hague, Netherlands	2004-2008	Slurry/Mixshield	10.53	34.54	2	115	4.5	Fine Dense Dune Sands, Soft Clays and Silts. High groundwater table	5 to 15	16 to 49	0.5 to 1.5	7 to 10	0.3 to 0.4	0.18-0.25	1) One sink hole at portal at the end of the drive. 2) Settlements up to 30mm during initial launch, but within construction site
				10.63 with overcut	34.88		165	6.5								3) Settlements beyond portals controlled in the 7 to 10 mm range 4) Internal Dia. = 9.4m, Segmental lining thickness= 450 mm
Heathrow Airside Road Tunnel	London, England	2004	Dual Mode - EPB and Compresed Air	9.18	30.11	2	190	7.5	Gravels over stiff, competent London Clay. High groundwater table	5 to 16	16 to 53	0.5 to 1.5	5 to 20	0.2 to 0.8	0.3	1) 90% of recorded surface settlement was less than 15 mm, 100% was less than 20 mm.
M30, North and South Bypass Tunnels	Madrid, Spain	2005-2008	EPB	15.1	49.53	2	225	8.9	Alluvial deposits, fissured hard clay with gypsum layers. High groundwater table	6 to 65	20 to 213	0.4 to 4	-	-	0.1 to 0.4	1) Settlement values not available but smaller than the 15.2m machine
				15.2	49.86		275	10.8					5 to 10	0.2 to 0.4		1) Portal Zones (very shallow cover) had compensation grouting and mortar pile improvement
Barcelona Metro Line 9	Barcelona, Spain	2002-2014?	EPB	12.06	39.56	1	210 to 240	8.3 to 9.5	Soft soils, stiff overconsolidated clay	10 to 24	33 to 80	1 to 2	-	-	-	
			Dual Mode - EPB and open mode	11.95	39.2				Sands, clay, and silts overlying gravels with sands				-	-	0.2 to 0.6	
									Heterogeneous mixed face with soft soils to weak to hard rock				-	-	0.7 to 1.0	1) Issues included segment stabilization behind shield (solved with lower-slump grout mix) and rock spalling and fracturing due to decompression at excavation face (solved by closing cutting wheel).
			EPB	9.4	30.8				Submerged fine silty sands and sandy silts				35 to 45	1.4 to 1.8	0.4 to 0.8	1) Higher settlements observed before exercising more stringent excavation control (settlement due to shield conicity alone apporx. 10mm)
													10 to 15	0.4 to 0.6	0.3 to 0.5	1) Mix revised to increase cement and reduce filler content. Lower settlement after more stringent excavation control measures adopted
Sao Paulo Metro Line 4	Sao Paulo, Brazil	2002-2011	EPB	9.5	31.2	2	-	-	Soil from altered gneiss, interbedded clay and sandy clay with gravel, interbedded stiff to hard clay with sands	25	80	2	6 (ave)	0.3	<0.4	

4.1.3 Settlement Trough Width. The horizontal distance from the tunnel centerline to the inflection point *i* is characterized by a trough width factor *K* and the depth to the tunnel springline, *z*. The trough width factor *K* is a function of ground conditions. The ranges of recommended *K* values are 0.2 to 0.3 for sands above the groundwater table, 0.4 to 0.7 for hard to soft clays (O'Reilly and New, 1982), and 0.2 to 0.6 for sands below the groundwater level, depending on the ratio of tunnel depth to tunnel diameter (Peck, 1969). The *K* values of different soil types selected for this study are shown Tables 2 and 3. The composite trough width parameter *i* of *N* soil layers above the tunnel springline, each of thickness z_N, is calculated using the following equation recommended by O'Reilly and New (1982).

$$i = K_1 z_1 + K_2 z_2 + \ldots + K_N z_N$$

Table 2: K Values for Different Soil Types

Soil Type	K
Soft Silty Clay	0.7
Firm Clay	0.6
Stiff Clay	0.5
Hard Clay	0.4
Silty Sand (above water table)	0.3
Sand, Gravel (above water table)	0.2

From O'Reilly and New (1982).

Table 3: K Values for Different Soil Types

Soil Type	K
Sand, Gravel (below water table, Z/D≤2)	0.6
Sand, Gravel (below water table, 2<Z/D<4)	0.5
Sand, Gravel (below water table, 4<Z/D<6)	0.4

Interpreted From Peck (1969).

Due to the nature of the Gaussian curve used to estimate the trough width, there is no point at which the estimated settlement actually equals zero. Instead, the curve flattens and approaches zero infinitely while never reaching it. Therefore, to approximate the trough width at which the curve is effectively zero in reality, it is necessary to use a trough width estimate, such as the *w* term defined below, or a limiting settlement value below which settlement effects are negligible. This concept is illustrated in Figure 3.

For a twin-bore tunnel, it is assumed that the settlement troughs are additive, resulting in a shape similar to that shown in Figure 4. The trough width here is equal to *2.5i* outward from each tunnel centerline (or the distance w on each side; see Figure 4) plus the spacing between the two tunnels. The K values used for this study are presented in Table 4. The values were chosen based on the geology; the rock formations were given a value similar to those as hard clay.

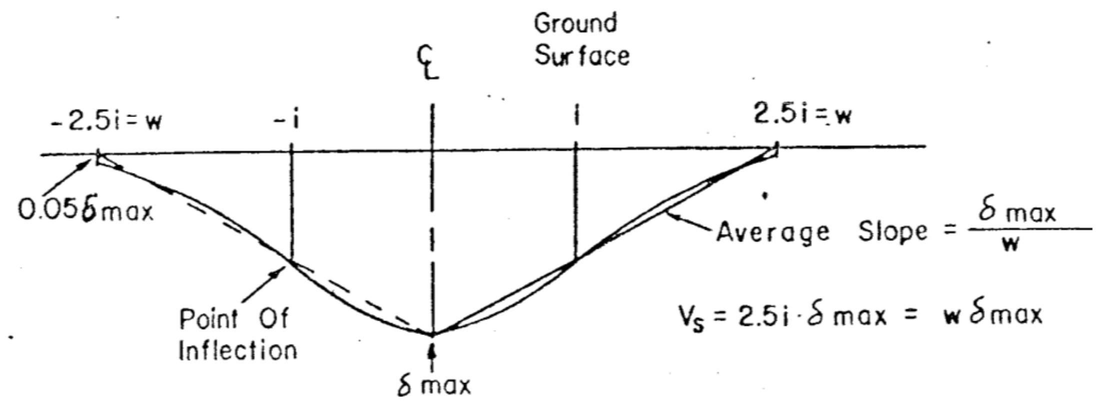


Figure 3: Width of Settlement Trough (Cording and Hansmire, 1975)

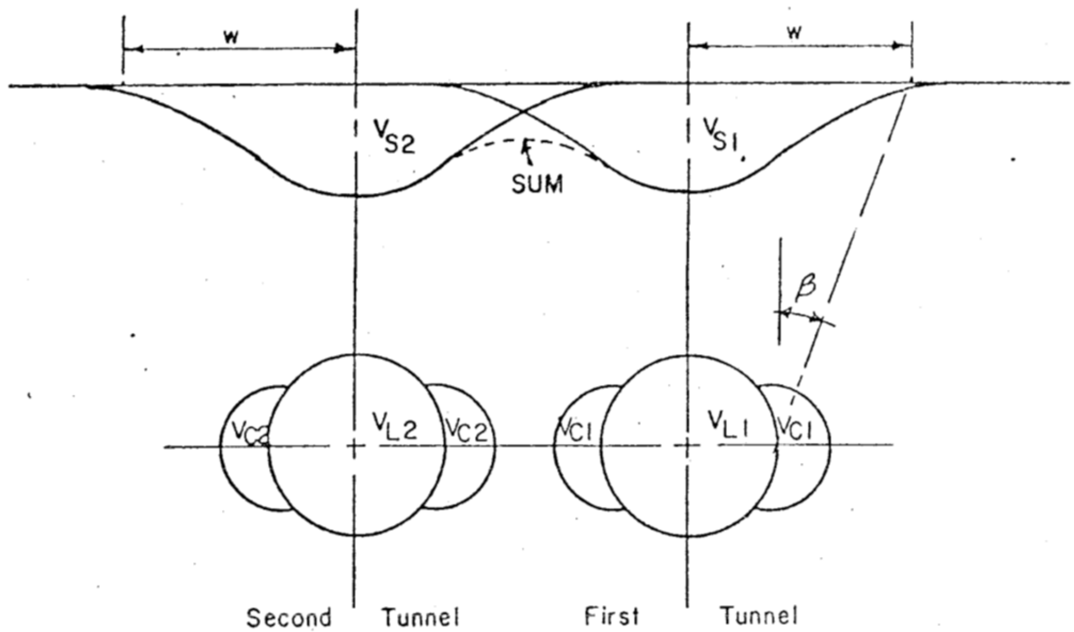


Figure 4: Settlement Trough for Twin-Bore Tunnels (Cording and Hansmire, 1975)

Table 4: K Values Used for SR-710 Geology

Soil Type	K
Alluvium Above Groundwater	0.3
Alluvium Below Groundwater	0.4 - 0.6[a]
Rock Formations (Topanga, Puente, Fernando, Basement Complex)	0.4

[a] K value depends on tunnel depth-to-diameter ratio as shown in Table 3

4.2 Cut-and-Cover Excavations

The TBM launch portals for both the Freeway Tunnel and LRT Alternatives, as well as the LRT stations, are assumed to be constructed using conventional cut-and-cover construction. Based on the preliminary design considerations, the temporary excavations for the freeway tunnel portals are expected to be supported by either slurry walls or soldier piles and lagging with ground anchors. The LRT stations and portals are supported using soldier piles and lagging, also with ground anchors and/or cross struts.

Ground movements associated with these excavations can be estimated using semi-empirical methods or with numerical methods from commercially available software. The simplified methods can provide estimations of ground movements with limited input parameters. However, the applicability of these methods is limited to the situations where the underlying assumptions are satisfied.

In this study, simplified methods are used for the preliminary assessment of building susceptibility to ground movements. The method used for this project estimates the movement of excavation support walls resulting from adjacent excavation and support. Data from Clough and O'Rourke (1990), the Los Angeles Metro Red Line Segment 2 Hollywood/Vine and Hollywood/Western Stations (Smirnoff, et al., 1997), the Capitol Hill Station excavation and the University of Washington Station excavations and other data (Long, 2001) from the Seattle, USA, area were used. The Seattle area excavations were in stiff/dense and overconsolidated soils or "competent soils." The Puente formation at the south portal and the very dense Alluvium at the north portal are also competent soils and therefore the Seattle data provides for a useful comparison.

Figures 5 and 6 indicate that for stiff walls, the horizontal and vertical movements are approximately 0.2% to 0.15% of wall height, respectively and that flexible walls have horizontal and vertical movements up to 0.5% of wall height. Excavations for the Hollywood/Vine and Hollywood/Western Stations, which were 60 and 70 foot deep excavations in Alluvium, did not exhibit a lateral movement of more than 3/8 inch. The lateral movement to wall height ratio would then be 0.05% to 0.04%.

The University of Washington Station excavation is a slurry wall supported excavation 122 feet in height and the Capitol Hill Station excavation is a soldier pile and lagging wall excavation approximately 90 feet deep. The maximum lateral movements shown by these excavations were on the order of 0.75 to 1 inch; this translates into a lateral movement to wall height ratio of approximately 0.07%.

Figure 7 indicates that the zone of influence is predicted to be 2 times wall height for granular soils and 3 times wall height for cohesive soils. Several of the excavations in the Seattle area, including the University of Washington and Capitol Hill Station excavations do not exhibit a zone of influence more than about 1Horizontal: 1Vertical (1H:1V). This leads to a preliminary conclusion that for flexible or stiff walls in competent soils, the Clough and O'Rourke data is perhaps conservative with respect to settlements, lateral movements and zone of influence. Lateral movements on the order of 0.1% of wall height and a zone of influence of approximately 1H:1V are perhaps more appropriate. Additionally, as stated in Cording et al. (2010), both the wall stiffness and constructed methods/practices can be specified to control ground movements at the source during the excavation. These conclusions will be further verified during subsequent design phases with additional data and numerical modeling of the excavations.

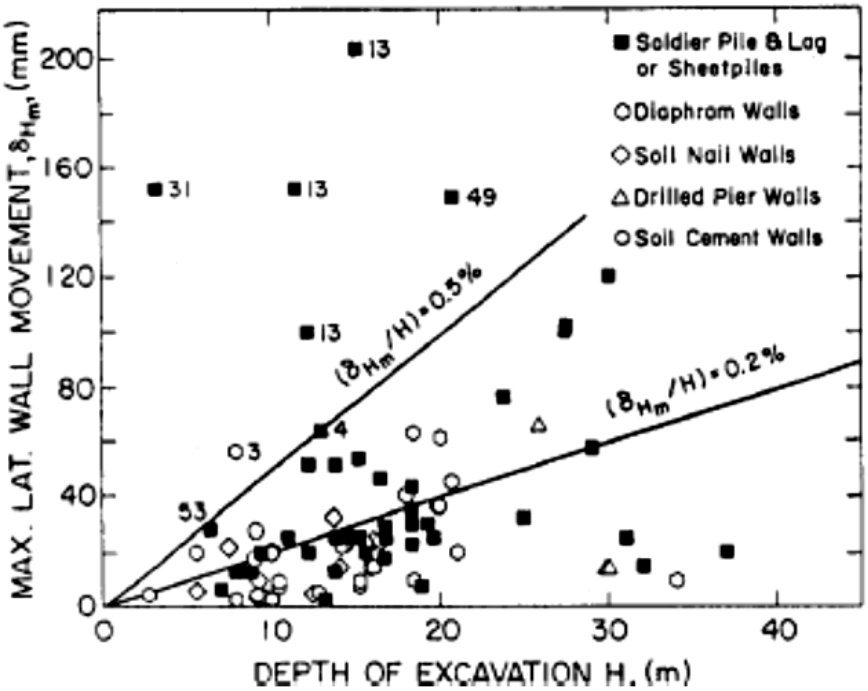


Figure 5: Observed Lateral Movements for Retaining Walls in Stiff Clays, Residual Soils and Sands (Clough and O'Rourke, 1990)

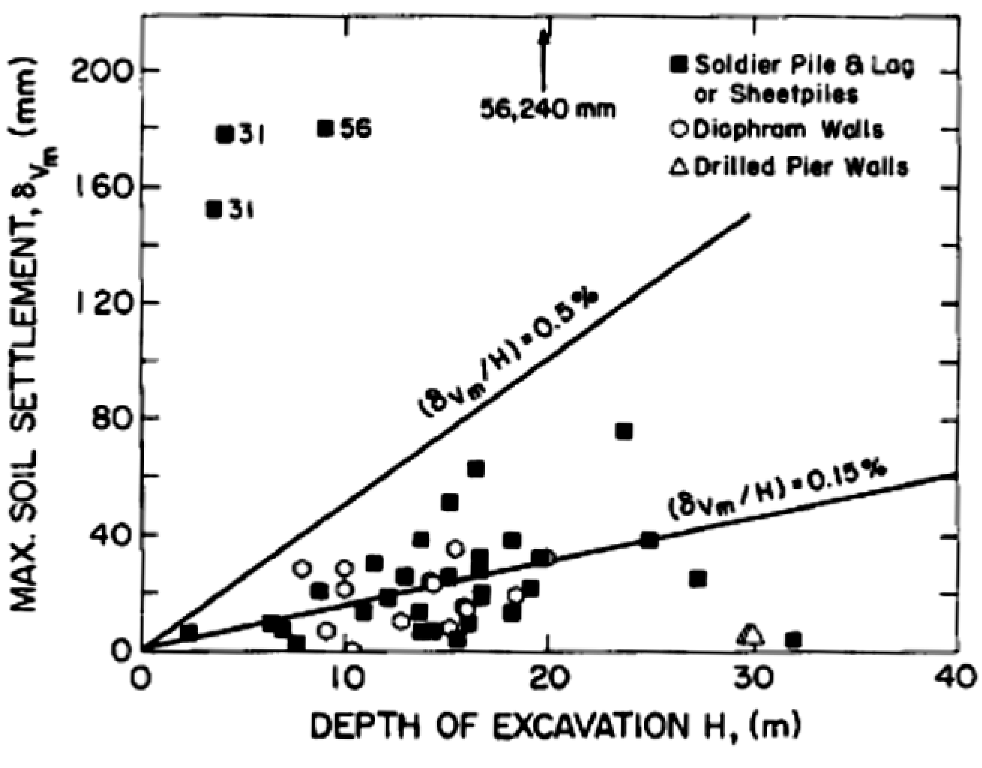


Figure 6: Observed Maximum Soil Settlements in the Retained Soil by Retaining Walls (Clough and O'Rourke, 1990)

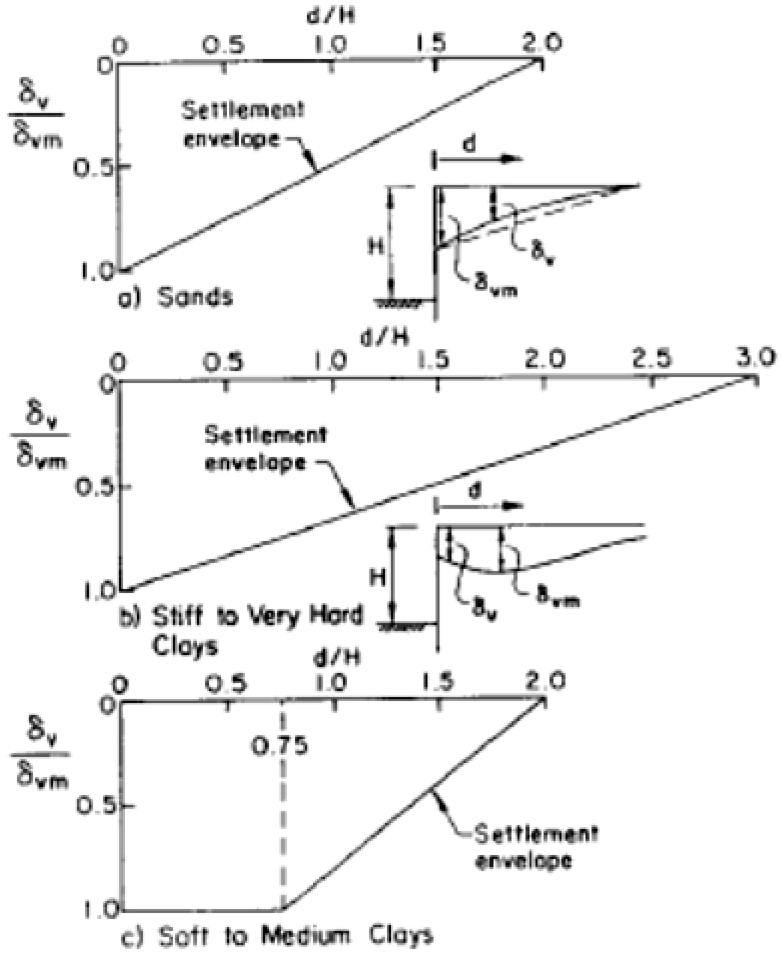


Figure 7: Dimensionless Settlement Profiles Adjacent to Excavations in Different Soil Types (Clough and O'Rourke, 1990)

4.3 Cross Passages

The cross passages in both twin-bore alternatives of this project are expected to be excavated by SEM in conjunction with ground improvement depending on the soil or rock type. Previous tunnel projects, such as U-Link in Seattle, saw less than 0.25% volume loss for cross passages in soil (Settlement Monitoring Data, 2012). Since the cross passages are located between two tunnels (already in the zone of influence from the bored tunnels), it is anticipated that buildings and structures located outside of the zone of influence of the bored tunnels are not likely to be affected by excavation of the cross passages. Furthermore, it is assumed that the ground zones around the cross passages would be sufficiently treated so that the excavation would not cause additional impacts on the adjacent existing structures. At this phase of the study, no further settlement analysis will be performed in support of the environmental documentation; however, analysis for ground movements resulting from cross passage excavation should be performed in future phases with the preferred alternative.

5 Preliminary Estimates of Zones of Potential Influence

The influence of tunnel, portal, and station excavations on existing structures are typically evaluated in several stages. The preliminary assessment includes the estimates of free-field settlements caused by the underground construction without considering the presence of the existing structures. The purpose of this preliminary stage is to identify zones of potential influence from anticipated ground movements in support of the environmental documentation. In this preliminary assessment, limits of the extent of potential influence are established and any structures located outside this zone require no further assessment. The stages that follow are usually structure- or building-specific and are beyond the scope of this TM; the structure-specific analyses will be performed in future phases of this study. The determination of these zones for both the bored tunnels and the open-cut excavations are detailed below. Subsequent analyses will be performed in future phases of the study.

Observation of buildings and structures during excavation may extend beyond these zones as a precautionary measure to confirm there is no damage. Monitoring and survey requirements will be developed through subsequent stages of design and construction

5.1 Bored Tunnels

5.1.1 Background. As shown in Figure 3, a settlement trough width can be assumed to be a distance of *2.5i* from the tunnel's centerline. However, a trough defined by the *2.5i* criteria alone can have varying values of surface settlement at the trough edge depending on how large the maximum settlement is, since the *2.5i* limit is generally 5% of the maximum settlement. An alternative method to establish the zones of potential influence is to determine the distance from the centerline to a limiting settlement value, such as 0.25 inches of settlement, or a combination of a maximum amount of settlement or a maximum slope of the settlement trough. In this case, the zone of potential influence would include the areas where settlement or a slope greater than the set criteria would occur. This method is commonly used if it is believed that damage to buildings or structures would be negligible beyond those limits. As settlement and slope do not capture the full extent of a building's response, other criteria such as angular distortion and horizontal strains will be examined in future phases of this study; however, that is not within the scope of this preliminary assessment.

Since the purpose of the zones of potential influence is to act as a screening criteria for buildings susceptible to potential damage, it is necessary to determine risk levels for different movement ranges that have been developed by various researchers. The FHWA technical manual for soft ground tunneling (2010) refers to tables from Wahls (1981) and Rankin (1988) for limiting potential values in varying risk categories. Table 5 shows the values from Rankin (1988, which indicate that a maximum slope, or tilt, limit of 1/500 accompanied by a maximum settlement of 0.4 inch as the limit for negligible damage.

Table 5: Damage Risk Assessment Chart (Rankin, 1988)

Risk Category	Maximum slope of building	Maximum settlement of building (mm)	Description of risk
1	Less than 1/500	Less than 10	Negligible; superficial damage unlikely
2	1/500 - 1/200	10-50	Slight; possible superficial damage which is unlikely to have structural significance
3	1/200 - 1/50	50-75	Moderate; expected superficial damage and possible structural damage to buildings, possible damage to relatively rigid pipelines
4	Greater than 1/50	Greater than 75	High; expected structural damage to buildings. Expected damage to rigid pipelines, possible damage to other pipelines

5.1.2 Project Criteria. The assumptions and analysis methodology were the same for both the Freeway Tunnel and LRT Alternatives. Settlement troughs were estimated at discrete locations spaced every 500 feet along each tunnel alignment – the freeway single-and twin-bore variations and the LRT tunnel. The settlement troughs were calculated using the volume loss percentages and trough width factors as described in the previous section.

Several recent tunneling projects have used criteria similar to those listed above in Section 5.1.1 for initial screening of building subject to potential damage. The Los Angeles Eastside Gold Line Extension considered a maximum tilt of 1/600 as the criterion for further evaluation of structures (Choueiry et al, 2007).

Because structure-specific tilt or slope calculations are not available at this point in the project study, the instantaneous slope of the settlement trough may be considered instead. While ground slope and tilt are not always equivalent, depending on a building's location along the settlement trough, slope gives a good indication of tilt and also of building damage (Rankin, 1988).

The design team is using a settlement trough slope value of 1/600 or 0.25 inches of settlement (whichever limit extends further from the tunnel centerline) as the criteria for determining the extent of the zones of potential influence from the bored tunnels. These values were used as the screening criteria for negligible risk in the Regional Connector Transit Corridor tunnels (The Connector Partnership, 2012).

5.2 Cut-and-Cover and Open Cut Excavations

It was discussed in Section 4.0, that based on recent data for competent soils the zone of influence, the zone of influence suggested by Clough and O'Rourke is perhaps conservative, and a 1H:1V limit appears suitable to support the environmental documentation. Therefore, the design team proposes using a criterion of 1H:1V to determine the limits of the zones of potential influence from the excavation of the cut-and-cover structures to support the environmental documentation. As noted earlier, this limit will be confirmed with additional data and analysis during future design stages.

5.3 Preliminary Results

The zone of potential influence shows the limit in plan beyond which settlement effects are assumed to be negligible, based on settlements in excess of 0.25 inches, or instantaneous slopes of greater than 1/600 for bored tunnels, and 1H:1V for cut-and-cover excavations. Refer to Attachment B for a plan view of both freeway and LRT alignments showing the zone of potential influence for each of the alternatives. The results for the bored tunnel portions are summarized in Table 6 and for the open-cut excavations in Table 7.

Table 6: Estimated Values for the Zone of Potential Influence for Bored Tunnels

		Zone of Potential Influence width (feet)	
		Average	**Maximum**
Freeway Tunnel Twin-Bore	Soil	310	360
	Rock	290	330
Freeway Tunnel Single-Bore	Soil	190	240
	Rock	160	170
LRT Tunnel Twin-Bore	Soil	90	130
	Rock	10	30

Table 7: Estimated Values for the Zone of Potential Influence for Cut and Cover Excavations

		Potential Influence Distance from Open Cut (feet)
Freeway Tunnel Twin-Bore	North Portal	100
	South Portal	130
Freeway Tunnel Single-Bore	North Portal	100
	South Portal	130
LRT Tunnel Twin-Bore	South Portal	50
	Alhambra Station	80
	Huntington Station	80
	S Pasadena Station	90
	Fillmore Station	80

6 Control of Ground Movement

6.1 Excavation Requirements

During the design phase of the project, the owner or owner's representative can set requirements for the contractor to meet with respect to equipment, tunneling practices, and operations to control/limit ground movements. Requirements that can be specified in the bidding documents which can help limit ground movements include but are not limited to:

- Selecting a pre-qualified contractor with experience mining with pressurized-face TBMs.
- Requiring that a pressurized-face TBM is used for the bored tunnel excavation, that the percentage ground loss is limited to a certain percentage, and that the contractor demonstrate that he can achieve that ground loss percentage with the machine selected.
- Requiring that a robust system be in place for monitoring volume of excavated materials in real time.
- Requiring that the TBM(s) proposed have a comprehensive and integrated tail void grouting system and the ability to inject bentonite along the shield to liming tail- and shield-related ground loss.
- Requiring a robust geotechnical instrumentation and monitoring program which would allow for real-time monitoring of movements to allow contingency measures to be implemented in a timely manner.
- Requiring a sufficiently-stiff support of excavation system for the portals and underground stations to limit horizontal and vertical ground movements which would cause damage to existing adjacent structures.

6.2 Additional Mitigation Methodology

The process of excavation by pressurized-face TBM will inherently be mitigating ground movements by limiting ground losses at the face by applying face pressure and along the shield by injecting bentonite as the TBM advances if required, and grouting the annulus left by the gap as described previously. These measures used with modern TBMs usually reduce ground movements to negligible amounts but, where necessary, additional mitigation measures may be required to reduce excavation-induced settlement and lessen or eliminate the ground movement effects on the adjacent structures. Several methods could be employed including:

- Permeation grouting
- Compaction grouting

- Compensation Grouting
- Underpinning

The applicability of these mitigation measures, as well as additional options, should be expanded in future phases of this study when a preferred alternative is selected.

7 Future Studies

In further phases of design, further evaluation will be performed. Anticipated settlement contours should be drawn for the preferred alternative, which will better show the magnitude and extent of horizontal and vertical excavation-induced ground movements. Additionally, a structure-specific analysis will be performed to better understand the response of the structures along the preferred alignment to the excavation-induced ground movements.

As one method to further evaluate influence to buildings, additional analysis may be performed using the method outlined in Boscardin and Cording (1989). The intent of a Boscardin and Cording Analysis is to evaluate the responses of buildings and structures to the ground movements and to determine which buildings or structures are potentially at risk of being damaged, requiring mitigation or repair.

The Boscardin and Cording method is an empirical method that predicts potential damage to brick bearing walls and small-frame structures based on the critical tensile strains estimated using a deep beam model. Upon the investigation of deep beam behaviors, Boscardin and Cording concluded that the first observable cracking will be controlled by shear-related deformations. Shear-related deformations are a function of the building angular distortion and horizontal tensile strain. The method, backed up by settlement data derived from actual field measurements, has gained worldwide acceptance in engineering practice.

Depending on the location of the building relative to the tunnel excavations, different portions of the building can lie in a hogging or sagging zone, which are separated from each other by the point of inflection of the settlement trough. Since the building portions in each zone experience different structural responses to the settlement and ground horizontal strains, they are considered separately, as recommended by Mair et al. (1996) and illustrated in Figure 8.

For the building portion located in a hogging zone, the neutral axis of the beam is assumed to be at the lower edge of the beam, and the maximum angular distortion is calculated using the equation recommended by Boscardin and Cording (1989). The horizontal tensile strain in the building is assumed to be the average ground horizontal strain within the building foundation in this zone.

For the building portion located in the sagging zone, the beam neutral axis is assumed to be at mid-height, and the angular distortion is calculated using the equation recommended by Wahls (1981). Since the ground horizontal strain in the sagging zone is compressive, the building horizontal strain in this zone is conservatively assumed to be zero.

In addition to the building maximum angular distortions calculated using a deep beam model, the maximum angular distortions of the settlement trough within the building extents in the hogging and sagging zones (defined as the slope of the settlement trough minus average rigid-body tilt of the building) are calculated in order to bracket the range of predicted damage. In this approach, the building is implicitly assumed to closely follow the shape of the settlement trough, yielding the upper bound angular distortion values. To be conservative in this phase, the more critical damage level resulting from the above two approaches (the deep beam model and the settlement trough angular distortion) is reported as the maximum predicted damage for the building.

The calculated maximum angular distortion and horizontal tensile strain for each building is correlated to a set of curves for constant critical tensile strain (strain in the building walls associated with a particular damage level). These curves define the limits of damage categories as shown in Figure 9. These curves were constructed based on a simple deep beam, assuming that the beam has an L/H ratio equal to 1 and Young's modulus to shear modulus (E/G) ratio equal to 2.6, and the neutral axis is located at the lower edge of the beam. Boscardin and

Cording's approach, though developed for unreinforced masonry (URM) buildings, is considered applicable for other types of buildings with finishes or cladding that exhibit similar brittle behavior to that of masonry.

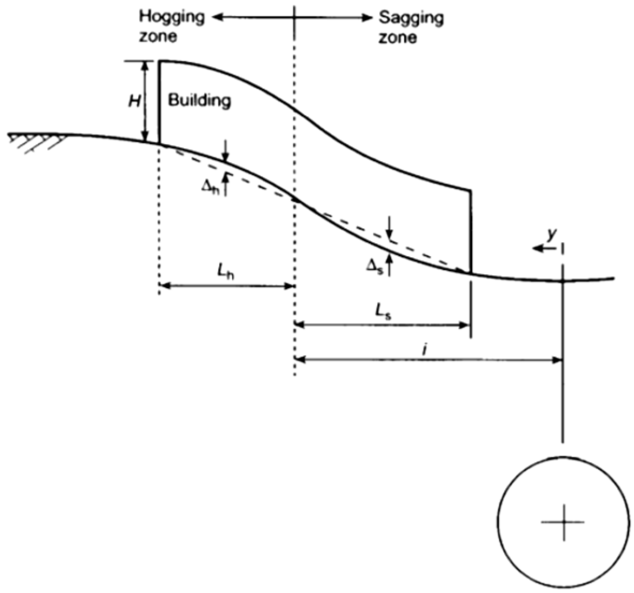


Figure 8: Building Length and Maximum Deflection in Hogging and Sagging Zones (Mair et al., 1996)

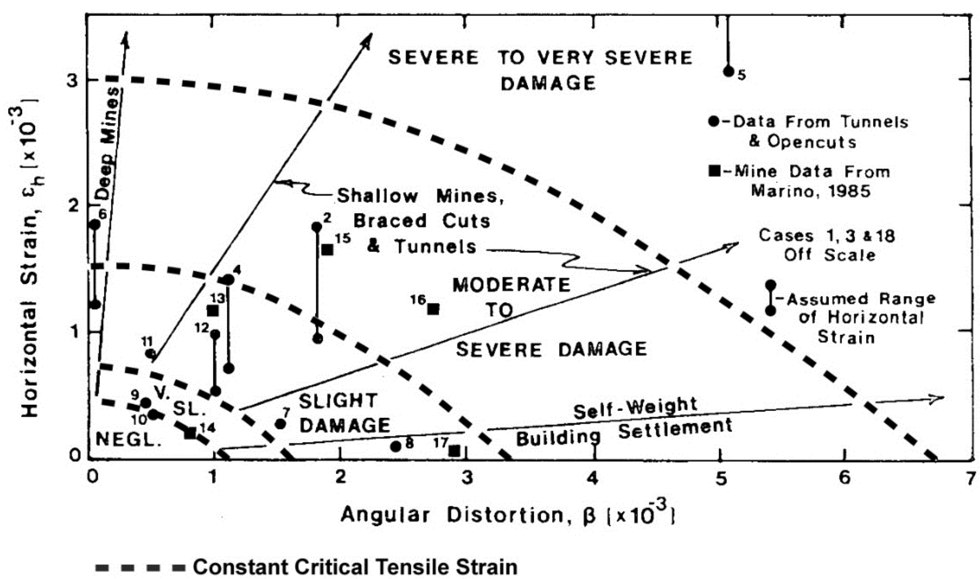


Figure 9: Relationship of Damage to Angular Distortion & Horizontal Strain (Boscardin and Cording, 1989)

8 Limitations and Recommendations

This TM provides a preliminary stage estimation of the potential limits of excavation-induced ground movements for the Freeway Tunnel and LRT alternatives. The focus of the study was to aid in the determination of the zones of potential influence for the environmental documentation phase of the project. In future phases of the work, the following should be studied in greater detail:

- Settlement contour plan showing expected vertical and lateral excavation-induced ground movements along preferred alternative
- Expected influence to utilities
- Settlements induced from excavation of seismic vaults

- Building-specific analyses along the alignment, especially in proximity to the stations/portals
- Railroad, major arterials, and other major infrastructure
- Settlement related to excavation of cross passages
- Stiffness of support of excavation walls to specify for cut-and-cover excavations to better understand and control wall movements and in turn ground movements related to cut-and-cover excavations
- Observation and monitoring for specific structures along the alignment based on anticipated ground movements and structure-structure specific classification

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10 Revision Log

Revision 0	January 27, 2014	Internal Review
Revision 1	April 14, 2014	Metro/Caltrans Review
Revision 2	June 18, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

Geologic Profiles

NOTES:

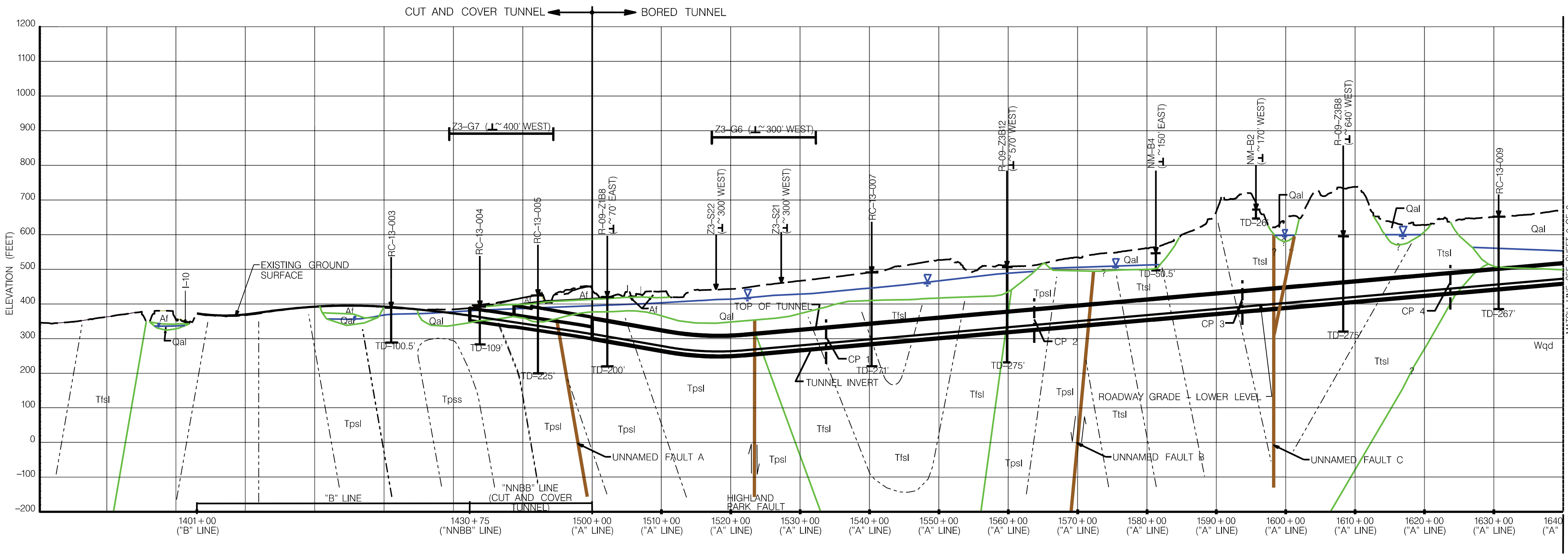
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3–G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

A–13–020 TD–149.8' GEOTECHNICAL BOREHOLE WITH TOTAL DEPTH AND PROJECTION:
A, R, RC, O–13–001 – CH2M HILL, THIS STUDY
R–09–Z1B8 – CH2M HILL, 2010
NM–B3 – NINYO AND MOORE, 1999
EMI–3 – EARTH MECHANICS INC, 2006
ES–2 – CALTRANS, 1974

CP – CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

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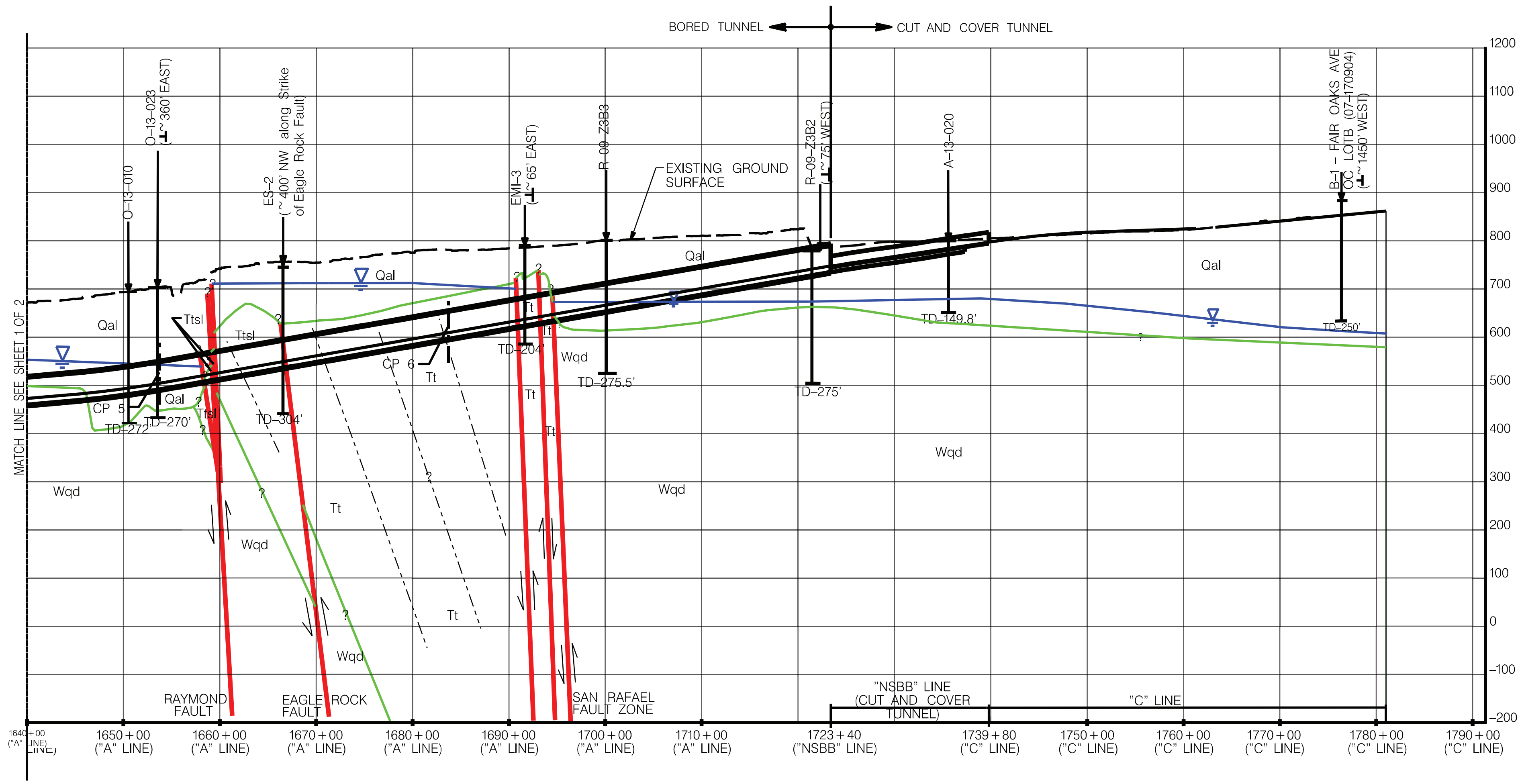
Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

- Af ARTIFICIAL FILL
- Qal ALLUVIAL SOIL
- Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER
- Tfsl FERNANDO FORMATION, SILTSTONE MEMBER
- Tpsl PUENTE FORMATION, SILTSTONE MEMBER
- Tpss PUENTE FORMATION, SANDSTONE MEMBER
- Tt TOPANGA FORMATION, UNDIFFERENTIATED
- Ttss TOPANGA FORMATION, SANDSTONE MEMBER
- Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER
- Ttsl TOPANGA FORMATION, SILTSTONE MEMBER
- Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
- INTRAFORMATIONAL CONTACT
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  - NM-B3 – NINYO AND MOORE, 1999
  - EMI-3 – EARTH MECHANICS INC, 2006
  - ES-2 – CALTRANS, 1974
- CP – CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 2 OF 2

NOTES:

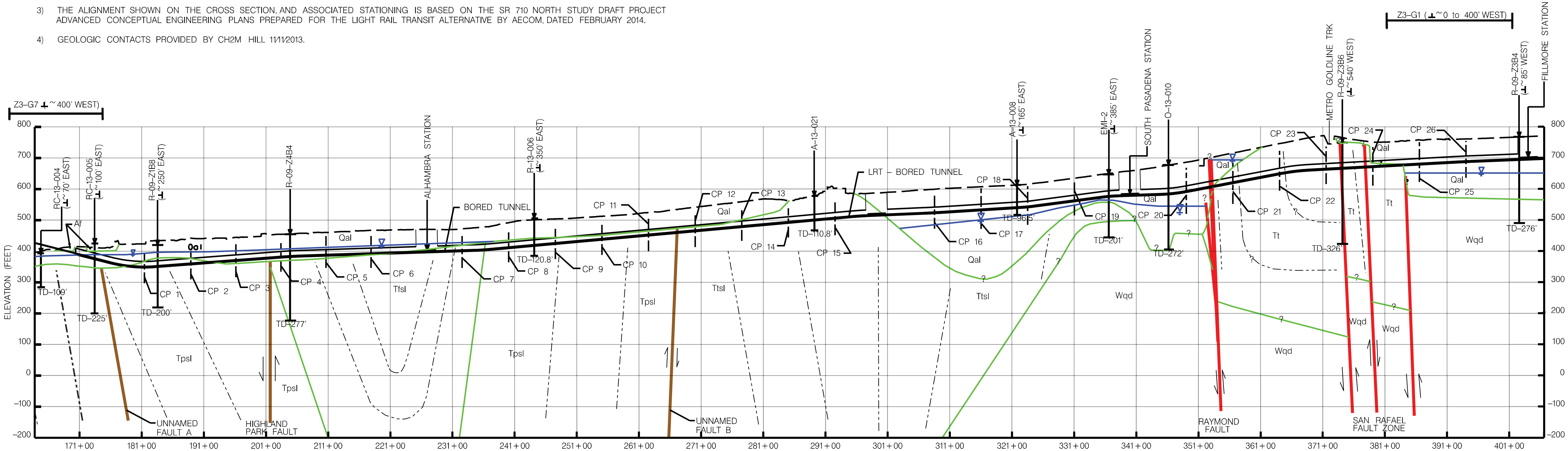
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT ADVANCED CONCEPTUAL ENGINEERING PLANS PREPARED FOR THE LIGHT RAIL TRANSIT ALTERNATIVE BY AECOM, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative



LEGEND

UNITS

Af ARTIFICIAL FILL

Qal ALLUVIAL SOIL

Tfcg FERNANDO FORMATION, CONGLOMERATE MEMBER

Tfsl FERNANDO FORMATION, SILTSTONE MEMBER

Tpsl PUENTE FORMATION, SILTSTONE MEMBER

Tpss PUENTE FORMATION, SANDSTONE MEMBER

Tt TOPANGA FORMATION, UNDIFFERENTIATED

Ttss TOPANGA FORMATION, SANDSTONE MEMBER

Ttcg TOPANGA FORMATION, CONGLOMERATE MEMBER

Ttsl TOPANGA FORMATION, SILTSTONE MEMBER

Wqd WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3–G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

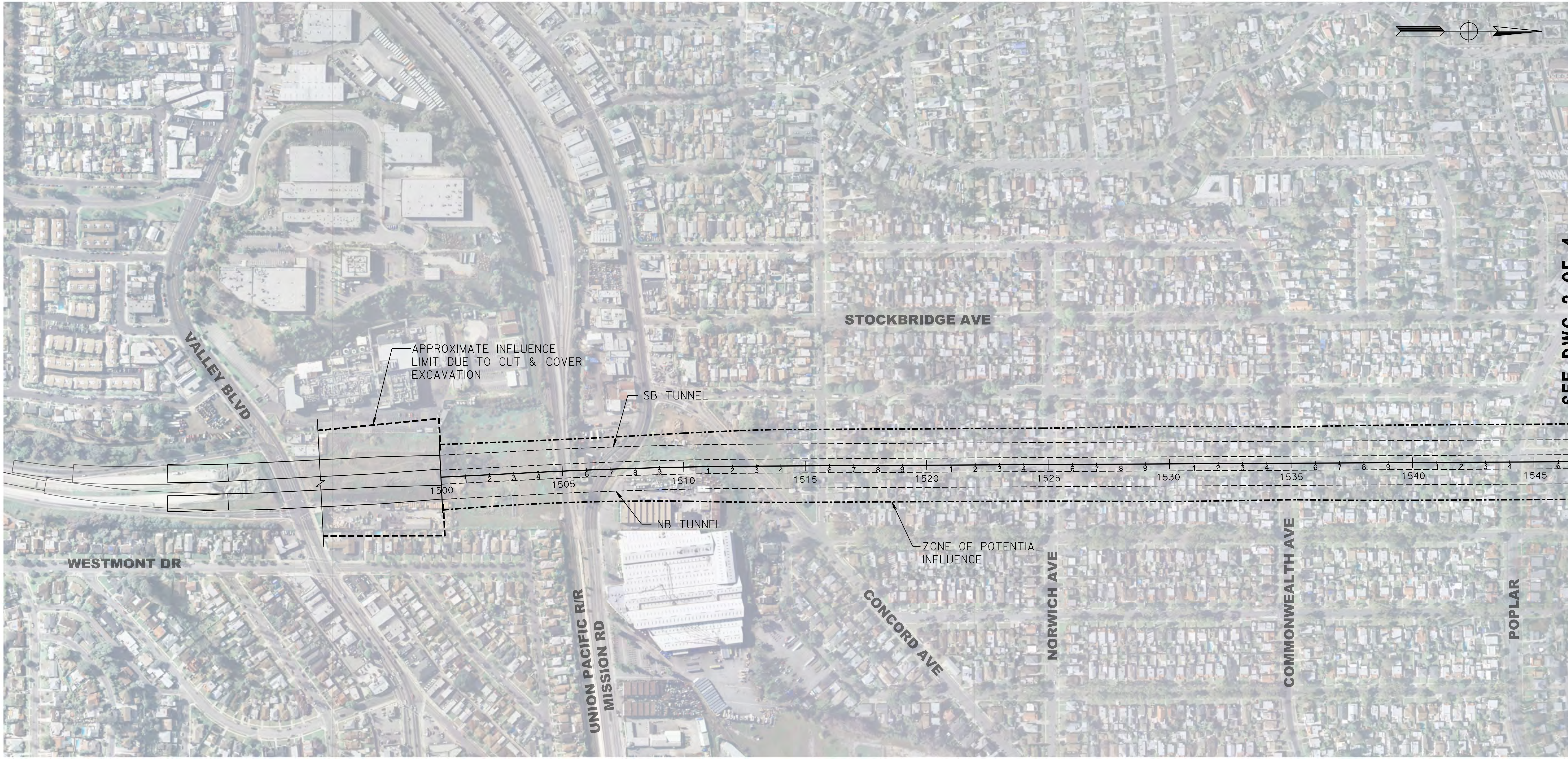
A–13–020 TD–149.8' GEOTECHNICAL BORHOLE WITH TOTAL DEPTH AND PROJECTION:
- A, R, RC, O–13–001 – CH2M HILL, THIS STUDY
- R–09–Z1B8 – CH2M HILL, 2010
- NM–B3 – NINYO AND MOORE, 1999
- EMI–3 – EARTH MECHANICS INC, 2006
- ES–2 – CALTRANS, 1974

CP – CROSS PASSAGE

LRT GEOLOGIC PROFILE

Attachment B

Zone of Potential Influence Plans



TWIN BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 1 OF 4

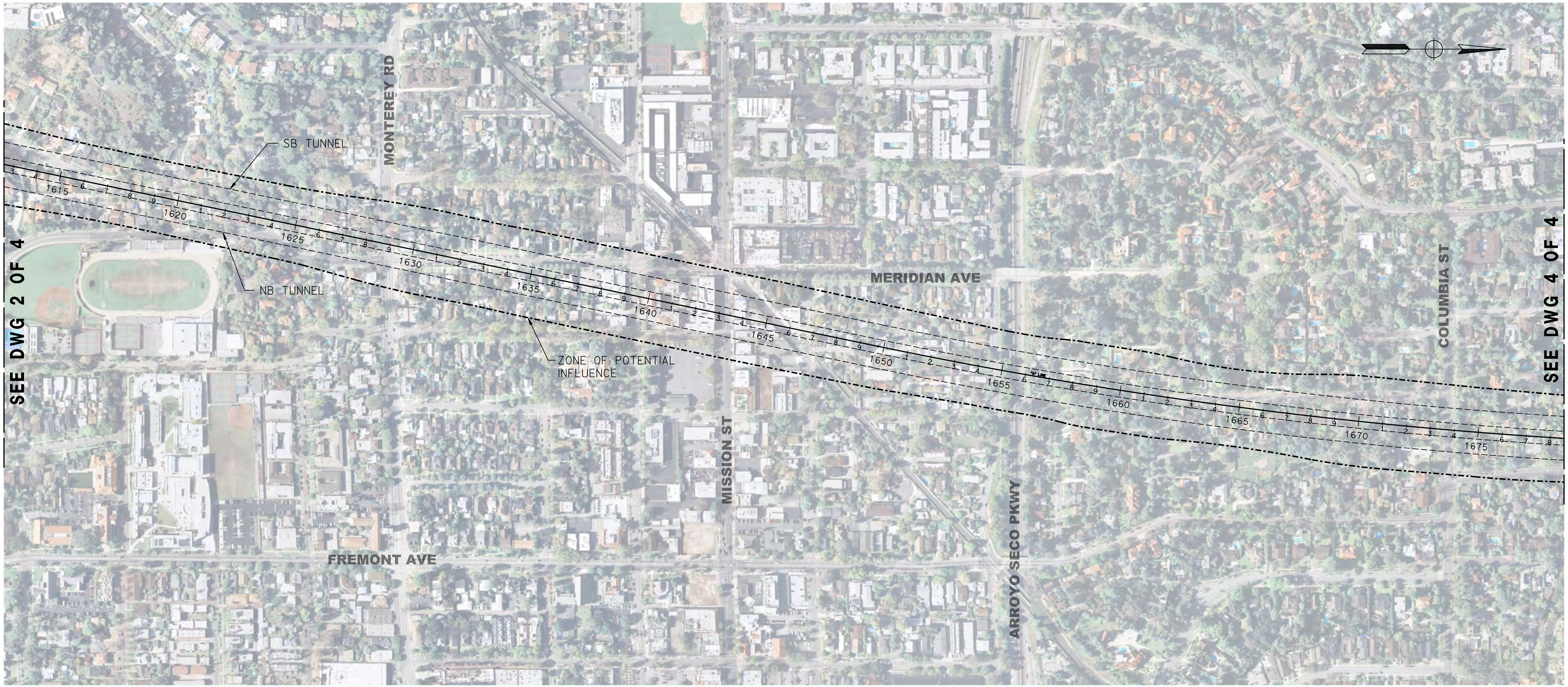
TWIN BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 2 OF 4



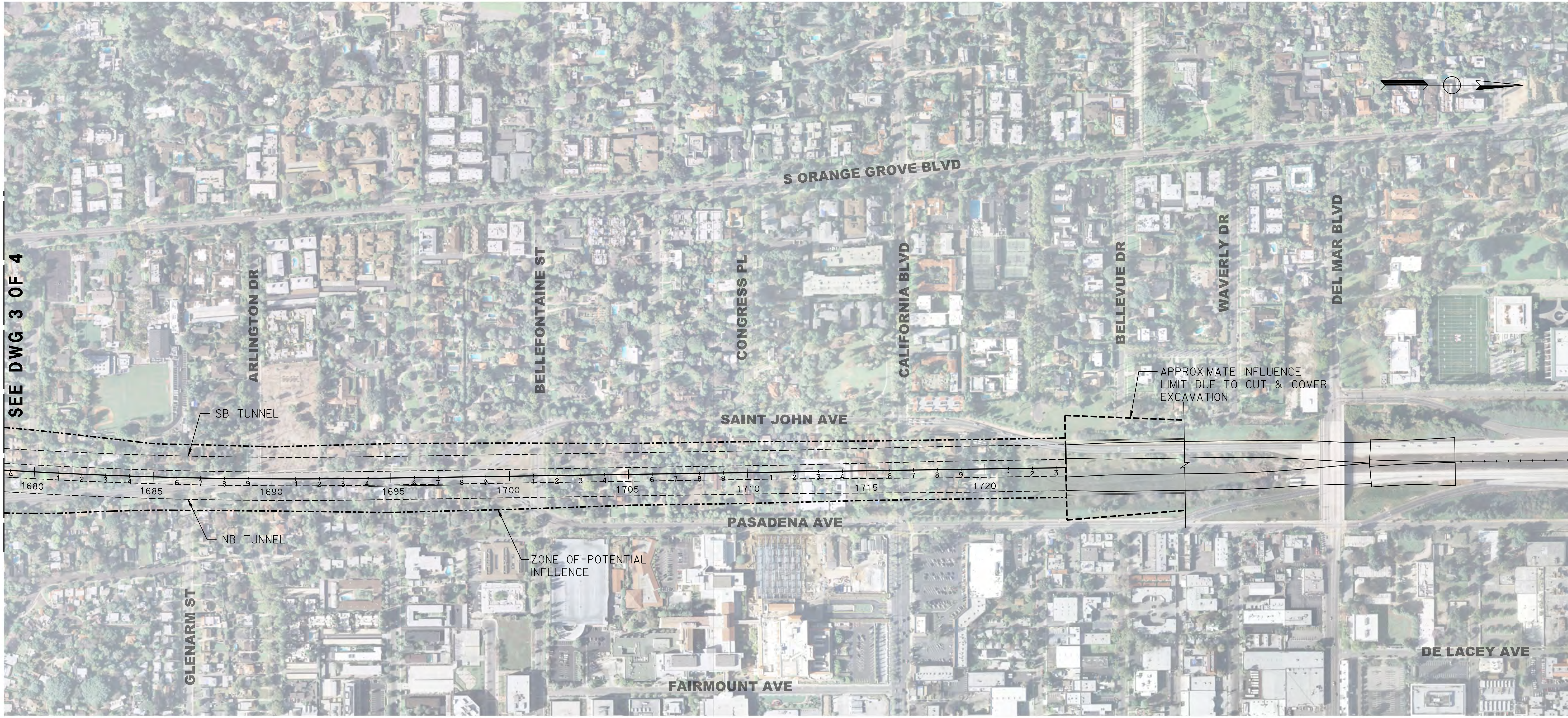
TWIN BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 3 OF 4



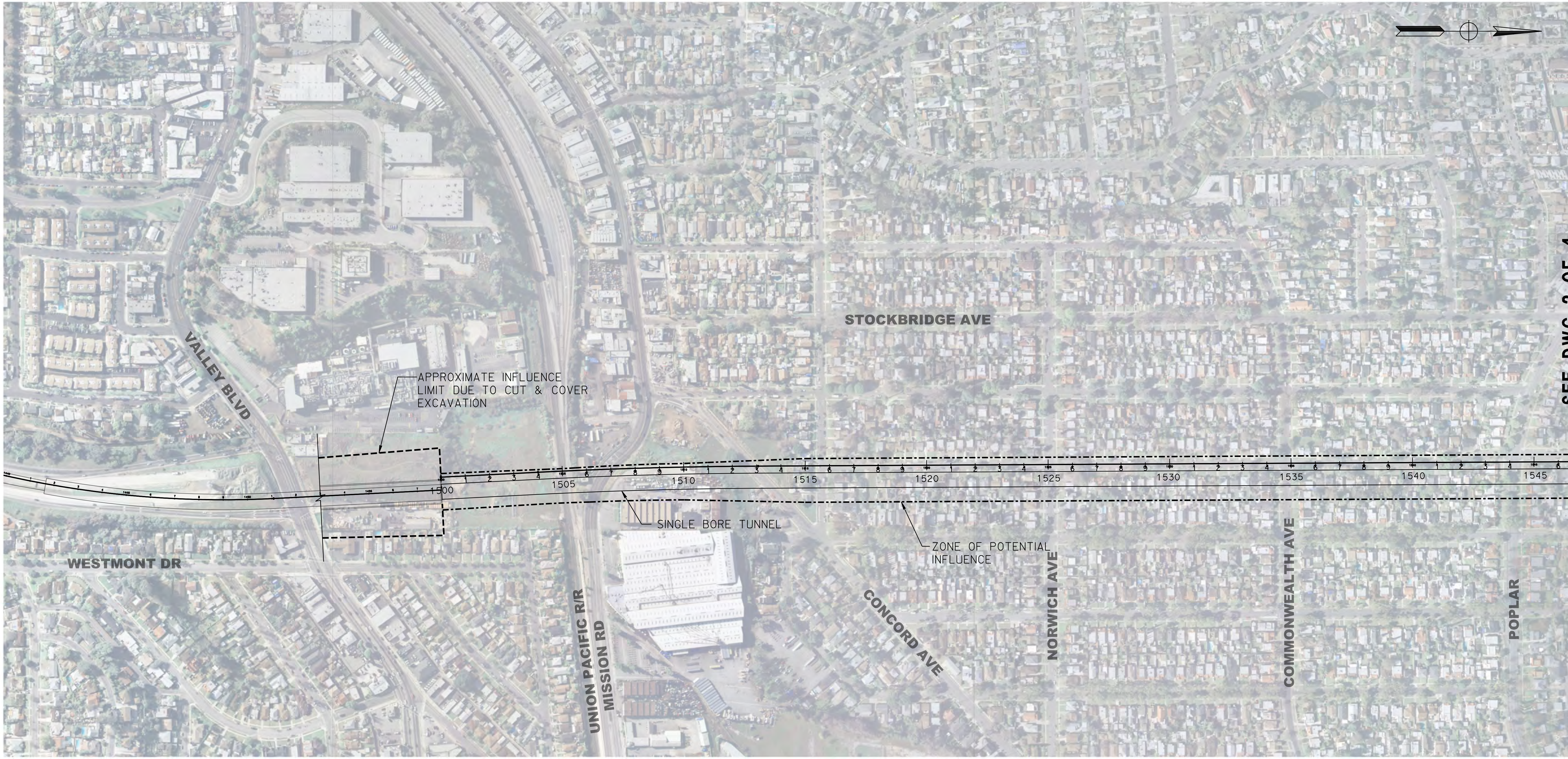
TWIN BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 4 OF 4



SINGLE BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 1 OF 4

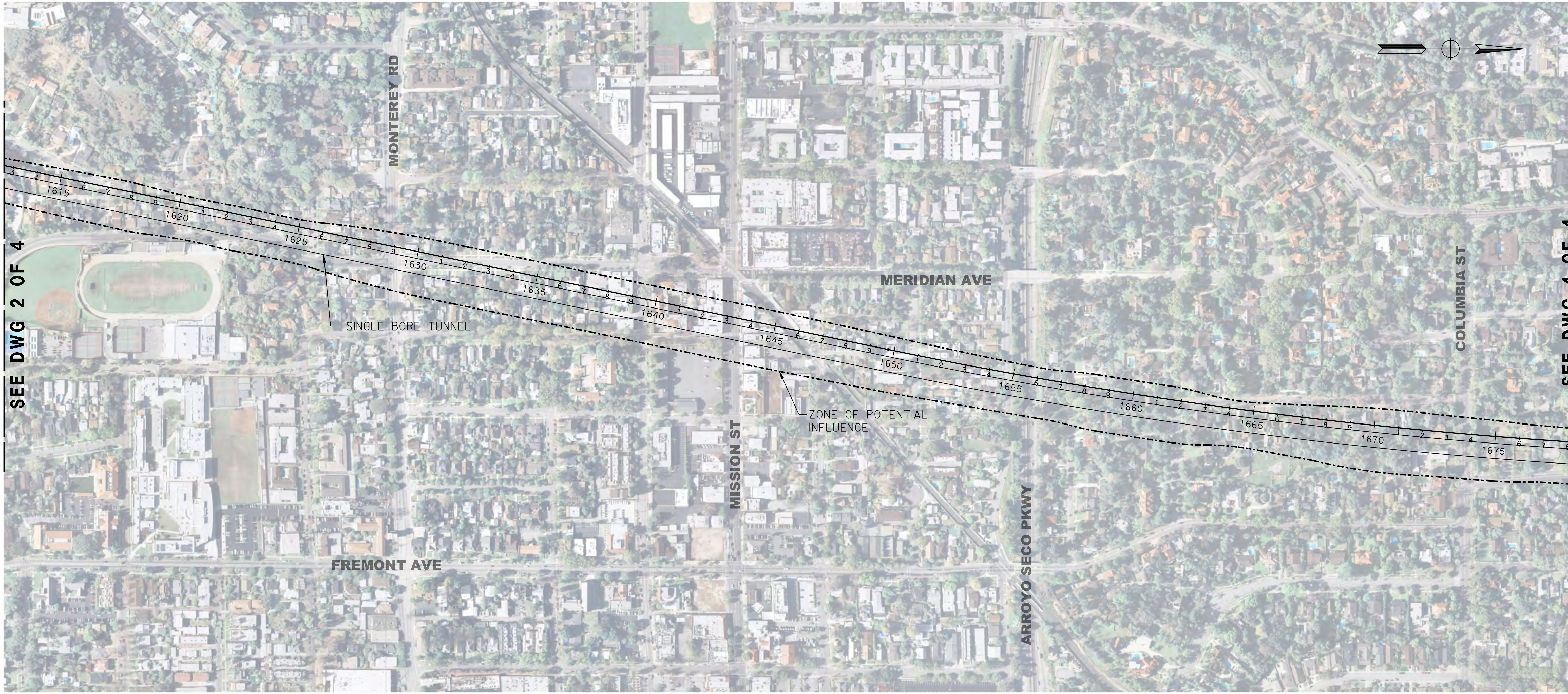
SINGLE BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 2 OF 4



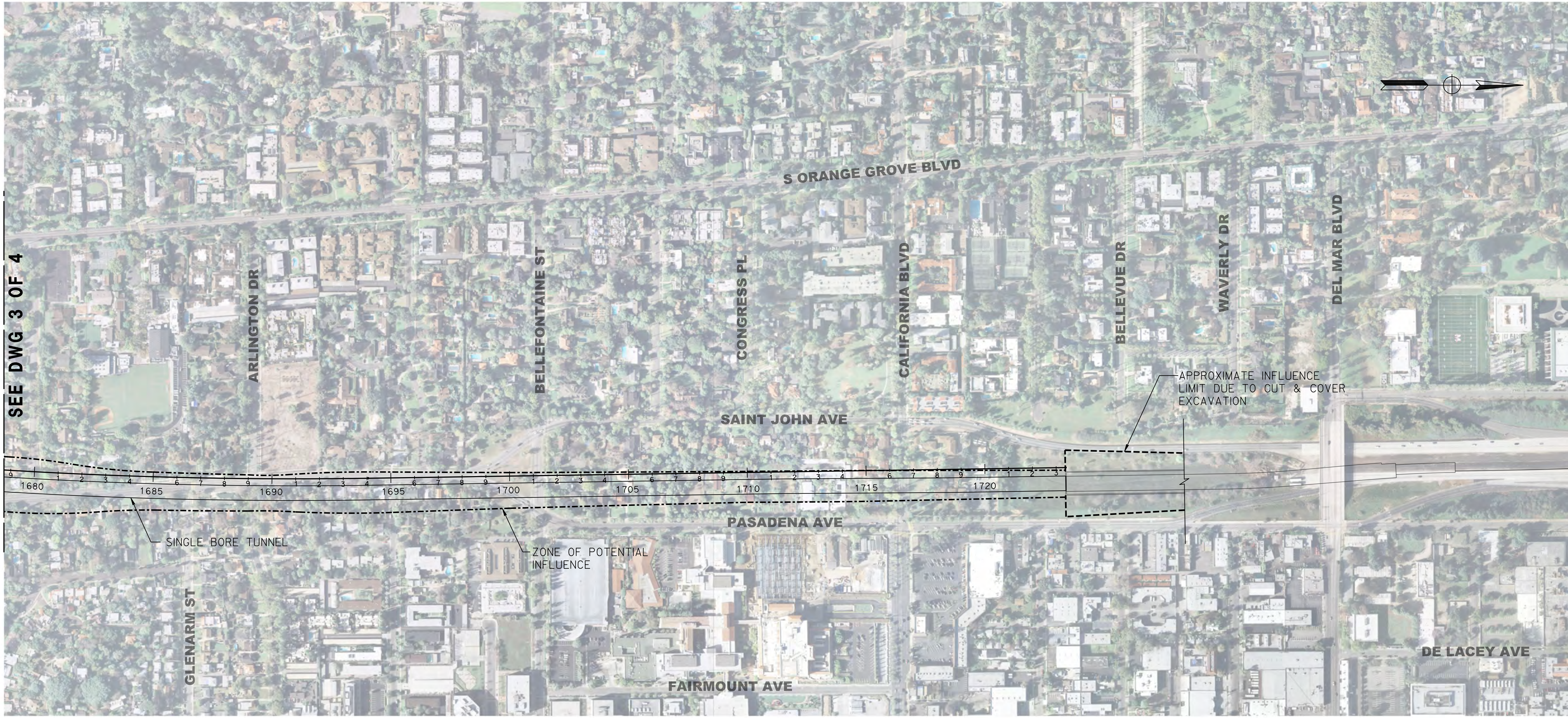
SINGLE BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 3 OF 4



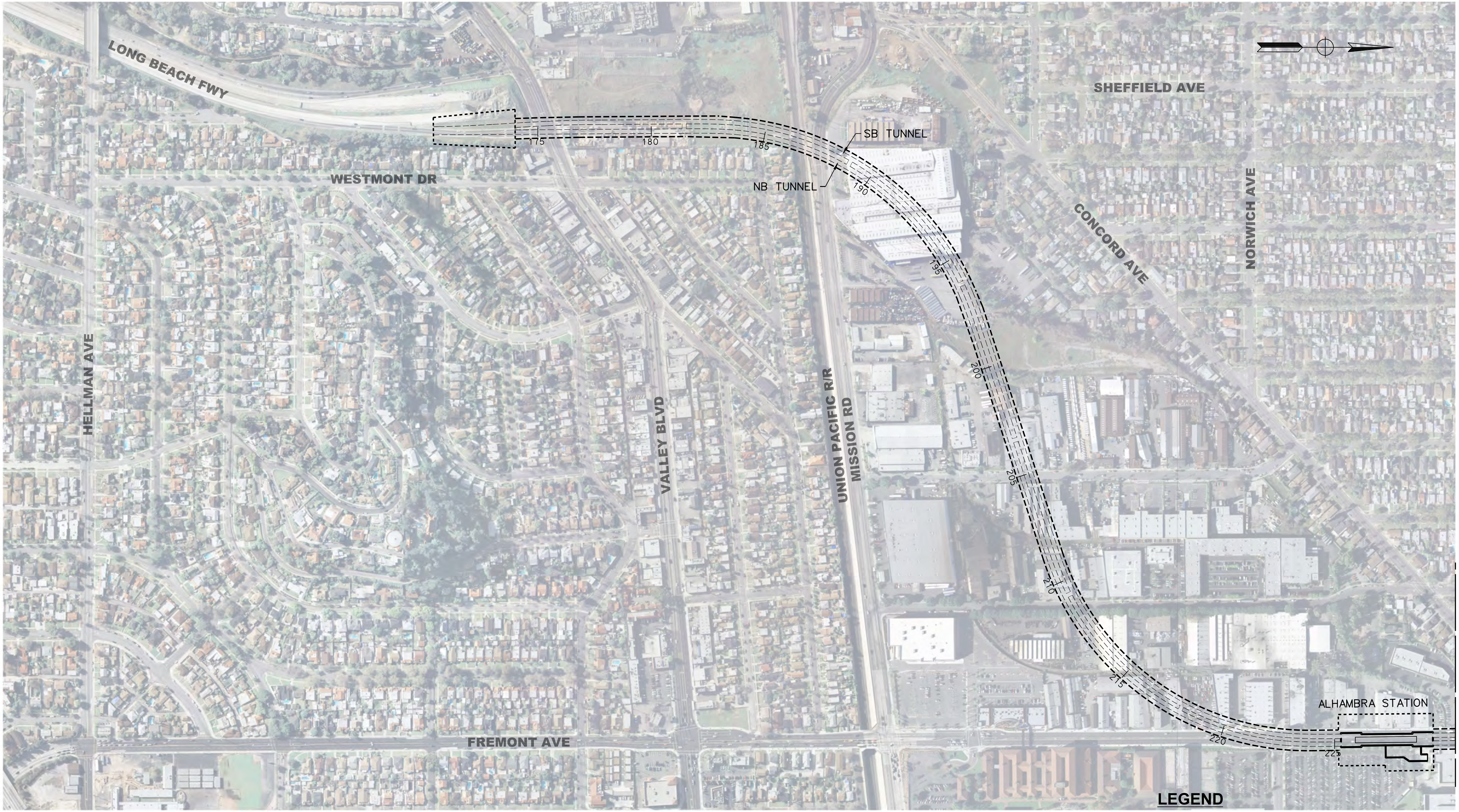
SINGLE BORE ALTERNATIVE

SCALE: 1"=400'

200' 0 200' 400'

JUNE 2, 2014

DRAWING NO. 4 OF 4



LRT SETTLEMENT PLAN

SCALE: 1"=400'

200' 0 200' 400'

LEGEND

ZONE OF POTENTIAL INFLUENCE (BORED TUNNELS)

ZONE OF POTENTIAL INFLUENCE (CUT & COVER EXCAVATIONS)

JUNE 3, 2014

DRAWING NO. 1 OF 4

LRT SETTLEMENT PLAN

SCALE: 1''=400'

200' 0 200' 400'

LEGEND

------------ ZONE OF POTENTIAL INFLUENCE (BORED TUNNELS)

.... ZONE OF POTENTIAL INFLUENCE (CUT & COVER EXCAVATIONS)

JUNE 3, 2014

DRAWING NO. 2 OF 4

LRT SETTLEMENT PLAN

SCALE: 1"=400'

200' 0 200' 400'

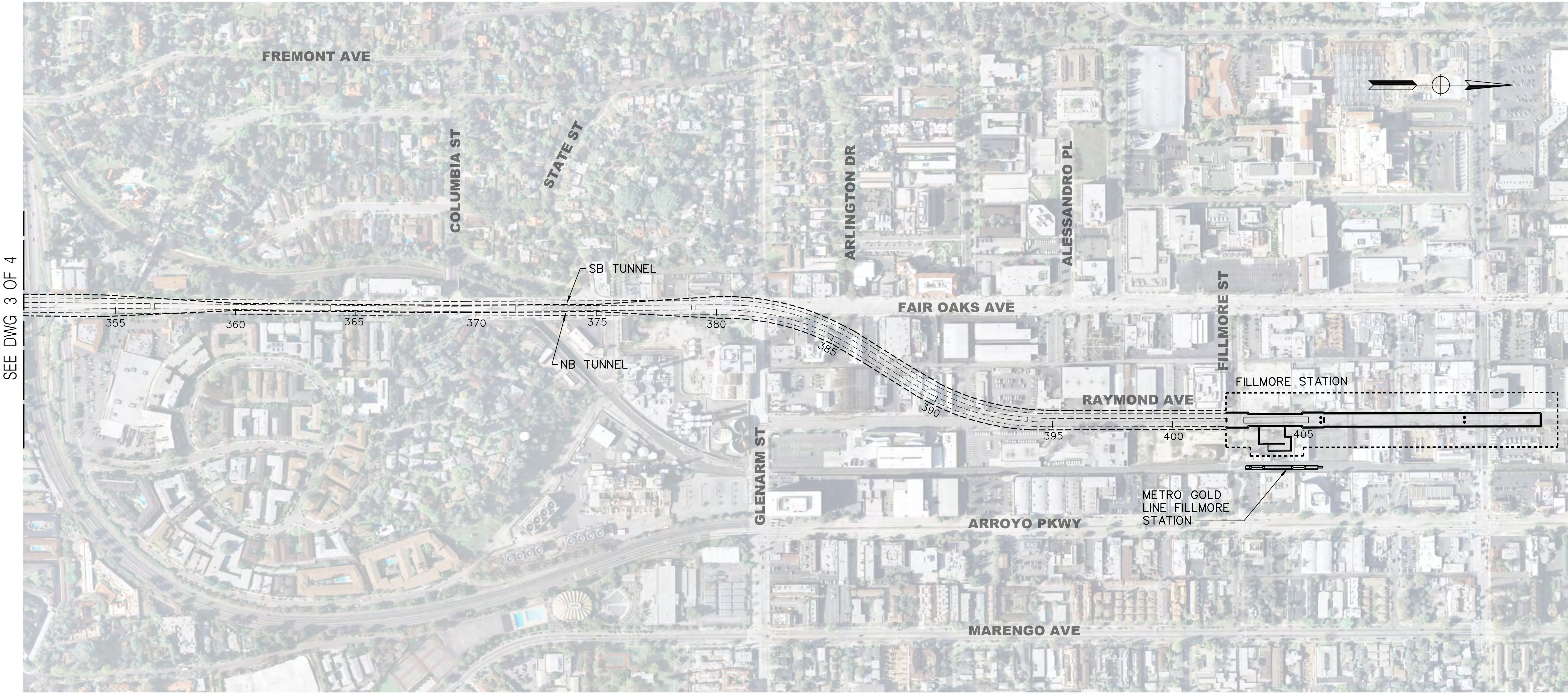
LEGEND

ZONE OF POTENTIAL INFLUENCE (BORED TUNNELS)

ZONE OF POTENTIAL INFLUENCE (CUT & COVER EXCAVATIONS)

JUNE 3, 2014

DRAWING NO. 3 OF 4



LEGEND

ZONE OF POTENTIAL INFLUENCE (BORED TUNNELS)

ZONE OF POTENTIAL INFLUENCE (CUT & COVER EXCAVATIONS)

LRT SETTLEMENT PLAN

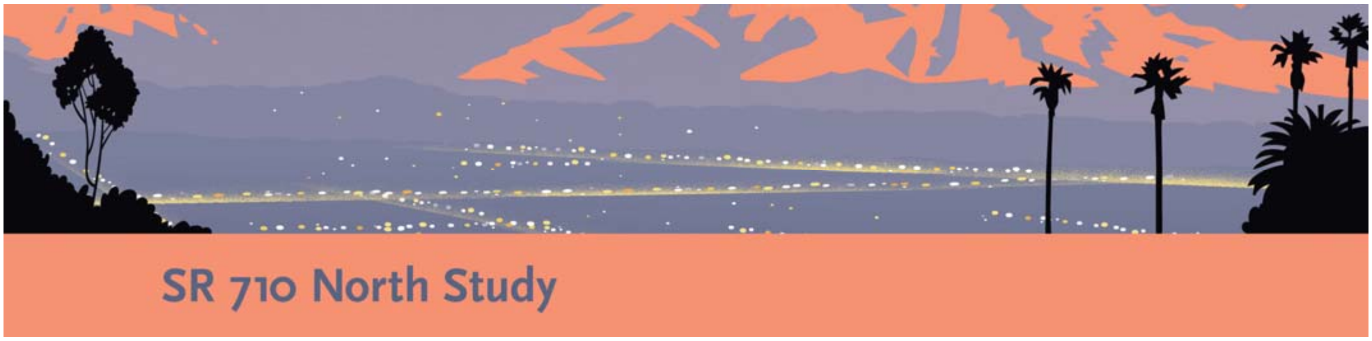
SCALE: 1''=400'

200' 0 200' 400'

JUNE 3, 2014

DRAWING NO. 4 OF 4

Appendix H
TM-6 Preliminary Design Concepts for Fault Crossings



TECHNICAL MEMORANDUM 6

Preliminary Design Concepts for Fault Crossings

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) discusses the evaluation of the preliminary design concepts for the portions of the bored tunnels that cross active and potentially active fault zones along the tunnel alignments for the Freeway Tunnel and LRT Alternatives. Because of the fault offset expected at these fault zones, widened tunnel cross sections (called vault sections) are proposed to accommodate the fault offset. This TM provides a brief discussion of background information including a general description of the tunnel alignments, anticipated geologic conditions, and the identified active and potentially active fault zones and their locations.

The focus of the discussion in this TM is on the design criteria and design basis related to design of the fault crossing, design concepts for the fault crossings, and the design methodology that is employed to evaluate one feasible fault crossings concept for each bored tunnel alternative. At this stage in the study, the details provided herein are of a conceptual nature, and the TM presents only one feasible option for each the fault crossings, which were developed in support of the environmental documentation.

Preliminary design concepts associated with the fault crossings such as the excavation methods, initial support if applicable, final lining and internal structure requirements for the vault sections are presented on the drawings provided in Attachment A of this TM. The preliminary design concepts for the normal tunnel sections are not addressed in this TM and can be found in *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* (JA, 2014d) and *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014e) for the Freeway Tunnel and the LRT Alternatives, respectively.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Tunnel Alignments and Anticipated Geologic Conditions

The tunnel alignments of the Freeway Tunnel and LRT Alternatives, anticipated geologic conditions, the fault crossings are briefly described below. It should be noted that site-specific geotechnical investigations have not yet been completed at each of the various fault zones and the description of expected conditions is based on limited data and our local experience with geologic conditions in this area. Future design studies will require site-specific data to be obtained in order to refine the design concepts discussed herein.

2.1 Freeway Tunnel Alternatives

The single-bore and twin-bore variations for the Freeway Tunnel Alternative have the same vertical and horizontal alignment, and it is anticipated that both Freeway variations would be mined using a tunnel boring machine (TBM) (JA, 2014c). The tunnels are expected to have a lining inside diameter of about 53.5 feet with an excavated diameter of approximately 60 feet (JA, 2014a). Each tunnel bore would accommodate four traffic lanes—two on the top deck and two on the bottom deck. Accordingly, the single-bore variation would provide a total of four traffic lanes (two in each direction), while the twin-bore variation would provide eight traffic lanes (four in each direction). Each bored tunnel would be approximately 22,340 feet long. The ground cover ranges from approximately 20 to 280 feet, with an average cover of approximately 150 feet. Figure 1 shows the vertical alignment of the Freeway single-bore and twin-bore alternatives.

The geologic conditions along the freeway tunnel alignments generally consist of Quaternary-age alluvium, weak Tertiary-age sedimentary rock formations (Fernando, Puente, and Topanga formations), and stronger crystalline basement complex rocks (JA, 2014b). As indicated on Figure 1, the majority of the Freeway Tunnel alternatives would be excavated in the sedimentary rock formations, with portions of the tunnels also in the alluvium and basement complex rocks.

The Freeway Tunnel Alternatives are anticipated to cross one active and two potentially active faults: the Raymond fault is considered an active fault, while the Eagle Rock and San Rafael faults are designated as potentially active. For planning purposes these latter two faults are also being treated as active faults (CH2M HILL, 2014). In addition to these active and potentially active faults, four other inactive faults (Highland Park fault, and Unnamed faults A, B, and C) are also expected to be crossed by the freeway alignments. The fault zones located entirely in rock, such as the Eagle Rock fault, are expected to be composed of crushed and intensely fractured and sheared rocks with clay gouge and fault breccia. The Raymond and San Rafael faults are located at the contact between rock and alluvium. These fault zones are expected to be composed of poorly graded alluvium (coarse-grained soils) on one side and sheared, heavily fractured, and highly weathered/altered decomposed rock on the other side. The approximate locations of these fault zones along the tunnel alignments are shown in Figure 1. Key characteristics of the active and potentially active faults relevant to the fault crossing concepts are described in Section 3.

2.2 LRT Alternative

The LRT Alternative includes approximately 23,000 feet of twin-bore tunnels that would be excavated with a TBM, and the excavated diameter of the tunnels is expected to be approximately 22 feet (JA, 2014a). The LRT tunnel alternative also includes four underground stations that would be excavated using cut-and-cover techniques. The ground cover ranges from approximately 10 to 90 feet, with an average cover of approximately 50 feet. Figure 2 shows the vertical alignment of the LRT Alternative.

The geologic conditions along the LRT tunnel alignment generally consist of Quaternary-age alluvium, and weak Tertiary-age sedimentary rock formations (Puente and Topanga Formations) (JA, 2014b). As shown in Figure 2, the LRT Alternative alignment is closer to the ground surface and expected to encounter primarily alluvium and the weak sedimentary rock formations. While it is likely that the majority of the LRT tunnel would be excavated in alluvium, it is also possible that mixed-face conditions (i.e., both rock and soil in the excavation face) would occur along the LRT tunnel alignment as the vertical tunnel profile is, at many locations, near the depth of the estimated contact between the alluvium and sedimentary formations.

The LRT tunnels are anticipated to cross one active and one potentially active fault. The Raymond fault is considered an active fault and the San Rafael fault is designated as potentially active. For planning purposes the latter fault is also being treated as an active fault (CH2M HILL, 2014). In addition to these active and potentially active faults, three other inactive faults (Highland Park fault, Unnamed faults A and B) may also be intersected by the LRT tunnel alignment. Generally, the fault zones located entirely in rock are expected to be composed of crushed and intensely fractured and sheared rocks with clay gouge and fault breccia. The Raymond fault is located at the contact between rock and alluvium. This fault zone is expected to be composed of poorly graded alluvial soil and weakly cemented, sheared, and intensely weathered to decomposed rock. The approximate locations of these fault zones along the tunnel alignment are shown in Figure 2. Key characteristics of the active and potentially active faults relevant to the fault crossing concepts are described in Section 3.

3 Seismic Design Criteria and Fault Characteristics

3.1 Seismic Design Criteria

The seismic design criteria applicable to the Freeway Tunnel Alternative is different from the criteria applicable to the LRT Alternative (CH2M HILL, 2014). The criteria are based on the requirements of the agency, either Caltrans or Metro, that eventually would operate the completed project.

3.1.1 Freeway Tunnel Alternative

Caltrans has designated the Freeway Tunnel Alternative as an Ordinary Nonstandard Facility in terms of developing seismic design criteria for this alternative. This facility classification is equivalent to the Recovery Route classification. Two levels of seismic events must be considered, consisting of the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE). The definitions and associated performance criteria for these two levels of design seismic event are provided in CH2M HILL (2014).

According to the Caltrans seismic design criteria, the SEE and FEE requirements must be satisfied for fault offsets that have an average return period of 1,000 years and 100 years, respectively (Caltrans, 2013a). The fault offsets that should be considered for the preliminary design as discussed in Section 3.2 are associated with the SEE seismic event. Fault offsets that may be associated with the FEE seismic event have not been defined. A summary of the fault design parameters for the SEE event is presented in Table 1.

3.1.2 LRT Alternative

Metro uses “Important Transit Facility” for the LRT Alternative classification. Two levels of seismic event, consisting of Maximum Design Earthquake (MDE) and Operating Design Earthquake (ODE), are to be considered for the LRT alternative (Metro, 2012). The definitions and associated performance criteria for these two levels of design seismic event are provided in CH2M HILL (2014).

According to Metro’s seismic design criteria, the MDE and ODE requirements must be satisfied for fault offsets that have an average return period of 2,500 years and 150 years, respectively. The fault offsets that should be considered for the preliminary design as discussed in Section 3.2 are associated with the MDE seismic event. Fault offsets that may be associated with the ODE seismic event have not been defined. A summary of the fault design parameters for the MDE event is presented in Table 1.

3.2 Fault Characteristics

Key features and characteristics of the active and potentially active faults relevant to the conceptual design of the fault crossings for the Freeway Tunnel and LRT Alternatives can be summarized as follows:

- **Raymond Fault.** This is the primary active fault crossed by both the Freeway Tunnel and LRT Alternatives. This north-dipping, east-west trending fault has a dominant left-lateral sense of offset, although some north side up-reverse slip is also likely. The ratio of the horizontal to vertical slip is estimated at about 5:1 (L:V). The 13-mile-long (21 km) fault zone is estimated to be up to 82 feet (25 m) wide where it intersects the Freeway Tunnel and LRT alignments at an angle close to 90 degrees. Based on the Caltrans guidelines for the SEE event (Caltrans, 2013a; CH2M HILL, 2014), the Freeway Tunnel Alternative should be designed for an estimated left-lateral (horizontal) offset of 1.64 feet (0.5 m) and a vertical reverse offset of 0.33 foot (0.1 m). For the LRT Alternative, Metro seismic criteria for the MDE (Metro, 2013; CH2M HILL, 2014) indicates the tunnel must be designed for a left-lateral (horizontal) offset of 3.28 feet (1.0 m) and a vertical reverse offset of 0.66 foot (0.2 m).

- **Eagle Rock and San Rafael Faults.** Both the Eagle Rock and San Rafael faults are expected to intersect the Freeway Tunnel Alternative, whereas only the San Rafael fault is expected to be crossed by the LRT

Alternative. The 7-mile-long (11 km) fault zones are estimated to be up to 164 feet (50 m) wide where they intersect the Freeway Tunnel and LRT Alternatives at an angle of approximately 70 degrees. Based on the Caltrans guidelines (Caltrans, 2013a), these fault zones are not subject to the fault offset mitigation requirements (in terms of the level of fault offset considerations). However, for the preliminary design purposes, a left-lateral fault offset of 1.64 feet (0.5 m) and a vertical reverse offset of 0.82 foot (0.25 m) are estimated for these fault zones where they intersect both the Freeway Tunnel and LRT Alternatives (Table 1). It is noted that this is likely associated with a 10,000+ year seismic event scenario (CH2M HILL, 2014), which conservatively satisfies both Caltrans and Metro criteria. There are insufficient data to preclude such event at present and it has therefore been addressed in this preliminary/conceptual design phase, however it is expected that the design criteria associated with these faults will be revisited and updated in the future design phases.

For preliminary design purposes, it is assumed that the fault offsets defined above for the design could occur on a single strand or plane of the fault (CH2M HILL, 2014).

4 Effect of Fault Offsets on Tunnel Structures

Earthquake-induced ground motions cause ground shaking and deformations and could result in ground failure (Hashash et al., 2001). Ground failure consists of liquefaction, slope instability, lateral spreading, and fault offset. This TM focuses on the effect of fault offsets on the tunnel sections of the Freeway Tunnel and LRT alternatives. The effect of ground shaking and the seismic design considerations are addressed in the *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* TM (JA, 2014d) and the *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* TM (JA, 2014e) for the Freeway Tunnel and LRT Alternatives, respectively.

At the location where a tunnel crosses an active earthquake fault, the fault offset would induce various stresses, such as shear stresses, tensile stresses, and compressive stresses in the tunnel lining, resulting in shear and tensile cracking, spalling, and crushing of the concrete in linings that aren't designed for fault offset (see Figure 3). A review of the few available documented case histories of tunnels that have experienced fault offset where no special seismic section was designed. The relevant cases include the Southern Pacific Railroad tunnels, the Longxi Tunnel in China, and the Inatori Tunnel in Japan (Kupfer et al, 1955, Hashash et al., 2001; Li, 2012; Shimizu et al., 2007). These case histories generally indicate the following:

- When tunnels with a circular or nearly circular shape and a good quality reinforced concrete lining are subjected to small discrete offsets (i.e., less than about 1 to 2 feet), they would experience severe cracking and spalling of the lining but not overall collapse, as illustrated in Figure 4, if they are not designed for offsets.

- When tunnels are subjected to larger offsets (greater than about 5 feet) and the linings are not designed for offset, major cracking of the lining and collapse of the lining are possible, if not probable, as illustrated in Figure 5. In addition, significant lengths of lining on either side of the fault offset zone would probably be heavily cracked and damaged and may require replacement.

5 Preliminary Fault Crossing Concept and Construction Methods

5.1 Preliminary Fault Crossing Concept

As discussed in Section 3.2, the design horizontal and vertical offsets range from 1.64 to 3.28 feet (0.5–1.0 m) and from 0.33 to 0.66 feet (0.1 to 0.2 meter), respectively, for the Raymond fault and are 1.64 feet (0.5 m) and 0.82 foot (0.25 m), respectively, for the Eagle Rock and San Rafael faults (see Table 1). These potential offsets would induce significant stresses in the tunnel linings, resulting in cracking and deformation (shearing) of the linings. To minimize the damage, special design features must be incorporated into the design to accommodate the anticipated ground offsets and minimize the potential overstressing in the linings.

- Minimize the potential overstressing in the linings: Depending on the anticipated magnitude of fault offsets, degrees of flexibility can be considered for the tunnel lining system in the fault zone. A special lining consisting of joints designed to accommodate some rotation and/or movement would provide flexibility and ductility, allowing the offsets to occur with minimal impact on the tunnel structure and its ability to be serviceable. Additionally, using compressible backfill behind the segments can provide capacity to accommodate part of the offset.

- Accommodate the anticipated ground offsets: For small anticipated fault offsets, a slightly oversized tunnel may be economic where normal tolerances can be utilized to accommodate the movements. To accommodate larger offsets, an oversized tunnel, with either a special lining section or an oversized vault excavation, for the portion of the tunnel within the fault zone and area susceptible to ground rupture would be more effective. This approach has been used previously for several other tunnel projects such as the Claremont Water Tunnel (Wilson et al., 2007), the SFPUC crossing of the Hayward fault with Pipelines 3 & 4 (Cornell University, 2009), the Red Line crossing of the Hollywood fault (Biggert et al., 2000), and is proposed for the BART Berkeley Hills Tunnels seismic retrofit (JA, 2014f).

Because of the magnitude of the design fault offsets for this study (greater than 12 inches), special design features such as the enlarged vault section for the fault crossings are considered to be necessary, and are provided for in the preliminary design concepts developed for this project. For the portion of the tunnel in the fault zone, the tunnel's internal cross section is enlarged to form a special vault section. The vault sections are large enough to accommodate both the horizontal and vertical components of the fault offset and then allow repairs and realignment of the road surface or tracks through the fault zone following a major earthquake that involves fault offset on the designated faults. The linings for the vault sections can be designed to accommodate the ground movement without requiring major repairs to the tunnel linings. There are several conceptual approaches for the lining of the enlarged vault section that could be taken into consideration:

- **Robust Lining**: For this option, the enlarged vault section is designed as structure consisting of a series of robust (strengthened/stiffened) structural rings with circumferential joints between these rings designed to allow slippage in the fault zone, as shown in Figure 6. These structural rings are designed to accommodate the anticipated asymmetric ground loads without collapse while the circumferential joints are designed to allow the lateral, vertical, and longitudinal components of the fault offsets to take place at the joints between the rings (refer to Figure 6). The advantage of this concept is the resulting tunnel structure can not only accommodate differential offsets but also limit the extent of potential damage to the tunnel lining in the fault offset zone. The disadvantage is that the final lining in the vault section becomes a discontinuous structure in the longitudinal direction.

- **Flexible Lining:** For this option, the enlarged vault section is designed as a flexible structure consisting of a relatively thin shotcrete lining anchored with a pattern of rock dowels. The design philosophy is that the rock-dowel-supported lining would allow the offset movement to take place and would require mainly shotcrete repairs after an earthquake. This approach was used on the Red Line crossing of the Hollywood fault (Biggert et al., 2000). This option provides a flexible lining system, but could be subject to collapse or major damage due to fault offset and may require substantial repairs after an earthquake.

- **Segmental Lining with Compressible Backfill:** An alternative to the two approaches discussed above is to use the proposed segmental lining system for the running tunnels with compressible backfill. For this option, an enlarged annular space between the ground and the segmental lining is formed in the vault section by over-excavation during the TBM advance. This enlarged annular space is then backfilled with compressible or crushable materials. The thickness of the annular space would be based on the design fault offsets and the compressibility of the backfill materials. The strength and stiffness of the compressible materials would be selected to be much lower than those of the tunnel segmental lining, but be able to maintain segmental lining support. These materials would be crushed when subjected to the large fault offsets and would absorb most of these offsets so that the displacements of the segmental lining could be minimized. The advantage of this alternative is to maintain structural continuity of the segmental lining in the longitudinal direction. The disadvantage is the need to construct a sufficiently large annular apace between the ground and the segmental lining, which would require excessive over-excavation through special provisions in the TBM design. Identifying and placing an appropriate compressible or crushable material may also be a challenge.

Based on the above discussion, an enlarged vault section with a robust lining system has been chosen as one viable preliminary design concept to move forward with in this study. This approach is expected to require fewer repairs after an earthquake than the other approach and better meets the desired performance criteria. The following sections of this TM are based on this approach and discuss its viability as a fault crossing concept based on the available geotechnical information and fault characteristics. The detailed information regarding the concept presented in the following sections is being used to support the environmental documentation; however, other fault crossing concepts should be evaluated in future phases of this study.

5.1.1 Freeway Tunnel Alternative Vault Section Concept

An oversized vault section excavated and constructed following TBM excavation and lining of the running tunnels was initially considered for the Freeway Tunnel conceptual vault section. However, the large size of the excavation (over 60 ft in diameter) and the generally poor ground conditions in and around the faults raised constructability issues as well as risk, cost and schedule implications while performing the oversized excavation work, and therefore other approaches were evaluated. Subsequently, a vault section utilizing a steel segmental lining was determined to be more cost effective and less risky than an oversized vault excavation. This approach is feasible because the magnitude of design offsets is relatively small compared to the thickness of the precast concrete segmental lining and therefore recommended as the preliminary design concept for the Freeway Tunnel fault crossings (refer to drawings in Attachment A).

This oversized special vault section is designed to accommodate the large design offsets (see Table 1). In this special vault section, prefabricated steel segments are installed during the TBM excavation and serve as both the initial ground support and the tunnel final lining. These segments would have a thickness of 20 inches, as compared to the 30-inch thick precast concrete segments used for the remainder of the bored tunnel. The outside diameter of the steel segmental lining is the same as that of the normal tunnel lining formed by the precast concrete segments installed outside the fault zones. However, the inside diameter of the vault ring is 55.2 feet, which is 1.7 feet (20 inches) larger than the normal ring. One advantage of this special vault section is that the steel segmental lining is installed in the tail shield of the TBM and the segments do not have to be removed later to construct an oversized vault. Therefore, use of the steel segmental lining in the vault section would substantially minimize the potential risks in terms of construction safety, schedule, and cost in comparison to an oversized vault-type excavation. These steel segments could be used in conjunction with a compressible backfill behind the segments to provided added capacity to accommodate movement.

Peck et al. (1972) define the flexibility ratio (F) for tunnel linings which compares the lining stiffness to the ground stiffness. When the flexibility ratio less than about 10 the lining stiffness is much stiffer than the ground and the steel lining is expected to behave essentially as a rigid ring (Peck et al., 1972). The proposed steel segmental lining has a flexibility ratio that ranges from less than one to 5 and is expected to behave as a rigid ring. It is noted that the actual ground conditions such as strength and stiffness in fault zones would be highly variable. The behavior of the steel segmental lining when subjected to the fault offsets is expected to vary significantly over the length of fault zones. The fault crossing concepts will be further investigated to address the effect of these variations in the future design phase.

The internal structure in the vault section is also modified from that for the normal section. The key changes are as follows:

- The width of roadways, measured between the inside surfaces of two walls on either edge of the roadways, on both the upper and lower decks is increased by 20 inches. These widened roadways would accommodate the expected magnitude of fault offset and maintain the required clearance after a major earthquake. The spaces for emergency egress and tunnel system equipment remain unchanged.

- The roadway slabs of both the lower and upper decks are supported on widened corbels. At both edges of the slabs, expansion joints are created between the slabs and the adjacent walls/corbels. These expansion joints are filled with deformable (compressible) materials to allow relative offsets between the slabs and the adjacent walls during a major earthquake.

- Struts are used at the upper deck level to connect the internal structure to the steel segmental lining ring so that the internal structure would move together with the lining ring during a major earthquake to minimize the increase of forces in the internal structure and limit its potential damage. This design concept does not require the internal structure to provide additional support to the tunnel lining in the vault section, similar to the rest of the tunnel section.

- Transverse joints in the internal structure are proposed. These transverse joints are spaced at 12 feet and located to coincide with every other circumferential joint of the steel segmental lining to form an integrated internal and lining structure consisting of a series of individually robust and stiff modules, which would allow them to move relative to each other during a major earthquake. The joint spacing of 12 feet is determined from the constructability perspective though the modules with circumferential/transverse joints at smaller spacing would behave better to accommodate relative movements.

The same vault section concept with steel segments is proposed for all three fault crossings along the Freeway Tunnel Alternative. Along each fault crossing, the vault section is centered on the main fault trace and its length is selected to cover the entire estimated width of the fault zone (CH2M Hill, 2014). On both sides of a fault zone, enlarged transition sections are required to accommodate the need for horizontal and vertical road realignment after an earthquake. The tunnel cross section in the transition sections is identical to that in the fault zone. A step change in the tunnel inside cross section (lining inside diameter) occurs at the intersection between the normal tunnel section and the vault section.

The length of the enlarged transition sections on either side of the fault zone is based on the magnitude of potential fault offsets and the requirements for the tunnel horizontal and vertical realignment at the locations of the fault zones. Table 2 summarizes the proposed lengths of the fault and transition sections at the three fault crossings for the freeway tunnel; the full length of the fault and transitions were provided by CH2M HILL.

The typical tunnel vault section at the Raymond fault crossing for the Freeway Tunnel Alternative is illustrated in Figure 7. Conceptual design details of the proposed special vault sections for the Freeway Tunnel fault crossings are presented on the preliminary design drawings provided in Attachment A. The detailed information regarding the concept presented herein is being used to support the environmental documentation; however, other fault crossing concepts should be evaluated in future phases of this study.

5.1.2 LRT Alternative Vault Section Concept

An oversized vault section is proposed for the LRT tunnel fault crossings in order to accommodate the large design offsets (see Table 1). The key design features for this vault concept are summarized as follows:

- The vault section has an inside diameter of 22 feet, which is 3 feet and 2 inches larger than the normal bored section with an inside diameter of 18 feet and 10 inches. This enlarged cross section would have sufficient room to accommodate the fault offset and track realignment following a major earthquake (refer to drawings in Attachment A).

- The vault section would be constructed using a three-pass approach (see Section 5.2). The final structure would consist of a 36-inch-thick cast-in-place reinforced concrete lining to provide a robust and strengthened ring and also be thick enough to prevent potential exposure of the ground after an earthquake.
- Circumferential joints along the vault section would be spaced every 12 feet within the vault section. This joint spacing is determined from the constructability perspective though the lining ring with circumferential joints at smaller spacing would behave better to accommodate relative movement. The vault section with these joints would behave like a "slinky" structure which would allow relative movement between adjacent rings during a major earthquake.

Based on the approach developed by Peck et al. (1972), the flexibility ratio (F) for the proposed vault section concrete lining is estimated to vary from less than 1 to 3, depending on whether the vault section is located in soil or rock. With a flexibility ratio less than 10, the proposed vault section concrete lining is expected to behave as an essentially rigid ring (Peck et al., 1972). It is noted that the actual ground conditions such as strength and stiffness in fault zones would be highly variable. The behavior of the final lining when subjected to the fault offsets is expected to vary significantly over the length of fault zones. The fault crossing concepts will be further investigated to address the effect of these variations in the future design phase.

Similar to the vault section proposed for the freeway tunnel fault crossings, the vault section is centered on the main fault trace and its length is selected to cover the entire width of the fault zone. On both sides of the vault, enlarged transition sections are required to accommodate the need for horizontal and vertical rail realignment after an earthquake. Over the length of the transitions, the tunnel cross section is varied in a series of steps (or a single step) to transition from the size of the normal bore to the size of the vault cross section.

The length of the enlarged transition sections is based on the magnitude of potential fault offsets and the type of tunnel (light rail), which would determine the requirements for the tunnel horizontal and vertical realignment. Table 2 summarizes the proposed lengths of the vault and transition sections at two fault crossings for the LRT Alternative; the transition sections were determined by estimating the length needed to realign the rail track after offset with a reverse, or S-, curve. The typical vault sections at the Raymond fault crossing for the LRT Alternative are illustrated in Figure 8.

Design details of the proposed vault section for the LRT tunnel fault crossings are presented on the drawings provided in Attachment A. The detailed information regarding the concept presented herein is being used to support the environmental documentation; however, other fault crossing concepts such as the use of compressible backfill between the ground and the segmental lining by over-excavation during the TBM advance (see Section 5.1) should be evaluated in future phases of this study.

5.2 Construction Method

5.2.1 Freeway Tunnel Alternative Vault Section

The steel segment linings used in the vault section of the Freeway Tunnel Alternative would be installed in the tail shield of the TBM, similar to the normal precast concrete segments. No additional excavation is required. The radial joints and every other circumferential joint of the finished steel segment linings would be bolted and/or welded to construct the 12-foot long stiffened rings. If required the welding of joints can be carried out behind the TBM trailing gear during its advance and would not be a critical path activity.

The internal structure for the vault section would be installed as part of the internal structure construction for the entire tunnel.

5.2.2 LRT Alternative Vault Section

The vault sections for the LRT alternative fault crossings would be constructed using a three-step approach. The basic construction sequence associated with this approach would include the following steps:

- In the first step, the running tunnels are excavated with a TBM and precast concrete segmental linings with bolted and gasketed joints are installed for tunnel support. These linings also serve as the tunnel final lining outside the vault sections.
- In the second step, commencing after the completion of TBM tunnel excavation, a presupport operation (such as a pipe canopy and/or ground improvement or a combination of both) is implemented, starting from one end of the vault section to stabilize/improve ground conditions. Each ring of the segmental lining installed in the first step is removed, additional soil or rock in the exposed surface is excavated to form an enlarged cross section for the vault, and the initial shotcrete lining is applied to the newly excavated surface of the enlarged tunnel section. The requirements for the presupport and shotcrete lining depend on the ground conditions. The excavation in the second step is carried out sequentially using the Sequential Excavation Method (SEM), also known as the New Austrian Tunneling Method (NATM).
- In the third step, a water/gas proof membrane is installed against the shotcrete lining completed in the second step and a cast-in-place concrete final lining is constructed inside the membrane as the final structure. This lining also serves to protect the membrane from damage.

The proposed general construction method/sequence and requirements of the initial support and the final lining for the vaults of the LRT alternative are shown on drawings in Attachment A to this TM.

6 Preliminary Evaluation of Fault Crossing Concepts

A preliminary design evaluation has been carried out to assess the effectiveness of the proposed fault crossing concepts for both the Freeway Tunnel and LRT Alternatives. This section presents a summary of the methodology, soil/rock parameters, material properties, assumptions, and results of this evaluation.

6.1 Methodology

Numerical analyses were employed for the preliminary design evaluation of the fault crossing concepts. These analyses were performed using the two- and three-dimensional finite-difference programs FLAC (Fast Lagrangian Analysis of Continua) Version 5 (Itasca, 2005) and FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) Version 5 (Itasca, 2012). The effect of the fault offsets on the tunnel structure was evaluated primarily based on the FLAC3D analyses. In these analyses, the forces induced by the fault offsets were estimated in the steel segment lining installed in the freeway tunnel vault section and in the concrete final lining installed in the LRT tunnel vault section. The internal structure to be installed in the freeway tunnel was not considered in the analyses for simplicity, and its performance when subjected to the offsets will be carried out in the future design phase. The FLAC2D analyses were also used to estimate the forces developed in the tunnel linings due to static ground loads. The performance of the linings was then assessed by combining the forces associated with fault offset from the 3D analysis with the forces associated with ground loading from these 2D analyses to determine if the structural design criteria are satisfied.

6.1.1 Fault Offset Analysis

When an active fault undergoes offset (relative) offsets perpendicular to a tunnel, the tunnel cross section at the fault would experience shear deformations in its transverse direction, resulting in longitudinal tension in the tunnel lining. The effect of fault offsets on the tunnel lining is three dimensional (3D) in nature. Therefore, 3D analyses are employed to simulate the fault offsets and estimate their impact on the tunnel lining. These 3D fault offset analyses are performed by imposing a certain relative offset between two blocks of ground in which the tunnel is located. In between these two blocks of ground is a zone of finite width equal to a 1-foot wide simulating a fault strand that is responsible for the fault offsets. This represents Scenario A of the two likely fault offset scenarios shown in Figure 9. The magnitude of imposed offsets is based on the SEE and MDE level earthquakes, as discussed in Section 3.1.1 and 3.1.2, for the Freeway Tunnel and the LRT Alternatives, respectively.

For simplicity, only one tunnel is considered in the fault offset analyses for the twin-bore alternatives. Figures 10 and 11 illustrate typical configurations of the FLAC3D models used in the fault offset analyses for the Freeway Tunnel and LRT Alternatives, respectively.

The effects of in situ stresses and groundwater pressures are not considered in the 3D fault offset analyses. These effects are evaluated in separate static analyses for design of the tunnel final lining. The combined effects from both the in situ stresses and the fault offset are estimated by combining the results from the 2D static analyses with those from the fault offset analyses assuming that the superposition principle is valid for the forces developed in the tunnel final lining. The accuracy of the results estimated based on this assumption may depend on the effect of in situ stresses in terms of the extent of plastic zone developed around the tunnel. However, due to the large design offset considered, its effect would significantly overshadow that from the in situ stresses on the tunnel final lining behavior. Any discrepancy caused by the limitation of this assumption is judged as insignificant based on the results of the 2D analysis and would not affect the conclusions of analyses.

It should also be noted that the free-field strains induced in ground during earthquake ground motions are expected to result in other modes of deformations, such as racking/ovaling, longitudinal compression-extension and bending on the tunnel final lining. The effects caused by the free-field strains are addressed in separate TMs for the Freeway Tunnel and LRT Alternatives (JA, 2014d,e). In general, the fault offset would have a more significant effect on the performance of the tunnel final lining than the free field strains. Therefore, the results from the fault offset analyses are considered dominant for design of the tunnel lining and would determine the design requirements.

Key assumptions used in the fault offset analyses include the following:

- The fault offset can be represented by the relative shear offsets of two adjacent blocks of ground separated by a fault strand of sheared, crushed ground with a width of 1 foot.
- Fault zones are either intensely fractured and sheared or in alluvial soil. The zone of ground surrounding the tunnel is poor quality prior to the fault offsets and would experience large plastic deformation (compressible) during the fault offsets.
- The ground is modeled as elasto-perfectly plastic material, and its behavior is governed by the Mohr-Coulomb strength parameters.
- For the LRT alternative, the presence of a water/gas proof membrane indicates a full slip condition between the tunnel final lining and the initial shotcrete lining should be assumed. With this full slip condition, shear stresses cannot be transferred between the tunnel lining and the ground through their contact.
- For the LRT alternative, the initial shotcrete lining installed during the second step of construction (see Section 5.2) is ignored for this fault offset analysis and considered to be part of the surrounding ground.
- For the twin-bore alternatives, the effect of the adjacent tunnel in terms of fault offset behavior is assumed to be minimal and any interaction between the adjacent tunnels would not have a significant impact on the results. The validity of this assumption can be verified in further evaluations during detailed design.

- The circumferential joints are simulated using structural liner elements assigned with a very low stiffness value representing expansion joint materials.

The ground (soil/rock mass and deformable fault zone) was modeled with conventional solid, continuum elements, and the final lining was modeled with liner elements in FLAC3D. The reason for choosing liner elements instead of shell elements to model the final lining is that an interface between the lining and the ground can be generated automatically with the use of liner elements. The interface properties associated with liner elements were used to simulate the behavior of the contact between the final lining and the initial shotcrete/ground.

To simulate the fault offset, the two blocks of ground surrounding the 1-foot wide fault strand, were prescribed to move with the same amount but in the opposite directions along the fault plane. The resulting total relative offset between these two blocks of soil/rock mass is equal to the design fault offset, as indicated in Table 1. Both the horizontal and vertical components of estimated fault offsets were considered in the analyses.

6.1.2 Static Ground Load Analysis

The static ground load analyses are performed to estimate the static stresses developed in the tunnel lining. For the freeway tunnel vault section, these analyses simulate TBM tunnel excavation and installation of steel segment lining only. For the LRT tunnel vault section, these analyses take into account TBM tunnel excavation and installation of the concrete segments, the vault section construction involving the removal of segments, excavation of the vault, application of the shotcrete initial lining, and construction of the concrete final lining. In order to capture the load transfer from the initial lining to the final lining in the long term under the static ground loading condition, a load-sharing scheme is employed (Sun, 2013). A 75% degradation factor is assumed for the initial shotcrete lining in accordance with Hoek (2002). The typical modeling procedure used in the static ground load analyses is:

- *Stage I:* Establish initial stress condition prior to excavation.
- *Stage II:* Simulate the TBM excavation and segment installation.
- *Stage III* (LRT alternative only): Simulate the vault section construction and establish static ground loads in the final lining. This stage involves the following steps: (1) remove the segments and install the shotcrete initial lining; (2) install the interface and the final lining; (3) set shotcrete forces (thrusts and moments) to zero; (4) establish groundwater table; and (5) cycle to equilibrium. This stage simulates the degradation of the initial lining and complete transfer of ground loads carried by the initial lining to the final lining.

The forces in the form of thrusts, shear forces, and bending moments developed in the tunnel lining due to the static loads are then combined with those estimated from the fault offset analyses to determine the combined effect on the tunnel lining when subjected to the fault offsets.

6.2 Design Sections

Two tunnel longitudinal fault crossing sections, one for each of the Freeway Tunnel and LRT Alternatives, were selected for the fault offset analyses. These vault sections are located at the Raymond fault crossings for the respective alternatives. The results from the analyses of these sections are applicable to either the San Rafael fault or the Eagle Rock fault since the anticipated fault offsets at these latter fault zones are either similar to or smaller than those at the Raymond fault zone (see Table 1). As the proposed fault crossing concept for all the fault zones considered is the same for each of the alternatives, application of the design for the Raymond fault zone to other fault crossings is reasonable. Figures 7 and 8 show the typical longitudinal sections used for the Freeway Tunnel and the LRT Alternatives, respectively.

6.3 Design Loads, Load Factors, and Load Combinations

The applicable codes, standards, and guidelines for the tunnel design of the Freeway and the LRT alternatives are presented in *Preliminary Design Concepts for the Freeway Tunnel and Cross Passage Linings* (JA, 2014d) and *Preliminary Design Concepts for the LRT Tunnel and Cross Passage Linings* (JA, 2014e), respectively. Other design criteria and requirements related to the service life and durability, loads, load factors, and load combinations are also discussed in the respective TMs. Refer to these TMs for additional details.

6.4 Inputs to Analysis

Key inputs to the fault offset and static ground load analyses include:

- Soil and rock mechanical properties
- Properties of steel and concrete segments, initial shotcrete lining and final lining at the fault/vault sections
- Soil and rock shear (S) and primary (P) or compressional wave velocities and associated dynamic moduli
- Design seismic ground motion parameters and associated fault offsets

6.4.1 Soil and Rock Properties

Soil and rock mass properties were determined from values presented in *Tunnel Ground Characterization* (JA, 2014b). Tables 3 and 4 present the soil and rock mass properties, respectively. These are properties used for the static ground load analyses. The Alluvium lower bound and Tt-2 parameters were used for the so-called “crushed zone” in soil and rock, respectively.

A portion of the LRT tunnel vaults would be excavated in soil at the Raymond fault and it is assumed that ground improvement would be required to stabilize these soils and control ground movements. The mechanical properties assumed for the treated soil deposits are presented in Table 5.

6.4.2 Initial and Final Lining Properties

Table 6 summarizes the material properties assumed for the steel and concrete segments, shotcrete initial lining and concrete final lining proposed for both the Freeway Tunnel and LRT Alternatives.

6.4.3 Soil and Rock Shear Wave Velocities and Dynamic Moduli

Table 7 presents the assumed shear wave velocities of the soil and rock of interest (CH2M HILL, 2014). These shear wave velocity values are associated with small strain (generally less than 0.001%) conditions, and are called the small-strain shear wave velocities in this TM. The corresponding maximum shear moduli (Gmax) of soil and rock can be estimated based on the small-strain shear wave velocities. During strong seismic ground shaking, the soil and rock deposits may experience shear strains much greater than 0.001%. Cyclic loading tests performed on stiff/hard soil samples indicate that the shear moduli of soil and rock may decrease with increase of shear strains due to material nonlinearity and softening, especially at large strain levels (Vucetic and Dobry, 1991; Murphy et al., 1978). The effective stiffness (shear modulus) of soil and rock at high strain levels that occur during seismic ground shaking would be much lower than that experienced under small strain conditions. Use of the small-strain shear wave velocities and the maximum shear moduli in the seismic analysis would lead to an underestimation of the potential ground and tunnel deformations induced by seismic ground shakings. Therefore it is important to use effective shear moduli for the seismic analyses that are estimated from effective shear wave velocities. Project-specific data for the correlations between shear modulus and shear strain are not available. Commonly, the ratio of effective shear wave velocity to small-strain shear wave velocity is assumed to range from 0.6 to 0.8 for soil and 0.8 to 1.0 for rock. In this TM, ratios of 0.7 and 0.8 were assumed for soil and rock, respectively. The soil and rock effective shear moduli, as presented in Table 8, are then back calculated based on the effective shear wave velocities using Equation 1. Further investigations in the future design phase are needed to verify whether the use of effective shear moduli is appropriate for the fault offset analyses.

$$G = \rho C_s^2 \tag{1}$$

Where:

C_s	=	effective shear wave velocity through the ground
G	=	effective shear modulus of ground
ρ	=	soil or rock density

6.5 Results of Analysis

This section summarizes the results of the analyses of the performance of the tunnel lining installed in the fault sections for the proposed fault crossings when subjected to fault offsets. The performance of the tunnel lining was evaluated against the seismic criteria, as discussed in Section 3. In the evaluation, the seismically induced loads were combined with the static loads (soil and rock overburden loads and groundwater pressure if applicable), with a load factor of 1.0.

As indicated in Section 3.2, the Raymond and San Rafael faults are potentially at the contact between alluvium and bedrock. Some of the vault sections would be constructed crossing from soil to rock. So a number of analyses were carried out to address the effects of variations in ground conditions on the vault section tunnel lining performance. Table 8 summarizes the numerical analyses performed in this TM.

As indicated in Section 6.2, the fault offset analyses were carried out only for the fault crossings at the Raymond fault zone. The evaluation for fault crossings at other fault zones will be performed in the future design phase.

6.5.1 Freeway Tunnel Alternative

This section discusses the performance of the freeway tunnel steel segmental lining when it is subjected to fault offsets. To better understand the effect of fault offsets on the performance of the vault section tunnel lining, the results from the 3D fault offset analyses which ignored the effect of in situ stresses and groundwater pressure (static loading conditions) are discussed first. The loads under the combined static and fault offset loading conditions are then discussed to evaluate the anticipated performance of the tunnel final lining.

Effect of Fault Offset Only

The deformed shape of a typical model with a fault offset of 1.64 feet (0.5 m) is shown in Figure 12. This shape is magnified 10 times for better visualization. The entire rock mass in the front moves to the left (shown as negative offset) for 0.82 foot (0.25 m), and that in the back moves to the right (shown as positive offset) for 0.82 feet, resulting in a total relative offset or fault offset of 1.64 feet. The offsets of the materials located within the 1-foot wide “offset” zone vary from negative 0.82 feet to positive 0.82 feet.

Figure 12 also shows a deformed shape (magnified by 10 times) of the tunnel lining following a 1.64-foot fault offset. From this model, it can be noted that the maximum offset between any two adjacent lining rings is less than 0.35 feet (4.2 inches). This maximum offset is much smaller than the thickness (20 inches) of the steel segmental lining indicating that there would be no exposed ground after the offset.

The results suggest that the ground adjacent to the tunnel would experience plastic deformation during the fault offsets, which would result in the ground absorbing significant portions of the ground movement as the very robust lining ring is able to resist some of the movements. Owing to the high stiffness of the tunnel lining, the segmented lining rings could move relative to each other when the circumferential slip joints are modeled (see Figure 12), resulting in a discontinuity in the lining system. This discontinuity would limit not only the magnitude of forces in the longitudinal direction, but also the extent of stress increases in the lining.

The maximum forces (axial forces and bending moments) developed in the tunnel lining of the vault section when it is subjected only to the fault offsets for various cases are summarized in Table 9. The results indicate:

- Use of the steel segment lining system with no circumferential connections (i.e. bolts removed) every other ring in the vault section is expected to significantly reduce the axial forces caused by the fault offsets in the longitudinal direction. However, the axial forces and bending moments in the transverse direction or hoop direction (perpendicular to the tunnel axis) are not predicted to change significantly. This suggests that a stiff, robust lining ring could be designed to withstand these pressures and to minimize its potential overstressing and prevent severe damage.

- The extent of the areas where high forces occur in the lining is predicted to be limited because of the presence of circumferential slip joints with no bolts. Over the length of vault section, the lining behaves as individual rings, which could move relative to each other. Limited forces would be transferred from one ring to the other. This would potentially limit the damage to the tunnel lining to areas adjacent to the zone where fault offsets are anticipated to take place. Any local damage to the lining is expected to be within the vault section. Therefore, the proposed length for the vault section (see Table 2), including the transition sections, appears to be adequate and should be controlled by the requirements of realignment needed following the offsets.

- The magnitude of the relative offsets at each ring is estimated to be less than the ring thickness for all of the cases analyzed, indicating that there would be no exposed ground, i.e. no potential unstable ground to invade the tunnel during a major earthquake. Any potential water ingress into the tunnel following the offsets could occur but can be repaired.

- Should the fault offsets occur entirely within alluvium, the impact to the vault section structure is expected to be similar to when the offsets occur entirely within the bedrock. Based on the geologic profile as shown in Figure 1 for the Freeway Tunnel alternatives, the Raymond and San Rafael fault zones could be present at the contact between the alluvium and the Topanga Formation. The results of the analyses indicate that the behavior of the vault section tunnel lining in terms of the fault offset effect is expected to be generally similar regardless whether the offsets take place in soil or rock.

- For a large-diameter tunnel like the freeway tunnel, it appears that the steel segmental lining system may also experience local distortions, other than whole body movements as complete rings, when subjected to the offsets. These local distortions could result in high stresses occurring in concentrated areas within individual rings, indicating that locally the vault section lining might suffer damage, but the overall lining ring should have adequate capacity to redistribute stresses and prevent collapse.

The fault offset analyses were only performed based on the design offset potentially occurring at the Raymond fault zone. The results of these analyses are applicable to the fault crossing design for the Eagle Rock or San Rafael fault zones as the offset effect in these fault zones are expected to be similar and larger vertical offset at these faults is not expected to result in significant difference in the tunnel lining performance.

Performance for Combined Loading Conditions

Performance of the tunnel lining in the vault section for the combined loading conditions is evaluated by combining the forces caused by both the excavation-induced tunnel deformation and the groundwater from the FLAC2D static analyses with those induced by the fault offset. The FLAC2D static analyses indicate that the forces developed in the steel segmental lining in the transverse (hoop) direction due to the static loading conditions range from approximately 20 to 30 percent of those induced by the assumed design fault offsets, indicating that the performance of the tunnel lining in the vault section is controlled primarily by the magnitude of fault offsets. The results indicate:

- The proposed preliminary fault crossing concept appears to be a viable option for addressing the potential offset effect.

- The fault offsets to be considered result in localized overstressing of the tunnel lining. This overstressing would primarily be the result of high bending moments and could cause local damage to the tunnel lining. However, no tunnel collapse is expected to occur because the overall lining ring would be very robust with adequate capacity to redistribute stresses within the ring to prevent collapse. Inspection and repairs if necessary of the tunnel structure following a major earthquake would be required. To mitigate the potential impacts of overstressing, an increase in steel plate thickness or yield strength may be evaluated in the future design phase.

- As indicated above, the magnitude of relative offsets at each ring is predicted to be less than the ring thickness, indicating that there would be no exposed ground. The tunnel lining design based on this concept is expected to experience some damage, but not collapse, when subjected to the design fault offsets, meeting the seismic performance criteria as discussed in Section 3.

It should be noted that the analyses presented in this TM only address the effect of fault offsets, and do not take into account the effect of other modes of ground deformations such as racking caused by the free field shear strains during earthquake ground motions. These effects were evaluated in a separate TM (JA, 2014d). In general, the effect of fault offset would result in a higher stress levels in the final lining than caused by racking. The results obtained in this TM can be considered an upper bound for the seismic demand from the perspective of the vault section final lining design.

In addition, the effect of fault offsets on the freeway tunnel internal structure is not addressed in this TM. As discussed in Section 5.1.1, the internal structure in the vault section is connected to the steel segmental lining ring to form an integrated internal and lining structure with a series of individually robust and stiff modules. With this structural system, the relative offsets between the steel segmental lining ring and the internal structure are expected to be limited during a major earthquake so significant damage to the internal structure could be prevented.

6.5.2 LRT Alternative

This section discusses the performance of the LRT vault section tunnel lining when it is subjected to the fault offsets. Similar to the discussion presented in Section 6.5.1 for the Freeway Tunnel Alternative, the results from the 3D fault offset analyses which ignored the effect of in situ stresses and groundwater pressure (static loading conditions) are discussed first. The loads under the combined static and fault offset loading conditions are then discussed to evaluate the anticipated performance of the tunnel final lining.

Effect of Fault Offset Only

The deformed shape of a typical model with a fault offset of 3.28 feet (1.0 m) is shown in Figure 13. This shape is magnified by 5 times for better visualization. The entire rock mass in the front moves to the left (shown as negative offset) for 1.64 feet, and that in the back moves to the right (shown as positive offset) for 1.64 feet, resulting in a total relative offset or fault offset of 3.28 feet. The offsets of the materials located within the so-called “offset” zone with a width of 1 foot vary from negative 1.64 feet to positive 1.64 feet.

Figure 13 shows a deformed shape (magnified by 5 times) of the tunnel final lining following a 3.28-foot fault offset. From this model, it can be noted that the maximum offset between any two adjacent final lining rings is less than 0.30 feet (3.6 inches). This maximum offset is much smaller than the thickness (36 inches) of the final lining. The results suggest that the ground adjacent to the tunnel would experience plastic deformation during the fault offsets, which would result in the ground to absorb significant portion of the offsets as the very robust lining ring is able to resist some of the movements.

Owing to the high stiffness of the final lining, the “structurally independent” sectional final lining rings connected by the circumferential expansion joints could move relative to each other when the circumferential slip joints are modeled (see Figure 13), resulting in a discontinuity in the lining system. This discontinuity would limit not only the magnitude of forces in the longitudinal direction but also the extent of stress increases in the final lining.

The calculated maximum forces (axial forces and bending moments) developed in the final lining of the vault section when subjected only to the fault offsets for various cases are summarized in Table 9. The results indicate:

- Use of the sectional lining system with circumferential slip joints in the vault is expected to significantly reduce the axial forces caused by the fault offsets in the longitudinal direction. However, the axial forces and bending moments in the transverse direction (hoop direction) are predicted to increase due to the fault offsets and resulting ground pressures. This suggests that a heavily reinforced, robust lining ring could be designed to withstand these pressures and to minimize its potential overstressing and prevent severe damage.

- The magnitude of relative offsets at each ring is predicted to be much less than the ring thickness for all of the cases analyzed, indicating that there would be no exposed ground, preventing ground inflow during a major earthquake. Any potential water ingress into the tunnel following the offsets could occur but can be repaired.

- The extent of the areas where increased forces occur in the final lining is predicted to be minimized due to the presence of circumferential slip joints. Over the length of vault section, the lining does behave as individual rings that can move independently relative to each other. Limited forces would be transferred from one ring to the other. This would potentially limit the damage to the tunnel lining to areas adjacent to the zone where fault offsets is anticipated to take place. Any local damage to the final lining is expected to be within the vault section. Therefore, the proposed length for the vault section (see Table 2), including the transition sections, appears to be adequate and should be controlled by the requirements of realignment needed following the offsets.

- Should the fault offsets occur entirely within the alluvium, the impact to the vault structure is expected to be similar to when the offsets occur entirely within the bedrock. Based on the geologic profile as shown in Figure 2 for the LRT Alternative, the Raymond and San Rafael fault zones could be present at the contact between the alluvium and the Topanga Formation. The results of the analyses indicate that the behavior of the vault final lining in terms of the fault offset effect is expected to be similar regardless whether the offsets take place in soil or rock.

- For the relatively small-diameter tunnel, like the LRT Alternative, the sectional final lining system is predicted to experience minor local distortions as well as whole body movements, when subjected to the offsets.

The fault offset analyses were performed only based on the design offset potentially occurring at the Raymond fault zone. The results of these analyses are applicable to the vault design for the San Rafael fault zone, but should be considered as an upper bound in terms of the offset effect for that fault zone.

Concrete Lining Subjected to Combined Loading Conditions

Performance of the final lining in the vault section for the combined loading conditions is evaluated by combining these forces with those induced by the fault offset. As indicated above, the concrete lining would be installed in the third step of vault construction (see Section 5.2.2). The excavation-induced tunnel loads are expected to contribute only a small fraction of the loads induced by the fault offsets in the final lining. The stresses developed in the final lining would be caused primarily by the groundwater and degradation of the shotcrete initial lining. As discussed in Section 6.1.2, the majority (about 75%) of ground loads originally carried by the initial lining are assumed to be transferred to the final lining in the long term. In addition, the final lining is assumed to carry the full hydrostatic pressure. With these assumptions, the forces developed in the final lining due to static ground loads and hydrostatic pressure were calculated based on the FLAC2D analyses. These analyses indicate that the forces developed in final lining in the transverse (hoop) direction due to the static loading conditions range from approximately 5 to 20 percent of those induced by the assumed design fault offsets, indicating that the performance of the tunnel final lining in the vault section is controlled primarily by the fault offsets. The results indicate:

- The proposed preliminary fault crossing concept appears to be a viable option for addressing the potential offset effect.
- The fault offsets to be considered results in localized overstressing of the tunnel lining. This overstressing would primarily be the result of high bending moments and could cause cracking in the lining. However, no tunnel collapse is expected to occur because the overall lining ring would be very robust with adequate capacity to redistribute stresses within the ring to prevent collapse. Inspection and repairs if necessary of the tunnel structure following a major earthquake would be required. To mitigate the potential impact of overstressing and cracking, an option for increase of the lining thickness and/or amount of reinforcement may be evaluated in the future design phase.
- As indicated above, the magnitude of relative offsets at each ring is predicted to be less than the ring thickness, indicating that there would be no exposed ground. The tunnel final lining design based on this concept is expected to experience minor damage, but not collapse, when subjected to the design fault offsets, meeting the seismic performance criteria as discussed in Section 3.

It should be noted that the analyses presented in this TM only address the effect of fault offsets, and do not take into account the effect of other modes of ground deformations such as racking caused by the free field shear strains during earthquake ground motions. These effects were evaluated in a separate TM (JA, 2014e). In general, the effect of fault offset would result in higher stress levels in the tunnel lining than those due to racking deformations. Therefore, the results from this TM are considered an upper bound of the seismic demand on the vault section tunnel lining.

7 Post-Earthquake Repairs

The purpose of the special vault sections are to accommodate movement associated with fault offset; however, post-earthquake repairs would be required because the tunnels are expected to experience some degree of damage due to the fault offsets during a major earthquake. Following a major earthquake involving fault offsets on one of the designated faults, the tunnel would likely be temporarily closed for inspection of any damage to structures and utilities. The inspection would also determine whether freeway roadway or LRT track realignment is needed and how much repair work is necessary. The duration of repair work would depend on the degree of damage and type of tunnels. Post-earthquake repairs required for the Freeway Tunnel and LRT alternatives would be different, as discussed below.

Freeway Tunnel Alternative

- For small offsets, significantly less than the design offset, minor damage is expected. Minor damage would consist of cracking and spalling of the concrete linings and small offsets at the circumferential joints between the lining rings. Realignment of the roadway would likely not be required. Water leakage through the circumferential joints may occur and can be cut off by grouting or installation of a drip shield. The tunnels are expected to be available to emergency vehicles within 24 hours after inspection and clean-up. The repair and realignment, if required, may be done during night closures.
- For offsets of the order of the design offset, damage, but no collapse, is expected. The damage may consist of cracking and significant spalling of the concrete linings and horizontal and vertical offsets of the order of 6 inches at the circumferential joints between the lining rings. It is anticipated that realignment

of the roadway would be required. Water leakage through the circumferential joints is expected and can be cut off by grouting or installation of a drip shield. Depending on the degree of damage, the repair work could be staged so one of two tunnels or one of two levels would be restored for service sooner.

LRT Alternative

- For small offsets, significantly less than the design offset, minor damage is expected. Minor damage would consist of cracking and spalling of the concrete linings and small offsets at the circumferential joints between the lining rings. Realignment of the LRT tracks may or may not be required depending on the track tolerance requirements. Water leakage through the circumferential joints may occur and can be cut off by grouting or installation of a drip shield. The tunnels are expected to be available to emergency service within 24 hours after inspection and clean-up. The repair and realignment, if required, may be done during night closures.

- For offsets of the order of the design offset, damage, but no collapse, is expected. The damage may consist of cracking and significant spalling of the concrete linings and horizontal and vertical offsets of the order of 6 inches at the circumferential joints between the lining rings. It is anticipated that realignment of the LRT tracks would be required. Water leakage through the circumferential joints is expected and can be cut off by grouting or installation of a drip shield. Depending on the degree of damage, the repair work could be staged so one of two tunnels would be restored for service sooner.

8 Summary of Preliminary Evaluation and Recommendations

This TM summarizes the preliminary design evaluations of the proposed fault crossing concepts for the Freeway Tunnel and LRT Alternatives. The concepts presented herein appear viable; however, there are also other feasible solutions for these fault crossings which could be further explored in future phases of the design. The evaluations are based on the seismic design criteria from Caltrans and Metro for the Freeway Tunnel and the LRT Alternatives, respectively and seismic hazards assessments carried out by CH2M HILL (2014). The evaluation is based on limited numerical analyses for the fault crossing at the Raymond fault zone only. The key findings from the preliminary design evaluation can be summarized as follows:

Freeway Tunnel Alternative

- The proposed steel segmental lining within the fault zones allows the fault offsets to be accommodated without having to remove the lining and overexcavate the tunnel to construct a vault. This significantly reduces the risk involved with construction of the fault crossing and also reduces the cost and construction schedule.

- Use of the segmented tunnel lining system in the vault section is expected to significantly reduce the axial forces caused by the fault offsets in the longitudinal direction. To minimize potential overstressing in the transverse (hoop) direction, very robust lining rings with thick steel plates or high strength steel for steel segments should be designed for this system.

- The extent of the areas where increased forces occur in the tunnel lining is predicted to be limited within the length of the vault section. Therefore, the proposed length of the vault section, including that of the transition sections, appears to be adequate and should be controlled by the requirements of realignment needed following the offsets.

- The proposed preliminary fault crossing concept appears to be a viable option for mitigating the effect of potential fault offsets. The tunnel lining that is designed based on this concept is expected to experience limited damage, but not collapse, when subjected to the design fault offsets, meeting the Caltrans' seismic performance criteria.

- The need for post-earthquake repair would depend on the magnitude of actual offsets experienced and degree of damage to structures and utilities.

LRT Alternative

- Constructing the vault section for the LRT alternative allows the higher fault offset to be accommodated. The construction risk is acceptable because the tunnel is small and also close to the ground surface which provides better access for ground improvement measures.

- Use of the segmented tunnel final lining system in the vault section is expected to significantly reduce the axial forces caused by the fault offsets in the longitudinal direction. To minimize potential overstressing in the transverse (hoop) direction, very robust lining rings with heave reinforcement for concrete lining should be designed for this system.

- The extent of the areas where increased forces occur in the tunnel final lining is predicted to be limited within the length of the vault section. Therefore, the proposed length of the vault section, including that of the transition sections, appears to be adequate and should be controlled by the requirements of realignment needed following the offsets.

- The proposed preliminary fault crossing concept appears to be a viable option for mitigating the effect of potential fault offsets. The tunnel final lining that is designed based on this concept is expected to experience limited damage, but not collapse, when subjected to the design fault offsets, meeting the Metro's seismic performance criteria.

- The need for post-earthquake repair would depend on the magnitude of actual offsets experienced and degree of damage to structures and utilities.

The following recommendations are made based on the findings of this TM:

- The findings presented in this TM are preliminary and are based on limited geotechnical data and seismic hazards information relevant to the fault offsets. Further evaluations in the future design phase are required when additional geotechnical data and seismic information become available in order to reduce uncertainties associated with the design inputs and assumptions.

- The proposed fault crossing concept is evaluated based on some critical assumptions regarding the physical properties, such as the presence of a compressed zone adjacent to the vault final lining. These assumptions involve significant uncertainties. To validate the proposed design concept, additional investigations are required to verify and justify these assumptions.

- Other fault crossing concepts, including but not limited to the use of low density cellular concrete (compressible) backfill around the vault/tunnel lining should also be investigated and compared with the proposed concept in order to come up with an optimal design for the fault crossings.

9 Limitations

The preliminary design concepts presented in this TM are based on limited geotechnical data that were available (CH2M HILL, 2014). A significant amount of additional geotechnical investigations and data gathering studies would be required in order to advance these design concepts to a complete preliminary design level.

10 References

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11 Revision Log

Revision 0	March 12, 2014	Internal Review
Revision 1	April 25, 2014	Internal Review
Revision 2	April 29, 2014	Metro/Caltrans Review
Revision 3	June 18, 2014	Metro/Caltrans Review
Revision 4	August 22, 2014	Incorporation into Tunnel Evaluation Report

Figures

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NOTES:

1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

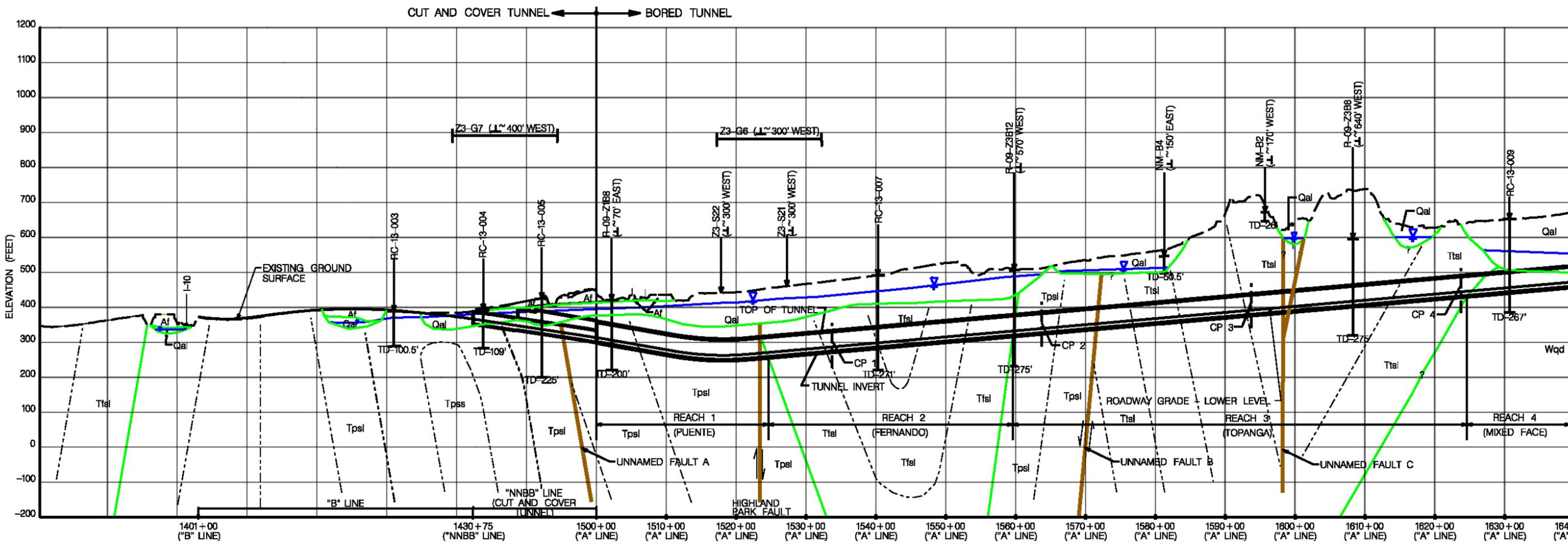
2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/1/2013.

5) PREDOMINATE GEOLOGIC FORMATION WITHIN EACH REACH IS SHOWN IN PARENTHESIS UNDER THE REACH NUMBER. "MIXED FACE" IS DEFINED AS SOIL OVER BEDROCK WITHIN THE FACE OF TUNNEL EXCAVATION.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

- GEOLOGIC CONTACT
- INACTIVE FAULT
- ACTIVE OR POTENTIALLY ACTIVE FAULT
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FIGURE 1
FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

Figure 1. Freeway Tunnel Vertical Alignment and Geologic Profile (Sheet 1 of 2)

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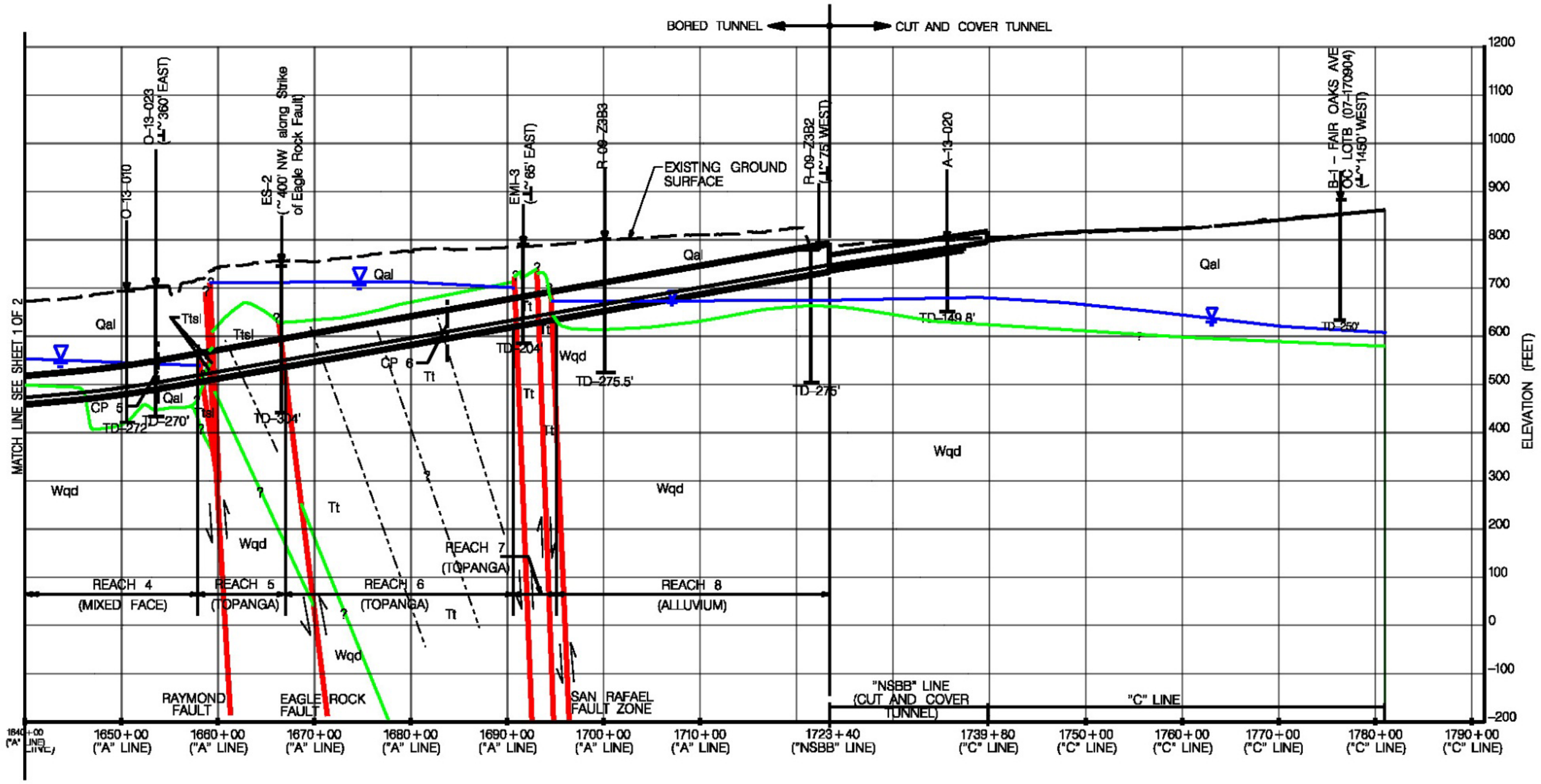

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FIGURE 1
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SHEET 2 OF 2

Figure 1. Freeway Tunnel Vertical Alignment and Geologic Profile (Sheet 2 of 2)

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Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative

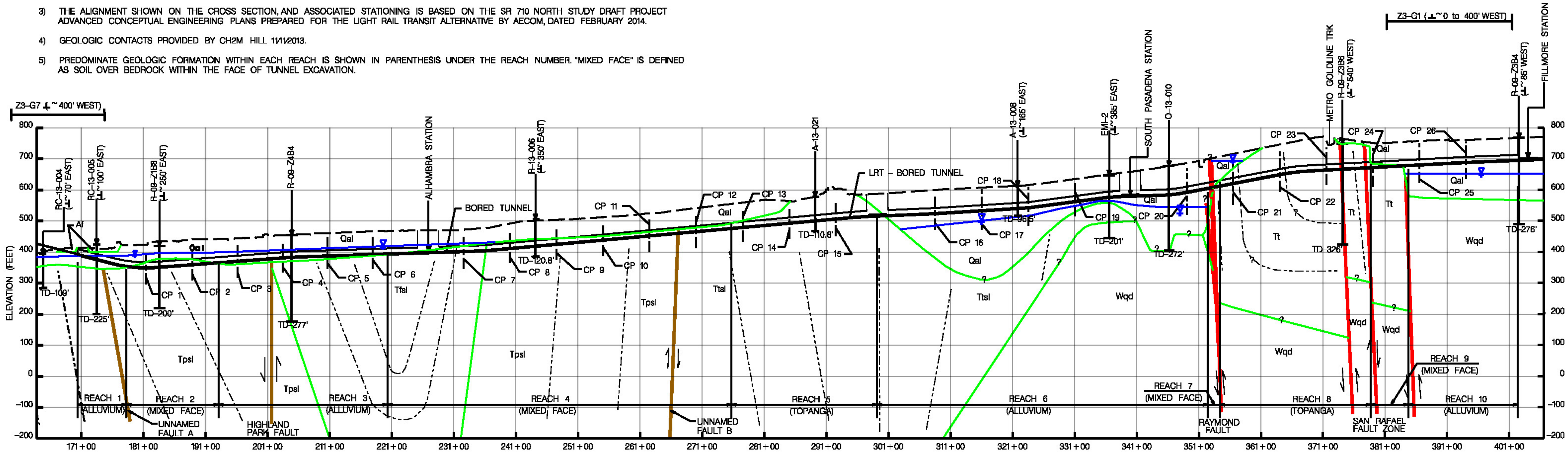

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FIGURE 2
LRT GEOLOGIC PROFILE

Figure 2. LRT Tunnel Vertical Alignment and Geologic Profile

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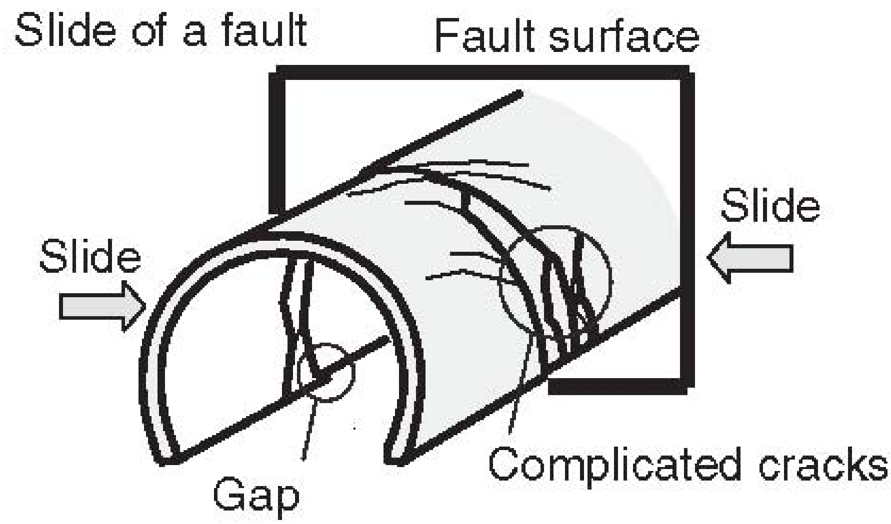


Figure 3. Effect of Fault Offsets on Tunnel Lining

Figure 4. Shear Damage to Section of Longxi Highway Tunnel in China, Subject to about 1 m Fault Offset, following the 2008 Wenchaun Earthquake (Li, 2012)

Figure 5. Collapsed Section of the Longxi Tunnel, China following the 2008 Wenchaun Earthquake (Li, 2012)

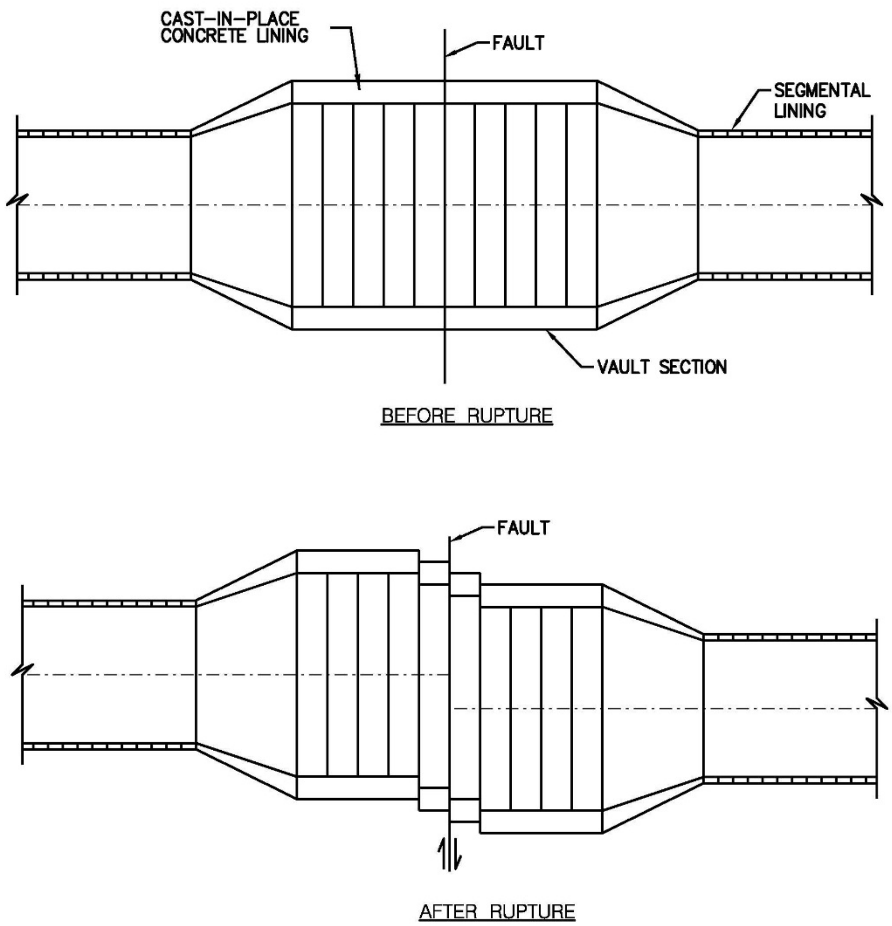


Figure 6. Plan Layout of the Vault, Demonstrating the Slinky Articulation Concept (JA, 2014f)

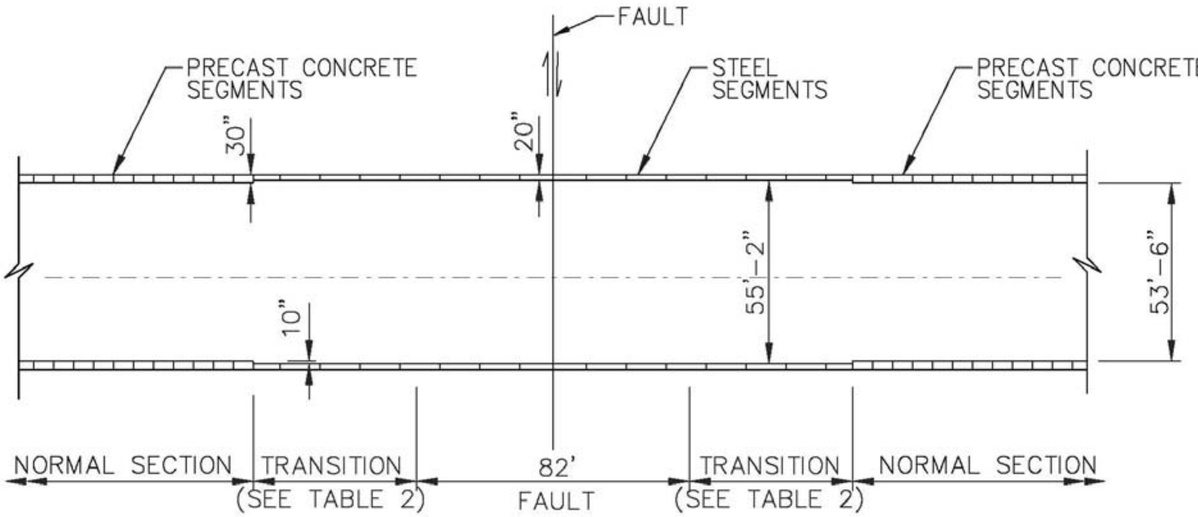


Figure 7. Vault Section at Raymond Fault Zone for Freeway Tunnel Alternative

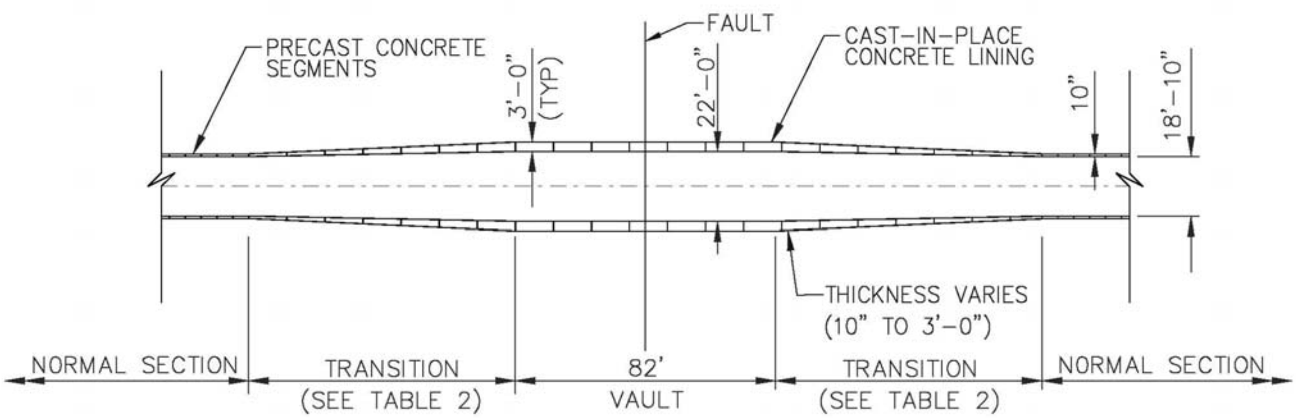
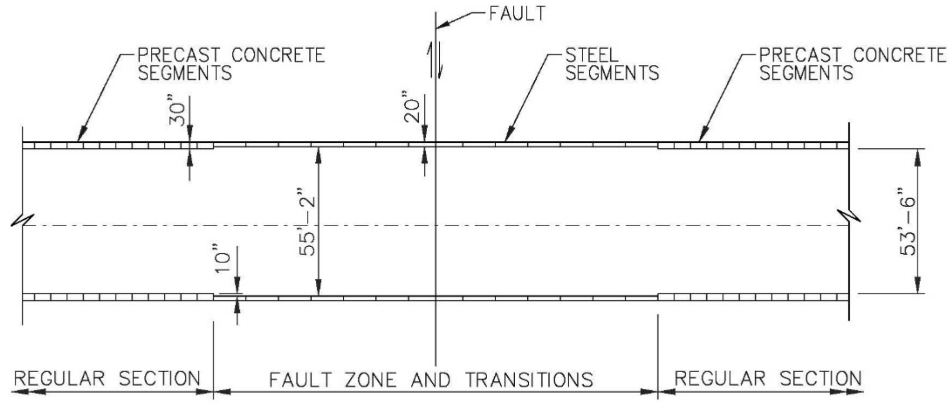
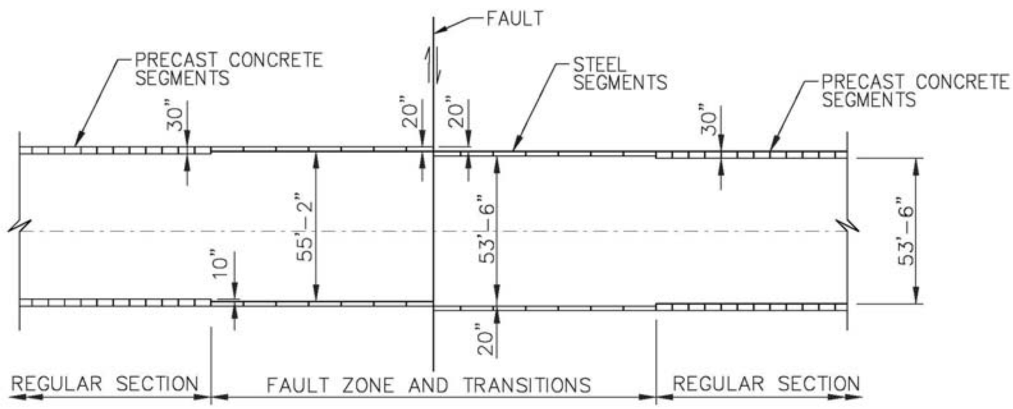


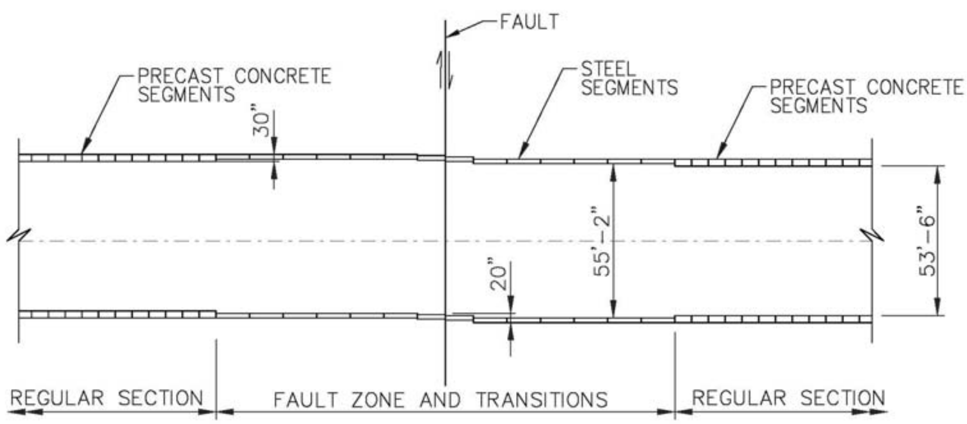
Figure 8. Vault Section at Raymond Fault Zone for LRT Alternative



(a) Before offset movement



(b) Scenario A: Slip on a single plane/joint



(c) Scenario B: Offset over a zone

Figure 9. Assumed Fault Offset Scenarios (Showing Freeway Tunnel Fault Crossing Concept)

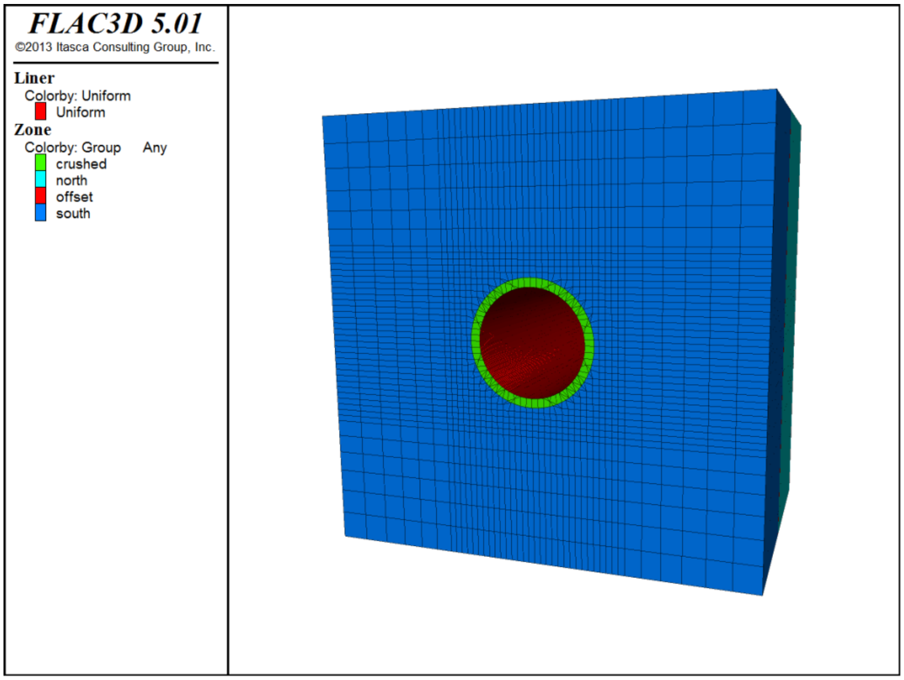


Figure 10. Typical Configuration of a Fault Offset Model for Freeway Tunnel Alternative

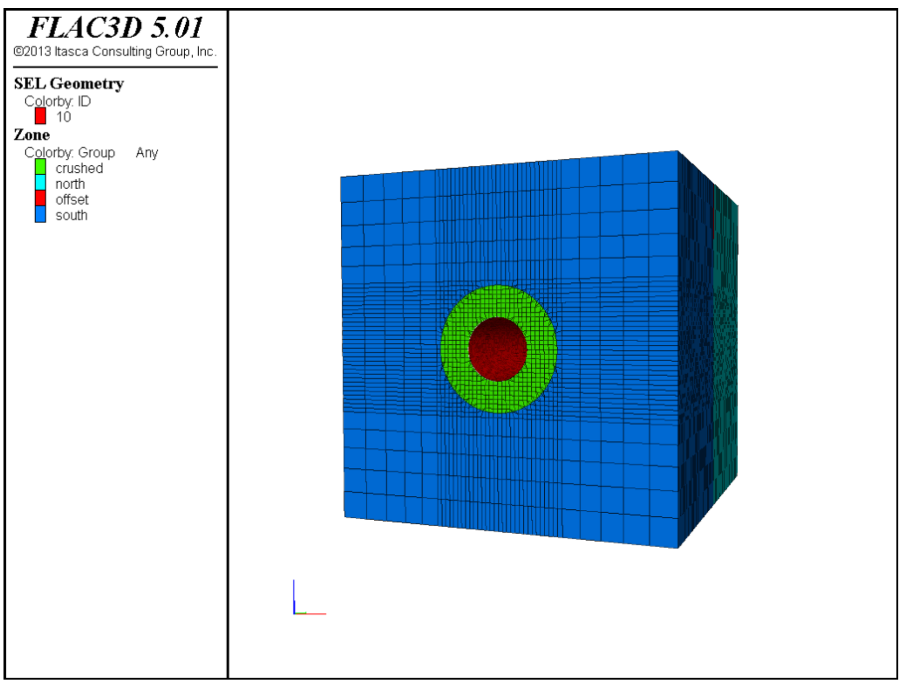


Figure 11. Typical Configuration of a Fault Offset Model for LRT Alternative

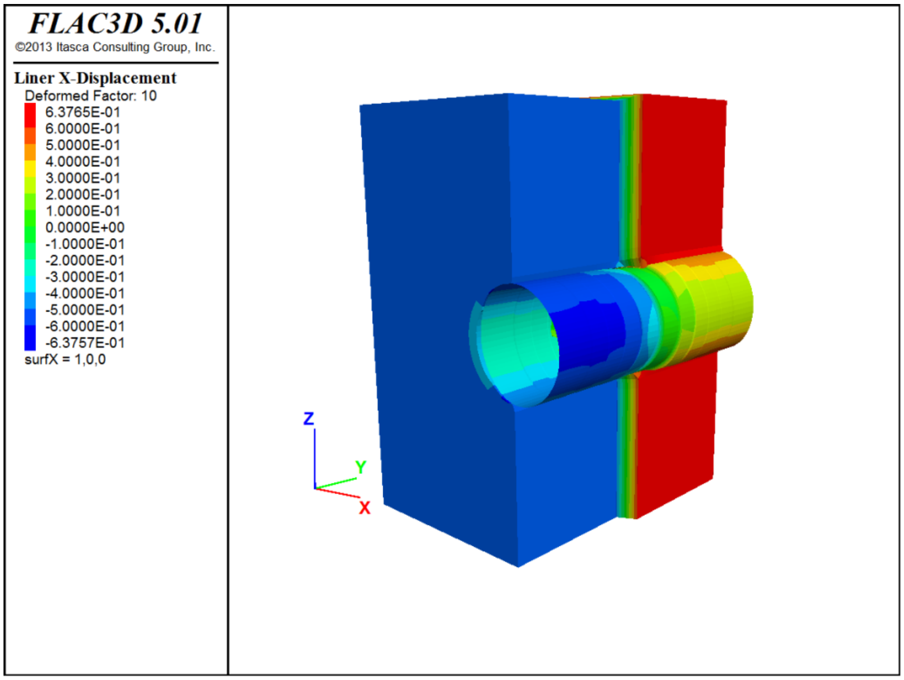


Figure 12. Deformed Shape (Magnified) of Fault Steel Final Lining for Freeway Tunnel Alternative

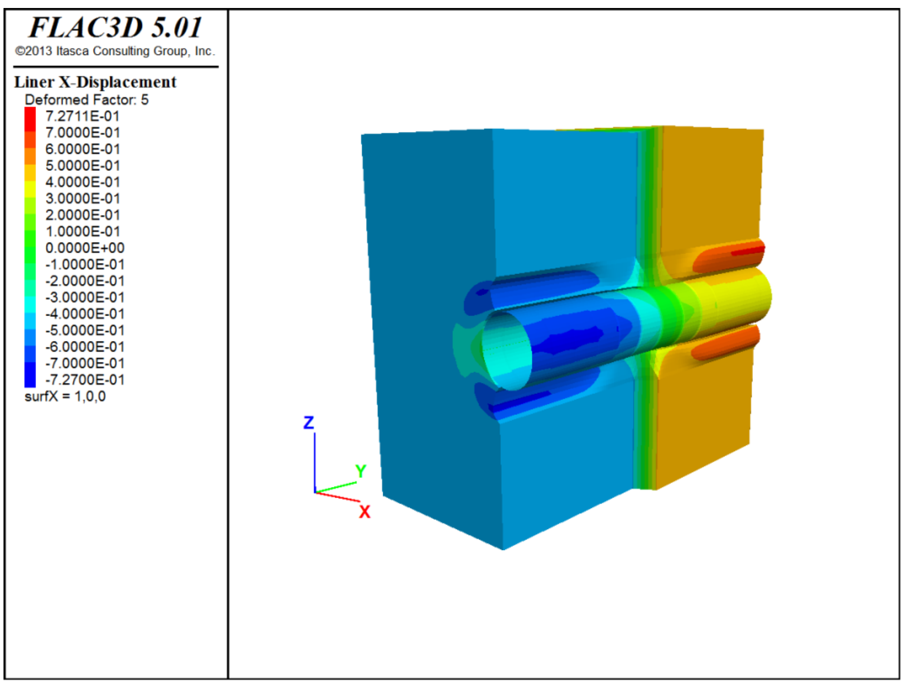


Figure 13. Deformed Shape (Magnified) of Vault Concrete Final Lining for LRT Alternative

Tables

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Table 1. Summary of Estimated Design Fault Offsets (Horizontal/Vertical) *

Alternative	Raymond Fault 82 ft wide (25 m)	San Rafael Fault 164 ft wide (50 m)	Eagle Rock Fault 164 ft wide (50 m)
Freeway Tunnel Alternative	1.64/0.33 ft (0.5/0.1 m) §	1.64/0.82 ft (0.5/0.25 m) §	1.64/0.82 ft (0.5/0.25 m) §
LRT Alternative	3.28/0.66 ft (1.0/0.2 m) §	1.64/0.82 ft (0.5/0.25 m) §	N/A (no crossing)

* Source: CH2M HILL, 2014.

§ It is estimated that 75 to 100 percent of offset would occur on a single (main) fault strand.

Table 2. Length of Vault Sections for Freeway Tunnel and LRT Alternatives

Fault	Length of Vault Section					
	Freeway Tunnel Alternative			LRT Alternative		
	Fault (ft)	Transitions (ft)	Total (ft)	Fault (ft)	Transitions (ft)	Total (ft)
Raymond Fault 82 ft wide (25 m)	82	2 X 25	132	82	2 X 80	250
San Rafael Fault 164 ft wide (50 m)	164	2 X 25	214	164	2 X 80	330
Eagle Rock Fault 164 ft wide (50 m)	164	2 X 25	214	N/A (No Crossing)		

Table 3. Recommended Soil Parameters for Preliminary Design Evaluation

Soil Type	Range	Total Unit Weight (pcf)	Deformation Modulus (ksi)	Poisson's Ratio	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Horizontal-to-vertical Stress Ratio (K_0) [Range]
Fill	Mean	120	2.0	0.30	0	32	0.5
Alluvium	LB*	125	6.9	0.35	0	32	0.6 [0.4–1.2]
	Mean		13.9		500	36	

* LB=Lower Bound.

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Table 4. Recommended Rock Mass Parameters for Preliminary Design Evaluation

Geologic Formation	Range	Total Unit Weight (pcf)	Average Applicable Depth (ft)	GSI Classification Parameters *				Hoek-Brown Model Parameters *			Equivalent Mohr-Coulomb Model Parameters		Deformation Modulus (ksi)	Poisson's Ratio	Horizontal-to-vertical Stress Ratio (K_0)
				Intact Rock UCS (psi)	GSI	mi	D	m_b	s	a	Cohesion (psf)	Friction Angle (degrees)			
Fernando Formation, Tf	LB	136	150	50	N/A						1,050	20	10	0.35	0.65 (0.5 – 0.8)
	Mean			300							1,950	29	25		
Puente Formation, Tp-2	LB	134	50	30	35	6	0	0.589	0.0007	0.516	250	17	10	0.30	0.7 (0.5 – 1.35)
	Mean			50	45	6	0	0.842	0.0022	0.508	400	22	15		
Puente Formation, Tp-1	LB	134	150	150	45	10	0	1.403	0.0022	0.508	1,300	26	35	0.30	0.7 (0.5 – 1.35)
	Mean			400	55	10	0	2.005	0.0067	0.504	2,300	36	70		
Topanga Formation, Tt-2	LB	134	50	30	40	6	0	0.704	0.0013	0.511	300	17	10	0.30	0.7 (0.5 – 1.35)
	Mean			60	50	6	0	1.006	0.0039	0.506	450	24	15		
Topanga Formation, Tt-1	LB	134	150	230	50	12	0	2.012	0.0039	0.506	1,800	32	40	0.30	0.7 (0.5 – 1.35)
	Mean			500	60	12	0	2.876	0.0117	0.503	2,950	41	100		
Basement Complex Rock, Wqd-2	LB	158	50	35	30	25	0	2.052	0.0004	0.522	450	25	15	0.25	0.5 (0.4 – 0.6)
	Mean			80	45	25	0	3.506	0.0022	0.508	800	35	20		
Basement Complex Rocks, Wqd-1	LB	158	150	250	50	25	0	4.192	0.0039	0.506	2,600	37	50	0.25	0.5 (0.4 – 0.6)
	Mean			680	60	25	0	5.991	0.0117	0.503	4,350	48	130		

Notes: * UCS=Uniaxial Compressive Strength; GSI=Geological Strength Index; mi=Hoek-Brown constant related to rock type and lithology; D=Disturbance Factor which depends upon the degree of disturbance caused by excavation; mb, s, and a=Hoek-Brown constants related to rock mass strength and characteristics.

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Table 5. Treated Soil Parameters

Total Unit Weight (pcf)	Compressive Strength (psi)	Deformation Modulus (ksi)	Poisson's Ratio	Effective Cohesion (psi)	Effective Friction Angle (degrees)
140	400	50.0	0.25	104	35

Table 6. Material Properties of Segmental Lining, Initial Shotcrete Lining, and Concrete Final Lining

Alternative	Lining Type	Thickness (inches)	Strength (psi)	Elastic Modulus (ksi)	Poisson's Ratio
Freeway Tunnel	Shotcrete Initial Lining	24	5,000	2,175	0.2
	Concrete Final Lining	48	5,000	4,030	
	Steel Segments *	20	50,000	29,000	0.3
LRT	Shotcrete Initial Lining	12	5,000	2,175	0.2
	Concrete Final Lining	36	5,000	4,030	
	Concrete Segments	10	7,000	4,800	

* Total height of steel section.

Table 7. Soil and Rock Mass Dynamic Parameters for Seismic Design

Soil / Rock Formation	Total Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Primary Wave Velocity (ft/sec)	Effective Shear Wave Velocity * (ft/sec)	Effective Dynamic Shear Modulus (ksi)	Effective Dynamic Young's Modulus (ksi)
Fill	120	500	816	350	3.2	7.6
Old Alluvium (<50 ft Deep)	125	1,080	2,248	756	15.4	41.6
Old Alluvium (>50 ft Deep)		1,650	3,435	1,155	36.0	97.1
Fernando Formation	136	1,080	2,248	864	21.9	59.1
Puente Formation (Weathered) (Tp-2)	134	1,600	2,993	1,280	47.3	123.1
Puente Formation (Tp-1)		2,200	4,116	1,760	89.5	232.7
Topanga Formation (Weathered) (Tt-2)	134	1,300	2,432	1,040	31.3	81.3
Topanga Formation (Tt-1)		2,900	5,425	2,320	155.5	404.4
Basement Complex Rocks (Weathered) (Wqg-2)	158	1,600	2,771	1,280	55.8	139.6
Basement Complex Rocks (Wqg-1)		3,500	6,062	2,800	267.1	667.9

* Assumed to be equal to $0.7C_s$ for soil and $0.8C_s$ for rock.

Table 8. Analysis Matrix

Design Alternative	Fault Location and Geologic Unit		Static (Tt-2/Qal)	Offset (Baseline – Without Joints)	Offset (Joint Concept)
Freeway Tunnel	Raymond Fault	In Topanga Formation	√	√	√
		In Alluvium	√	N/A	√
LRT	Raymond Fault	In Topanga Formation	√	√	√
		In Alluvium	√	N/A	√

Table 9. Summary of Calculated Forces in Steel Segmental Lining in Vault Section for Freeway Tunnel Alternative

Fault Location and Geologic Unit		Axial Force * (Transverse) (kips/ft)	Axial Force (Longitudinal) (kips/ft)	Moment ǂ (Transverse) (kips/ft)	Moment (Longitudinal) (kips/ft)
Raymond Fault	Case F1: Baseline Without Joints, in Topanga Formation	-3,272 to 60	-1,216 to 902	-991 to 716	-312 to 353
	Case F2: Joint Concept, in Topanga Formation	-2,225 to 315	-48 to 36	-936 to 706	-235 to 165
	Case F3: Joint Concept, in Alluvium	-2,419 to 534	-36 to 30	-783 to 785	-184 to 155

* Forces are positive in tension and negative in compression.

ǂ Moments are positive when bending inside and negative when bending outside.

Table 10. Summary of Calculated Forces in Final Lining in Vault Section for LRT Alternative

Fault Location and Geologic Unit		Axial Force * (Transverse) (kips/ft)	Axial Force (Longitudinal) (kips/ft)	Moment ǂ (Transverse) (kips/ft)	Moment (Longitudinal) (kips/ft)
Raymond Fault	Case L1: Baseline Without Joints, in Topanga Formation	-498 to 79	-668 to 602	-916 to 1,160	-205 to 287
	Case L2: Joint Concept, in Topanga Formation	-510 to 42	-15 to 21	-907 to 1,176	-114 to 103
	Case L3: Joint Concept, in Alluvium	-708 to 116	-26 to 30	-1,596 to 1,751	-195 to 181

* Forces are positive in tension and negative in compression.

ǂ Moments are positive when bending inside and negative when bending outside.

Attachment A

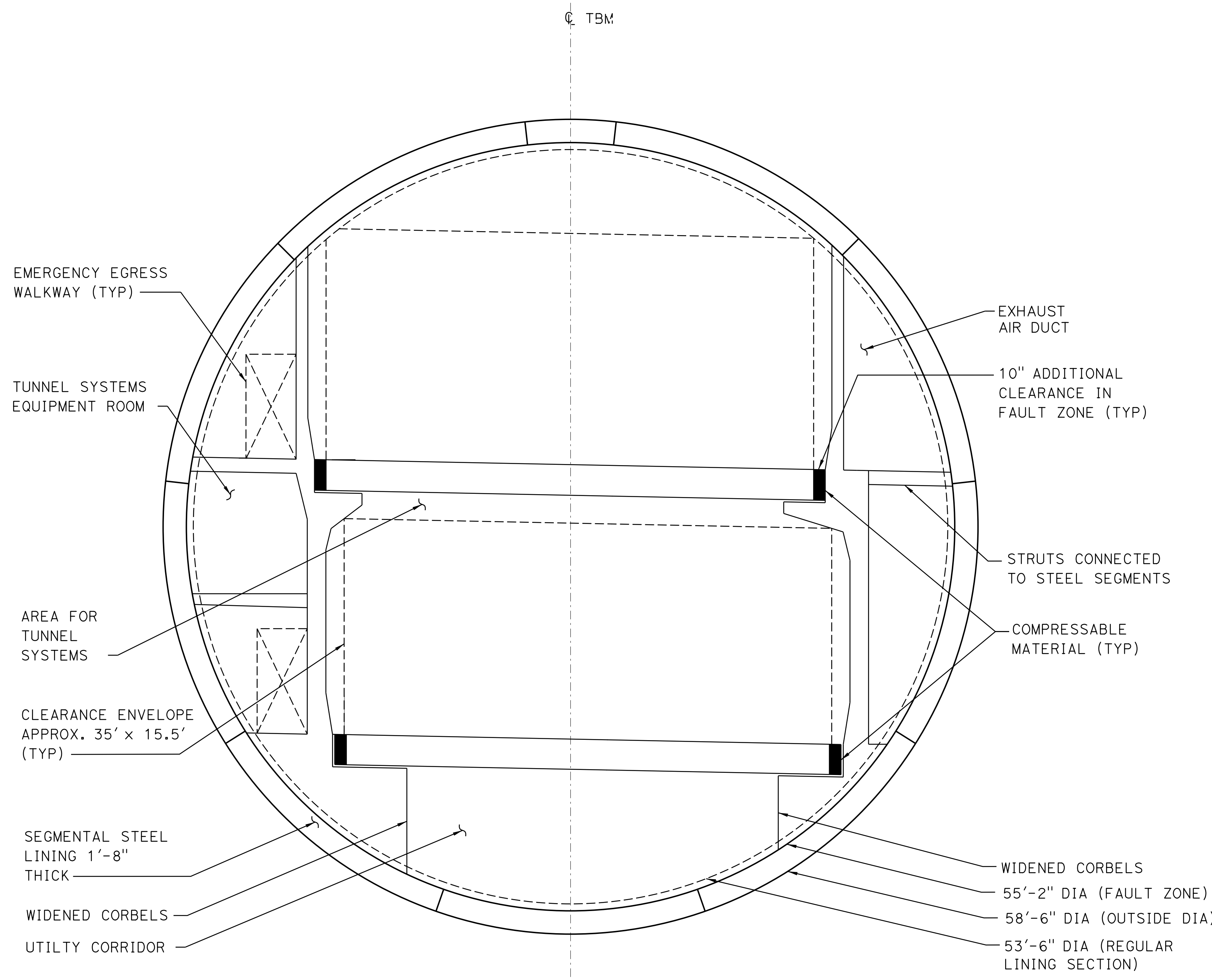
Conceptual Fault Crossing Drawings

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



TYPICAL SECTION AT FAULT CROSSING

S5.01

DESIGNED BY	J. VANGREUNEN/ Y. SUN	DATE	3-03-2014
DRAWN BY	J. TOLES	DATE	3-03-2014
CHECKED BY	S. KLEIN	DATE	3-03-2014
APPROVED	S. DUBNEWYCH	DATE	3-03-2014

X
PROJECT ENGINEER

SR 710 NORTH STUDY
FAULT CROSS SECTION

BRIDGE NO. TBD | UNIT:
SCALE: AS NOTED | PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

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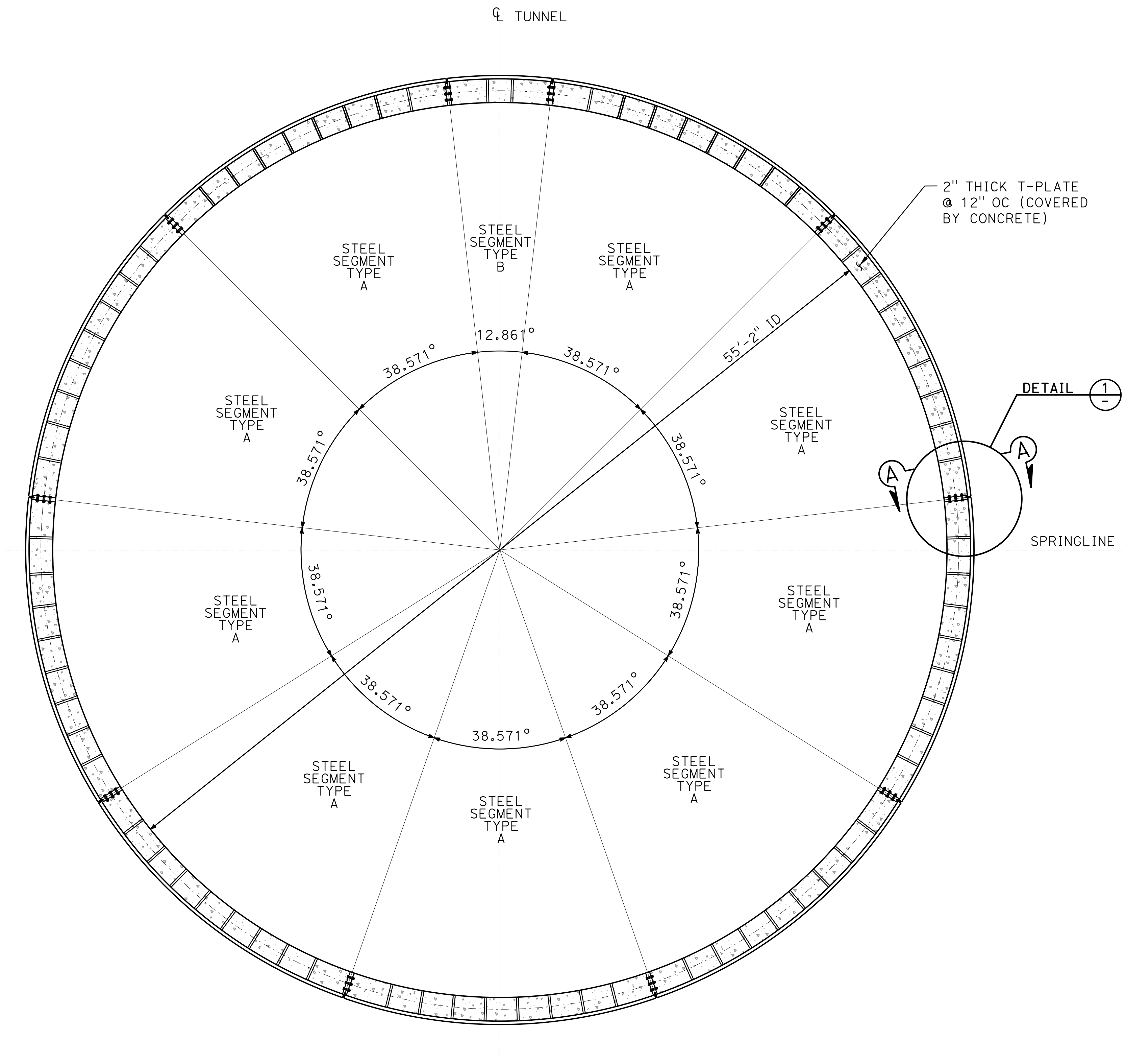
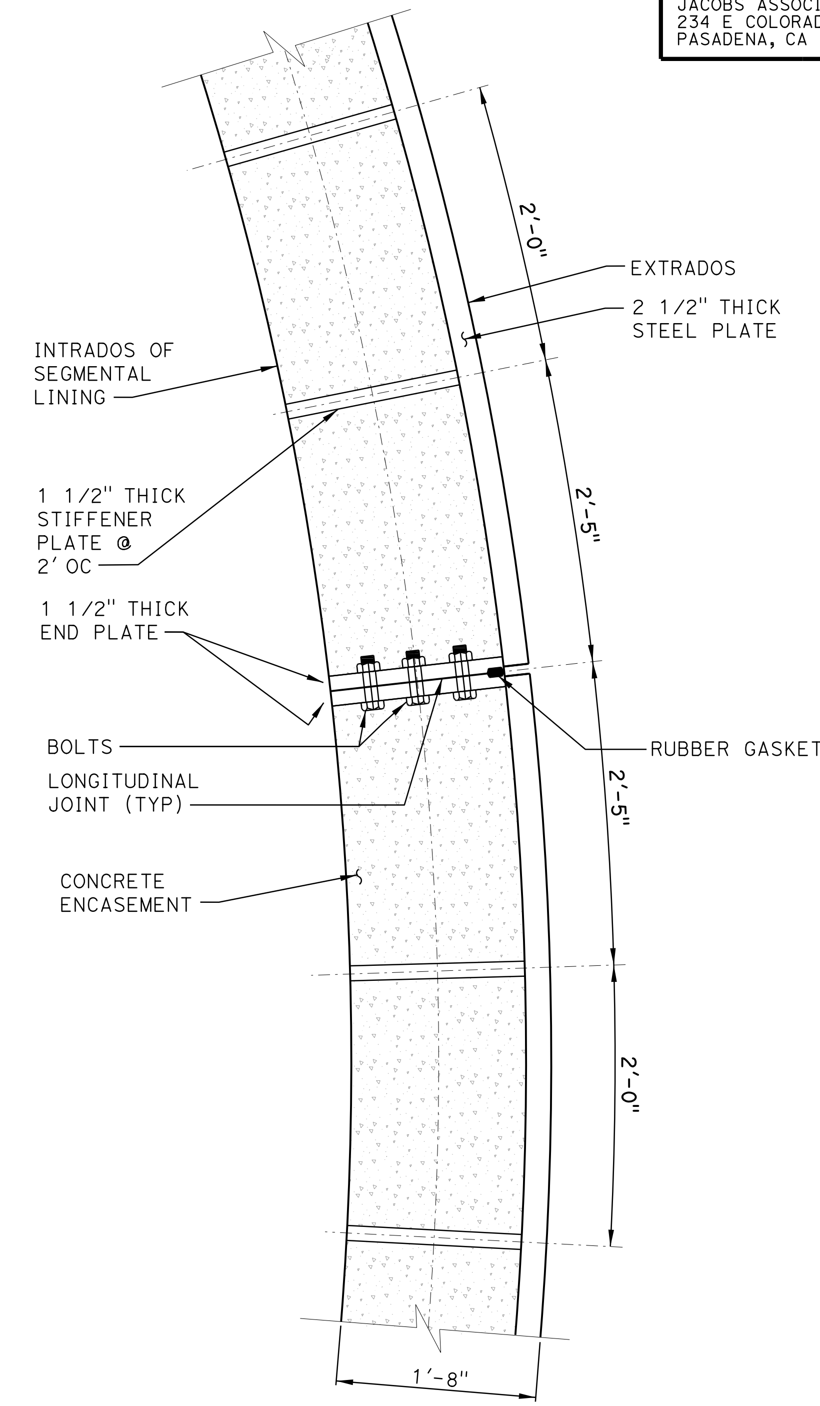
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



STEEL SEGMENTAL LINING



DETAIL 1 –

S5.02

DESIGNED BY	J. VANGREUNEN/ Y. SUN	DATE	3-03-2014
DRAWN BY	J. TOLES	DATE	3-03-2014
CHECKED BY	S. KLEIN	DATE	3-03-2014
APPROVED	S. DUBNEWYCH	DATE	3-03-2014

X
PROJECT ENGINEER

SR 710 NORTH STUDY

STEEL SEGMENTAL LINING SECTION

BRIDGE NO. TBD | UNIT:
SCALE: AS SHOWN | PROJECT NUMBER & PHASE:

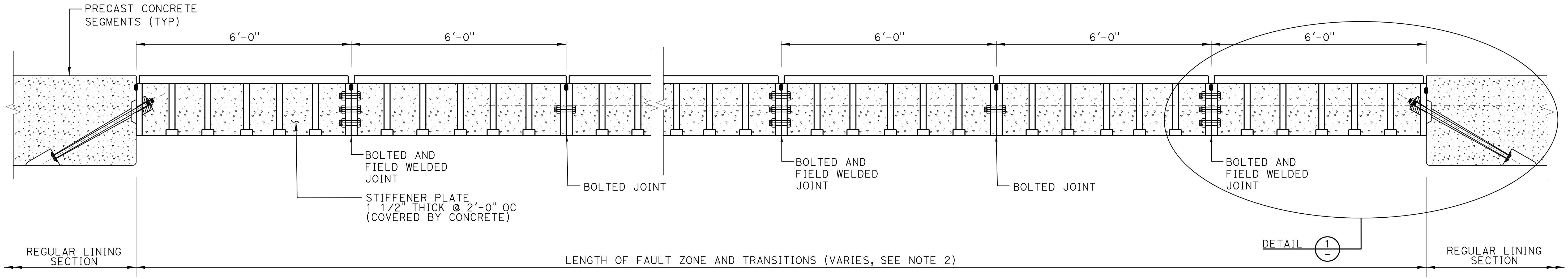
DESIGN OVERSIGHT
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

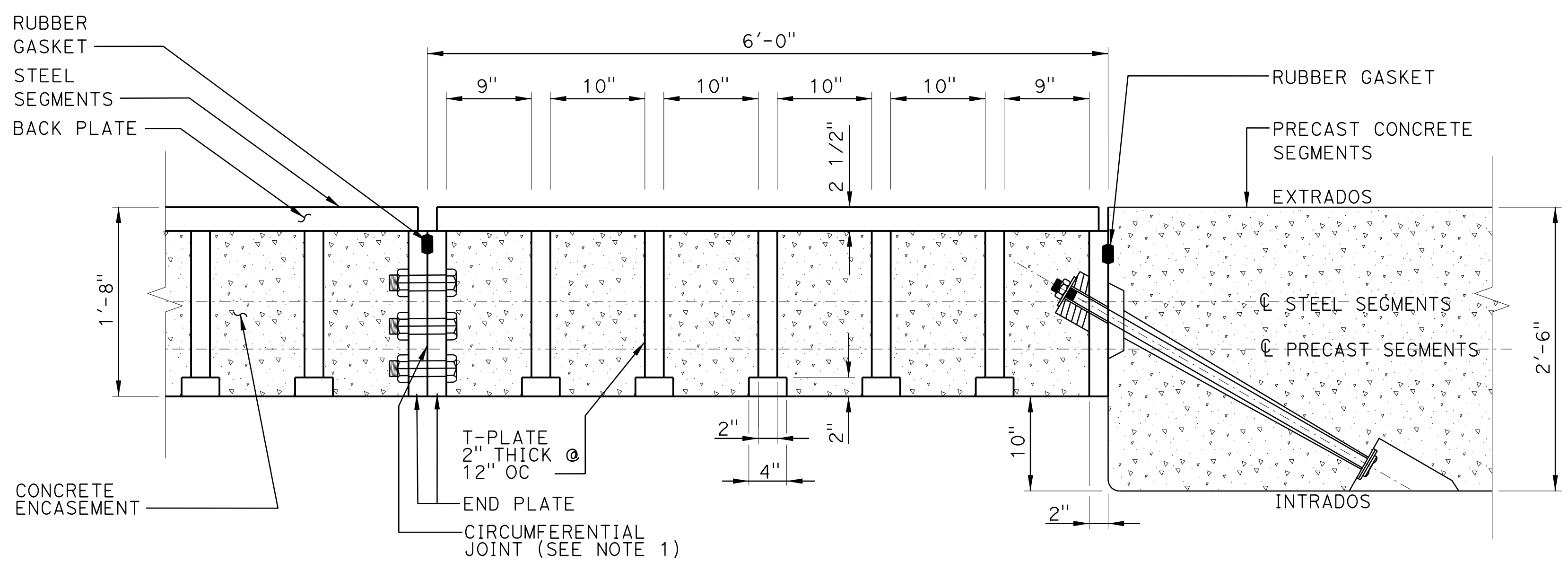
METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



LONGITUDINAL FAULT SECTION A - S5.02

NTS



DETAIL 1 -

NTS

NOTE:

1. CIRCUMFERENTIAL JOINTS OF STEEL SEGMENTS ARE WELDED ON INTRADOS AT EVERY OTHER JOINT.

2. LENGTH OF FAULT ZONE AND TRANSITIONS DETERMINED BY WIDTH OF FAULT ZONE AND OPERATIONAL REQUIREMENTS TO RE-ALIGN ROADWAY AFTER RUPTURE.

S5.03

DESIGN OVERSIGHT

SIGN OFF DATE

DESIGNED BY	J. VANGREUNEN/ Y. SUN	DATE	3-03-2014
DRAWN BY	J. TOLES	DATE	3-03-2014
CHECKED BY	S. KLEIN	DATE	3-03-2014
APPROVED	S. DUBNEWYCH	DATE	3-03-2014

X
PROJECT ENGINEER

SR 710 NORTH STUDY

LONGITUDINAL FAULT SECTION

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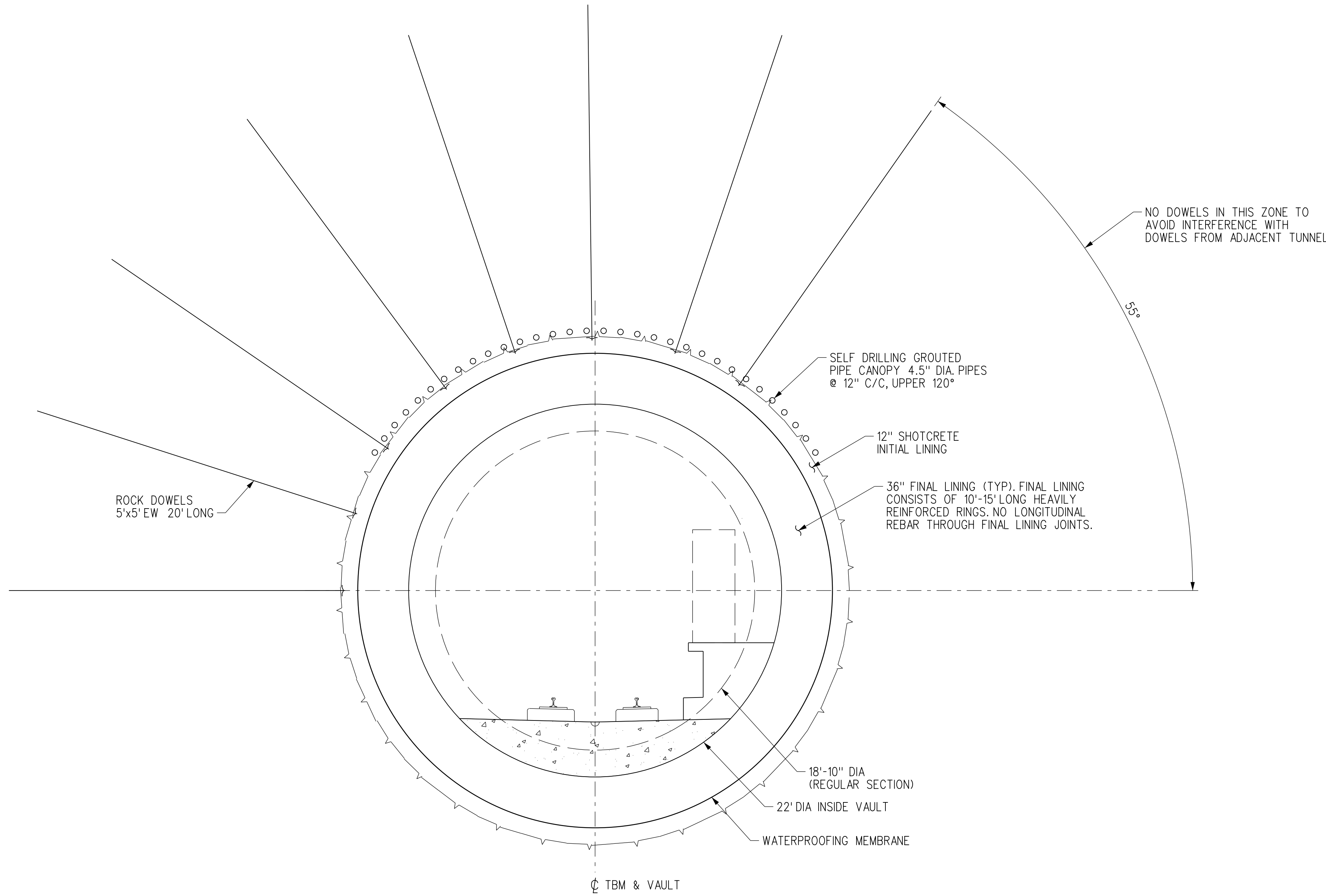

FAULT CROSSING VAULT A Y-201

NOTES:

1. SEE Y-502 AND Y-503 FOR LOCATIONS OF PIPE CANOPY AND ROCK DOWELS.
2. DETAILS OF TRACK BED AND JOINT DETAILS NOT SHOWN.

4' 2' 0 4' 8'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY J. VANGREUNEN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
FAULT CROSSING
DETAILS
SHEET 1 OF 2

CONTRACT NO
DRAWING NO Y-501
REV
SCALE 1/4" = 1'-0"
SHEET NO

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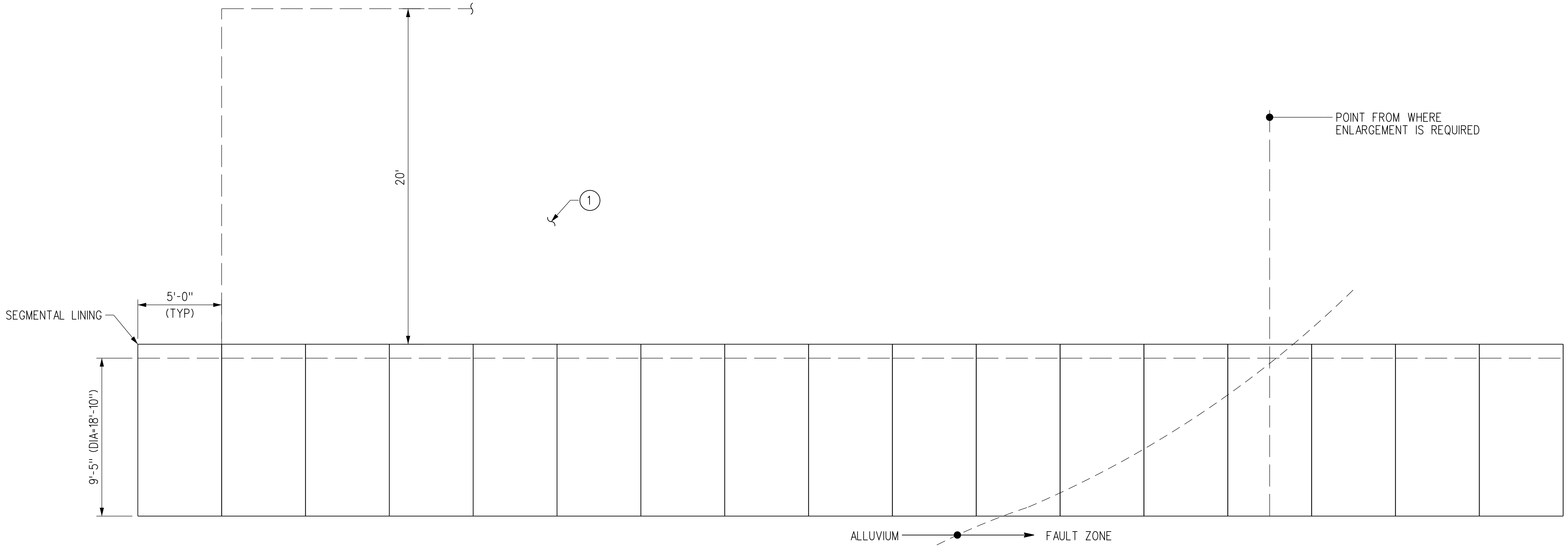
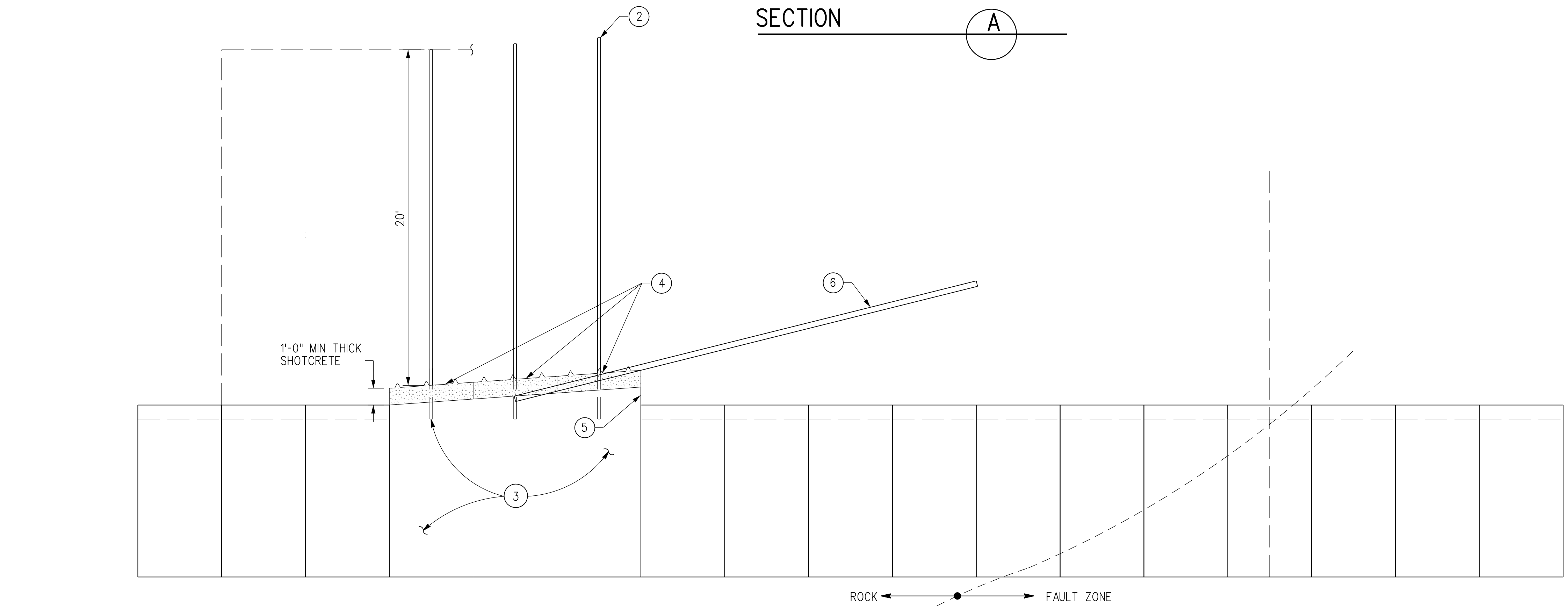


SECTION A



SECTION B

VAULT CONSTRUCTI ON SEQUENCE

1. PERFORM GROUND IMPROVEMENT IN ALLUVIUM/FAULT ZONE AS REQUIRED. GROUND IMPROVEMENT METHODS INCLUDE, BUT ARE NOT LIMITED TO, PERMEATION GROUTING OR CHEMICAL GROUTING. TREATED GROUND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 400 PSI AT 28 DAYS. TREATED GROUND SHOULD REACH SPECIFIED FULL STRENGTH PRIOR TO COMMENCING THE SUBSEQUENT STAGES.

VAULT CONSTRUCTI ON SEQUENCE

2. INSTALL ROCK DOWELS (20' LONG) THROUGH EACH SEGMENT RING.
3. REMOVE ONLY ONE SEGMENTAL LINING RING AT A TIME. EXCAVATTE GROUND TO ACHIEVE THE FINAL LINING PROFILE. CUT OFF PORTIONS OF ROCK DOWELS AS NECESSARY.
4. PLACE FIBER REINFORCED SHOTCRETE LINING OVER EACH SEGMENTAL LINING RING REMOVED USING LATTIC GIRDERS. INSTALL MINIMUM 12" SHOTCRETE BEFORE START OF NEXT RING SEGMENT. AS EACH SHOTCRETE RING IS COMPLETED TO FULL THICKNESS PLACE TEMPORARY BACKFILL TO AVOID DAMAGE TO THE INVERT.
5. SUPPORT FACE AS REQUIRED WITH SHOTCRETE.
6. INSTALL PIPE CANOPY.

4' 2' 0 4' 8'

PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

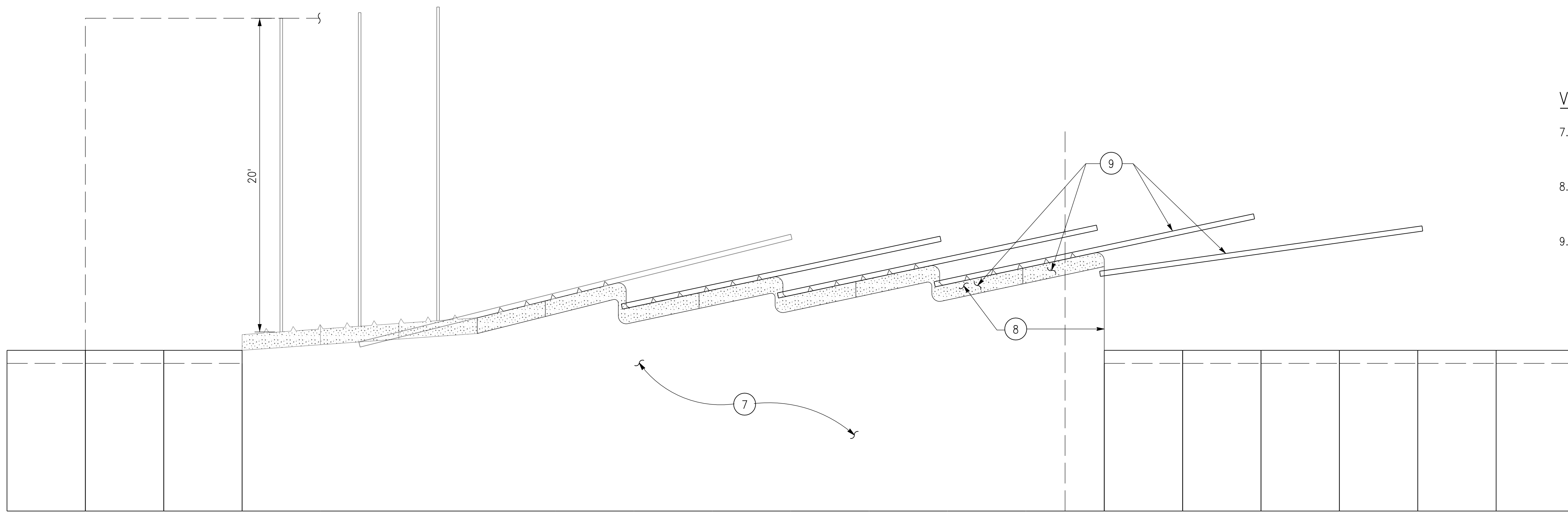
REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY VANGREUNEN/HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY
Metro

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
VAULT CONSTRUCTION
SEQUENCE
SHEET 1 OF 2

CONTRACT NO
DRAWING NO Y-502
REV
SCALE 1/4" = 1'-0"
SHEET NO



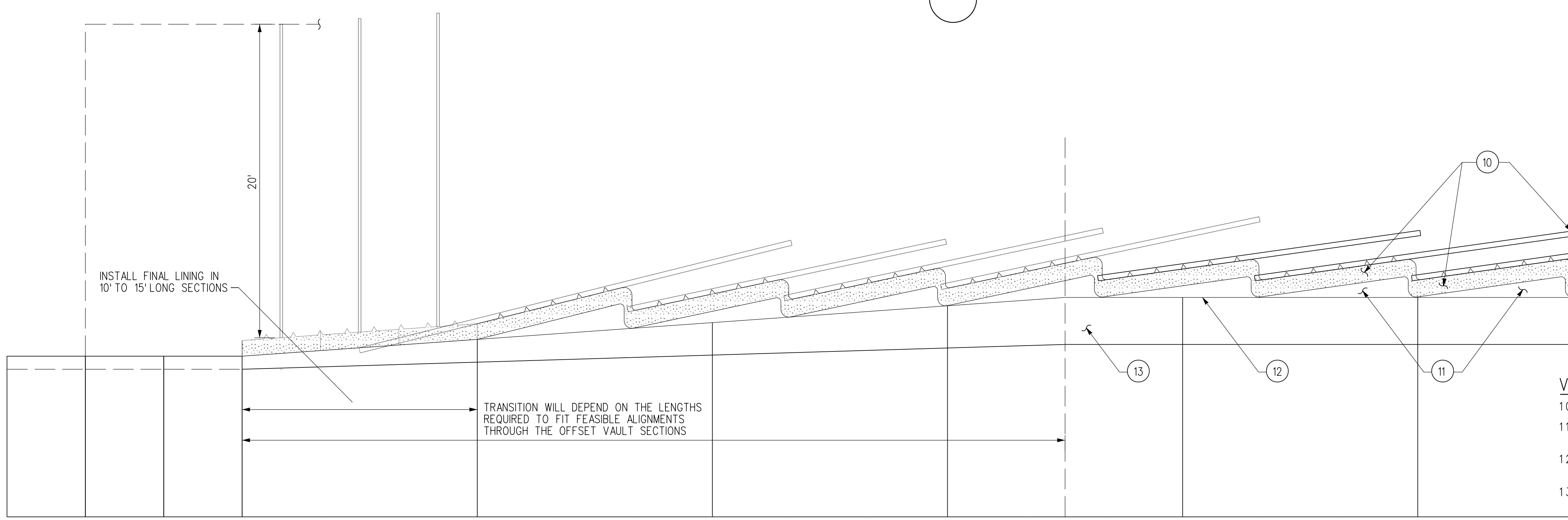
VAULT CONSTRUCTION SEQUENCE

7. REMOVE A MAXIMUM OF TWO SEGMENTAL LINING RINGS UNDER THE PIPE CANOPY INSTALLED IN STEP 6. REMOVE NO MORE THAN ONE RING AT A TIME.

8. PLACE SHOTCRETE LINING OVER EACH SEGMENTAL LINING RING REMOVED AS DESCRIBED IN STEP 4. SUPPORT FACE AS REQUIRED WITH SHOTCRETE.

9. REPEAT UNTIL THE FULLY ENLARGED SECTION IS REACHED. PIPE CANOPY ANGLE IN STEP 9 CAN BE DECREASED. IN EACH STEP, INSTALL THE PIPE CANOPY TO ACHIEVE AN OVERLAP OF AT LEAST TWO SEGMENT RINGS.

SECTION C



VAULT CONSTRUCTION SEQUENCE

10. REPEAT STEPS 7 TO 9 FOR THE ENTIRE VAULT LENGTH.

11. INFILL THE OVEREXCAVATED AREAS AND APPLY A LAYER OF SMOOTHING PLAIN SHOTCRETE.

12. AFFIX WATERPROOFING MEMBRANE TO THE SMOOTH SHOTCRETE FACE.

13. INSTALL THE CAST-IN-PLACE FINAL LINING WITH JOINTS AS SHOWN.

SECTION D

4' 2' 0 4' 8'

PRELIMINARY

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REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY VANGREUNEN/HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY S. DUBNEWYCH
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
VAULT CONSTRUCTION SEQUENCE
SHEET 2 OF 2

CONTRACT NO
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SHEET NO

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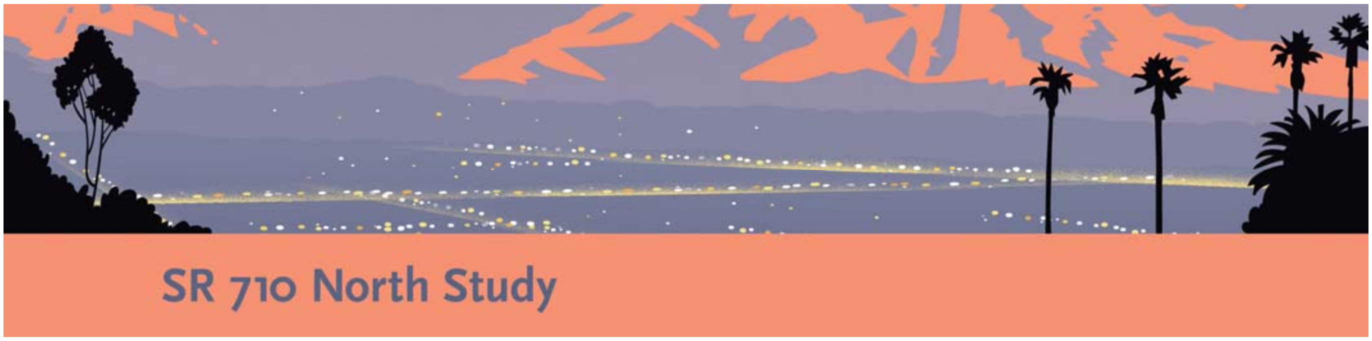
Appendix I
TM-7 Preliminary Design Concepts for the Freeway Portal Excavation Support Systems



TECHNICAL MEMORANDUM 7

Preliminary Design Concepts for the Freeway Portal Excavation Support Systems

PREPARED FOR: Metro
COPY TO: Caltrans
PREPARED BY: Jacobs Associates/CH2M HILL
DATE: August 22, 2014
PROJECT NUMBER: 428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Task Description and Scope

This technical memorandum (TM) describes the preliminary design concepts developed for excavation support systems of the construction portals for the Freeway Tunnel Alternative. Construction portals for the Freeway Tunnel Alternative would be located at each end of the bored portion of the alignment. The tunnels are expected to be mined from both the north and south portals, so that two tunnel boring machines (TBMs) simultaneously excavate each tunnel bore. Each portal would support TBM mining activities and would also serve as a laydown area for the contractor during construction activities.

Portal excavation and support system concepts apply to both the twin- and single-bore freeway alternatives. The portal excavations are expected to remain open for the duration of tunnel construction, estimated to be about 4 to 6 years, and permanent roadway features and portal structures would be constructed and the excavation backfilled by the completion of the project.

The TBMs would be launched from these construction portal excavations, which would also be used to stage construction activities for the tunneling work. In the permanent case, a cut and cover box structure would be constructed within the construction portal excavation; however these permanent works are beyond the scope of this TM and are not discussed herein. Preliminary design concepts shown on drawings for the portal excavations and support systems are included in Attachments A and B for the twin-bore and single-bore options, respectively.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Portal Geometry

The Freeway Tunnel Alternative geometry affecting the construction portals is described below and is what the preliminary design concepts in this TM are based on. As the vertical and/or horizontal alignment of the tunnel is optimized in future phases of the work, the design concepts should be re-evaluated and refined.

2.1 Twin-Bore Variation

2.1.1 North Portal The north portal excavation for the twin-bore variation of the Freeway Tunnel Alternative is shown on Drawings S4.01 and S4.02 (Attachment A) in plan and section. It is assumed that the area around the portal area would be filled and graded to approximate El. 810 ft prior to tunneling, and the portal excavation and support systems indicated on the drawings would be constructed after the filling takes place. As shown in the section (Drawing S4.02), the height of the excavation at the headwall would range from approximately 80 to 100 feet because the existing ground on the west side of the portal area is higher than that at the east side. The side walls of the portal would also be of a similar height near the head wall, and would decrease in height as the future roadway continues north and the excavation becomes shallower. As shown on Drawings S4.01 and S4.02, the excavation support is shown for the two sidewalls for a length of 500 feet from the headwall; this length of 500 feet is shown because it is long enough to launch the TBM with all of its trailing gear. The portal length excavated to launch the TBMs would ultimately be left to the contractor and would depend on several other factors including the construction sequencing with the permanent works. The width of the excavation is sufficient to launch two TBMs and also to accommodate the permanent cut and cover tunnels which would be constructed after excavation of the bored tunnels.

2.1.2 South Portal The south portal excavation for the twin-bore variation is shown on Drawings S4.03 through S4.05 (Attachment A) in plan and section. The existing ground surface north and south of Valley Boulevard has been artificially raised in the past, as seen in the drawings. This area, therefore, consists of artificial fill, and it has been assumed that the working area would be graded to approximate El. 415 ft before construction takes place; the portal excavation and support systems indicated on the drawings would be constructed after the grading takes place. The top of the head wall would be at El. 415 ft, and the bottom of the excavation would be at approximate El. 290 ft, making the excavation about 125 feet deep. The side walls of the temporary excavation would be of a similar height near the head wall, and would decrease in height as the future roadway continues south and the excavation becomes shallower. As shown on Drawings S4.03 and S4.04, the excavation support is shown for the two sidewalls for a length of 500 feet from the headwall; as discussed previously, this is sufficient length to launch the TBMs. The width of the excavation is sufficient to launch two TBMs and also to accommodate the permanent cut and cover tunnels which would be constructed after excavation of the bored tunnels.

2.2 Single-Bore Variation

The portals for the single-bore variation of the Freeway Tunnel Alternative are similar in depth and concept to those described for the twin-bore option; however the width is reduced by approximately 115 feet to accommodate the single tunnel bore. The drawings for the single-bore portals (S4.01 through S4.06) are in Attachment B.

3 Anticipated Geotechnical Conditions

Anticipated geotechnical conditions were evaluated based on geological data contained in the Preliminary Geotechnical Report provided by CH2M HILL (2014), and are detailed in the following sections. In addition to what is detailed below, it should be noted that there is a low to moderate potential of encountering naturally occurring oil and/or gas, most likely within that Puente Formation, along the Freeway Tunnel Alternative, including the portal locations. As more geotechnical information becomes available in future phases of this study, the groundwater levels and geologic contacts should be better refined, and the vertical alignment should be optimized based on the updated information.

3.1 North Portal

Figure 1 shows a generalized geologic profile at the north portal. Boring R-09-Z3B2 (CH2M HILL, 2014) was drilled in 2009 in the immediate vicinity of the north portal. Boring A-13-020 (CH2M HILL, 2014) was drilled recently within the footprint of the portal. This boring indicates that the subsurface conditions consist of dense to very dense alluvium, which is predominately granular, underlain by the Wilson Quartz Diorite basement rock. The water table is about 40 to 60 feet below the portal excavation, just above the contact between the alluvium and the rock, based on the monitoring well developed in R-09-Z3B2, which indicated a water table at 145 feet below existing grade. As mentioned previously, the area is expected to be filled to approximate El. 810 ft as shown in Figure 1 and the drawings in Attachments A and B. Based on this profile, the portal excavation would be entirely in the alluvium and fill.

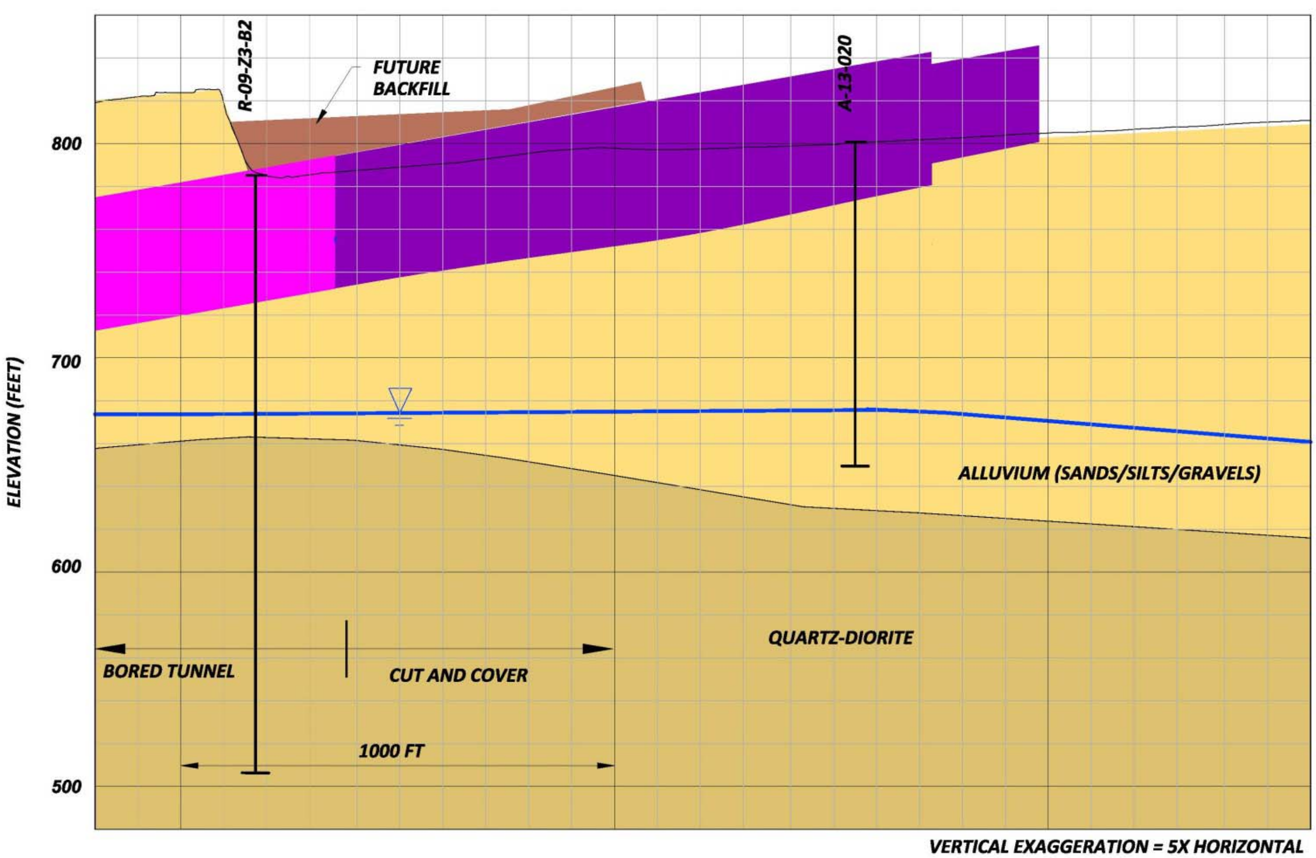


Figure 1: Generalized North Portal Geologic Profile

3.2 South Portal

Figure 2 shows a generalized geologic profile at the south portal. The geotechnical profile provided by CH2M HILL (2014) indicates that the subsurface conditions consist of 25 to 60 feet of loose to very dense alluvium underlain by the Puente Formation. The thickness of the alluvium is less at the north end of the portal and increases towards the south end. The alluvium in this area is expected to be variable and consists of sands and silty and clayey sands as wells as clays, and the water table is approximately 25 feet below the current ground surface throughout the portal excavation. Artificial fill is present above the alluvium throughout the area to be excavated, but it is expected that this area would be graded to approximate El. 415 ft before portal excavation begins and the majority of the fill would be removed.

The Puente Formation at the south portal consists of two distinct rock types based on the borings advanced in this area, as described below. While the location of the change in rock is not well-defined, Boring RC-13-005 is generally being used as a reference to demarcate the change from one rock type to the other. The actual location of the transition between the two rock types should be established more accurately with additional investigations during future design phases.

To the north of Boring RC-13-005 northward, the Puente Formation is a very weak to extremely weak claystone/mudstone with unconfined compressive strength (UCS) values in the range of 100 to 200 psi and short term unconsolidated-undrained shear strengths in the range of 75 to 120 psi. Based on the interpretation of the boring logs at this portal location, visual observation of the cores, and the laboratory testing on core samples, it appears that the Puente Formation at this location is very similar to a hard clay rather than a jointed rock, in terms of expected engineering behavior. The Boring R-09-Z1-B8, which was drilled in the vicinity of the portal, shows that the standard penetration test spoon was able to penetrate to a depth of approximately 60 feet into the Puente rock with SPT blow counts (N) of between 30 and 50 blows per foot. The rock cores do not exhibit much structure or jointing, and the UCS values are low. Based on Atterberg Limits testing, the Plasticity Index and Liquid Limit range between 11 and 22, and 20 and 28, respectively. The material is cohesive and is classified as a low plasticity clay (CL) based on the Plasticity Chart. All of the above seem to indicate that the engineering behavior of the Puente at this location would be similar to a hard clay (or a soil) and not rock. Figure 3 shows a core photograph from Boring Z1-B8, where it resembles a hard clay.

To the south of Boring RC-13-005 southward (approximately 800 feet south of the portal headwall), the Puente Formation is sandstone/siltstone with a UCS between 500 and 2200 psi. The rock is highly jointed and fractured, and these joints and fractures could transmit groundwater. A few soil-like (sand-like) zones are present in this rock-like portion of the Puente Formation. Figure 4 shows a core photograph from Boring RC-13-005 showing a laminated, well-bedded siltstone, and Figure 5 shows a photograph from Boring RC-13-004 where some joints and fractures in this sandstone are visible.

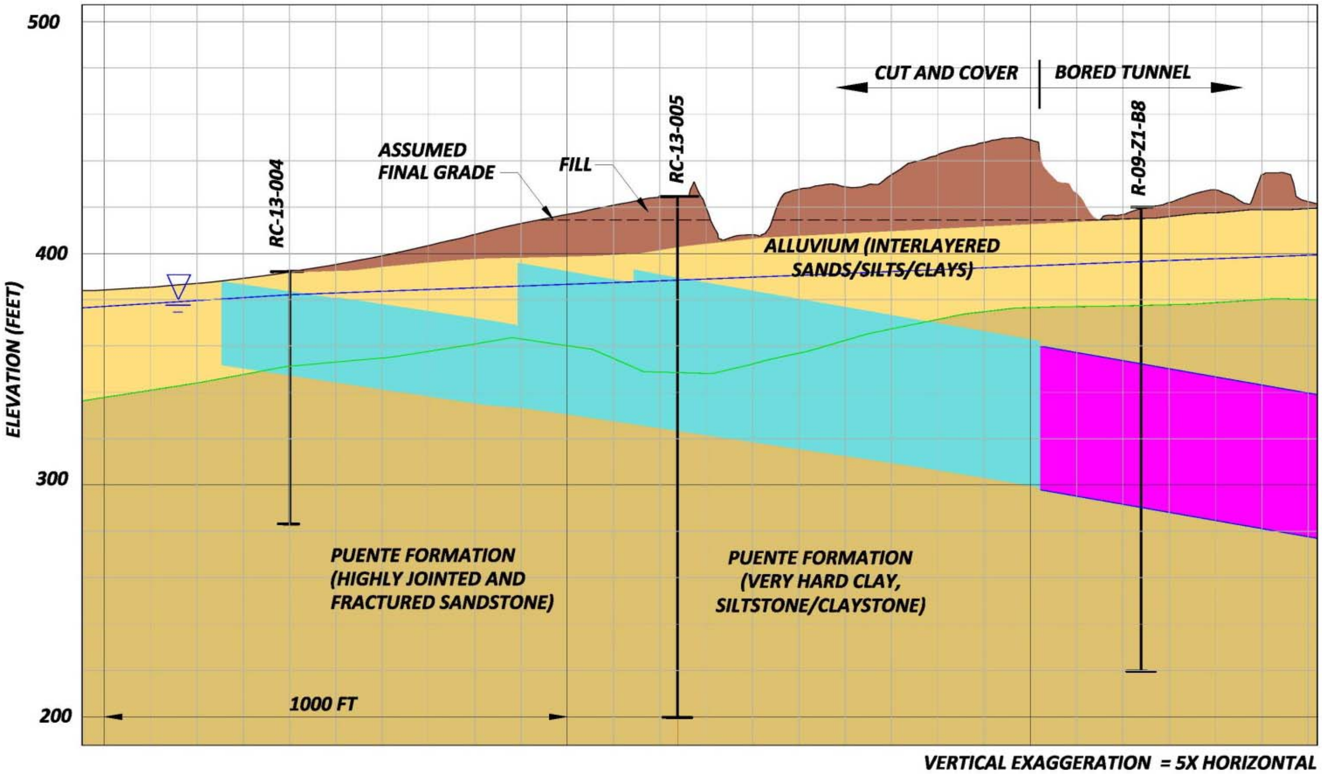


Figure 2: Generalized South Portal Geologic Profile

Figure 3: Boring Z1-B8 Core (Puente Formation) from 125 to 135 feet below Grade

Figure 4: Boring RC-13-005 Core (Puente Formation) from 93 to 98 feet below grade

Figure 5: Boring RC-13-004 Core (Puente Formation) from 54 to 59 feet below grade

4 Criteria and Assumptions

The following references were used to develop the preliminary design concepts for the excavation support systems:

- FHWA-IF-99-015 (FHWA, 1999) and the Caltrans Trenching and Shoring Manual (Caltrans, 2011b) for general design of the excavation support system and lateral pressures on walls.

- Caltrans Bridge Design Manual (Caltrans, 2011a) and AISC 13th edition (AISC, 2005) for the design of structural concrete and steel.

Soil parameters for the preliminary design concepts were developed using the boring logs and data provided by CH2M HILL (2014). Soil density was assumed based on the moisture-density testing performed in the alluvium and the SPT data, where available, were used to estimate friction angles for granular materials. The at-rest lateral stress ratio (K_0) in the alluvium was estimated from the friction angle using the Jaky relation, while K_0 values in the Puente Formation were based on the results of the pressuremeter testing.

A surcharge load was included in the preliminary analysis – the lateral load due to surcharge used was 240 psf for the upper 20 feet followed by 120 psf for the next 10 feet. This is a conservative value for the lateral load on the wall due to vertical surcharge surrounding the excavation from heavy equipment during construction.

At this time, seismic design parameters for the temporary portal excavations have not been established. Seismic design for the portal walls would be performed in subsequent design stages; however it is anticipated that a ground motion with a 100-year return period would be used for seismic design of these structures. Typically, the safety factors that are used in static design are adequate to cover the additional seismic loads for the construction level earthquake usually specified for portal walls; it is not expected that the seismic design criteria would change the currently proposed wall types.

5 Design Considerations

The design considerations are the same for both the single- and the twin-bore options. These considerations are described below for the north and south portals. Vertical excavation support walls are being considered at both locations; a sloped excavation was not considered at this time as it would require more space, decreasing the space for the contractor’s working area.

5.1 North Portal

At the north portal, the depth of the excavation ranges from approximately 80 to 100 feet, and groundwater is below the base of the excavation. Geotechnical conditions indicate dry, cohesionless soils, and therefore groundwater control measures would not be necessary for portal excavation. The main issue would be preventing the cohesionless old alluvium from sloughing into the excavation. Either a continuous wall system or a wall system such as soldier piles and lagging can be used for this purpose. In a soldier pile and lagging wall system, excessive sloughing is prevented by limiting the excavation lifts to no more than 4 to 5 feet. Due to the depth of the excavation, internal bracing or tiebacks would be required to withstand the lateral earth pressures and restrain ground movements, which could affect nearby structures or utilities. As the soils to be retained are predominantly granular and dry, no significant difference is expected between the short term and long term (4- to 6-year time frame) lateral soil pressures.

5.2 South Portal

At the south portal, the depth of the excavation is approximately 125 feet, and the groundwater is shallow, making a significant portion of the excavation below the groundwater table. The geotechnical conditions indicate that the south portal consists of the Puente Formation overlain by saturated alluvium and fill. The Puente Formation at the south portal is of two distinct types, as mentioned previously. Primary design considerations at the south portal include groundwater control and the long-term (4- to 6-year time frame) strength and behavior of the soil-like fine-grained Puente Formation, which would exhibit the engineering behavior of saturated hard/very hard clay. These materials, when saturated, exhibit a strength reduction in the long term which is accompanied by water pressure returning to hydrostatic conditions which increases wall loads. This effect is commonly known as the change from undrained to drained conditions.

Dewatering may be considered for groundwater control at the south portal. Dewatering has the advantage of significantly reducing wall loads in all the formations discussed above. The main disadvantage to dewatering at the south portal is that for an excavation of the size considered, large volumes of water (over the time that the temporary excavation would remain open) could be generated and need to be disposed of and possibly treated. Additionally, dewatering would be most efficient and economical in the alluvium/fill and in the fractured rock-like Puente Formation sandstone, whereas dewatering in the fine-grained soil-like Puente Formation may not be economical. Additionally, the environmental impacts associated with dewatering, such as settlement and disturbing/spreading any environmentally-impacted groundwater, must be considered. Feasibility of dewatering at the south portal can be further investigated during subsequent design phases. Properties exist at the south portal approximately 75 to 100 feet from the east wall; the impacts of dewatering mentioned above would require further investigation at these properties and for other adjacent structures. Such studies will be undertaken during future design phases.

As an alternate to dewatering, wall systems can be used that would essentially cut off water from entering the excavation. If a section of the portal with the rock-like fractured/jointed Puente Formation sandstone is encountered at the excavation base, groundwater control measures to cut off water inflows from the excavation base would likely be necessary if dewatering is not employed. An advantage of using a watertight excavation support system is that only minimal amounts of groundwater have to be treated and disposed. A disadvantage is that the lateral wall loading increases due to the water pressure – as with hard clays, the short-term shear strength upon excavation would be high in the fine-grained Puente Formation, but this would reduce in the long term as the pore pressures equilibrate.

5.3 Portal Headwall Support

At the headwall of either portal, the excavation support system cannot be supported with tiebacks, since a TBM has to mine through this zone and the tiebacks would interfere with the mining operations. In addition, for a slurry wall type headwall, steel reinforcing cannot be used over the area of the tunnel bores, which could have a diameter of over 60 feet. Fiber Glass (GRP) reinforcing would likely not be feasible given the lateral loading on the wall and the large span of the tunnel bore. GRP soil nails for lateral wall support would also likely not be feasible for the same reasons.

5.4 Analysis and Design Methods

Limit equilibrium methods were used to analyze the excavation support systems, and during future design phases, as more detailed geotechnical information becomes available, numerical methods may be used to analyze the systems. The program Shoring V8 (CivilTech, 2012) was used for the analysis, and lateral pressures were estimated as explained above in the criteria and assumptions.

At the north portal, where the retained soils are predominantly granular, braced pressure diagrams as appropriate for sands were used. At the south portal, varying pressure diagrams were used depending on the soil type. In the fine-grained Puente Formation, braced pressures as appropriate for hard clays were used, while in the saturated fill/alluvium, which is predominantly granular, braced pressure diagrams as appropriate for sands were used considering buoyant unit weights for the soil pressures. Water pressure was taken into account for the excavations expected in saturated materials.

The estimated lateral pressures, together with wall geometric parameters, were input into the program to analyze wall moments, shears, and tieback loads. These loads were then used to determine a preliminary concept for the size of the wall elements per the AISC and Caltrans design codes as appropriate. The unbonded and bond lengths of the tiebacks were estimated per FHWA IF-99-015 (1999).

6 Recommended Preliminary Design Details

The following sections detail the preliminary design concepts used for this phase of the study. The details presented below are expected to change in future phases of this project as more geotechnical information becomes available and/or when the alignment is optimized.

6.1 North Portal

Several wall types could be considered for the excavation support system at the north portal, including continuous wall systems, soldier piles and lagging, and soil mix walls (auger or cutter soil mix). These wall types are installed to the depths required at the north portal fairly routinely. Among these, a soldier pile and timber lagging wall supported with tiebacks appears most suitable from a constructability and structural design standpoint. For the wall depth, which is in the 80 to 100 foot range, this is also an economical wall type. Based on the preliminary analysis, the steel soldier piles are expected to be a W18 section at an 8-foot spacing, and at the range of depths required, these would most likely need to be drilled rather than be driven. Tiebacks can be installed through the soldier pile itself, which eliminates the need for walers, and the horizontal tieback spacing becomes the same as the pile spacing. The tiebacks have been preliminarily arranged to be at an angle of 15 degrees from the horizontal with the first row 7 feet from the pile top and the vertical spacing between the first and second rows at 15 feet. Thereafter, all other rows are at a 12-foot vertical spacing. Tieback loads are expected to be in the range of 190 kips to 240 kips, and bond lengths are expected to be about 40 feet in the alluvium. This concept is show on Drawings S4.01 and S4.02 in Attachments A and B.

After the installation of the soldier piles, ground improvement as described below would be performed at the headwall and then excavation of the portal would begin. Excavation would proceed in approximately 5-foot lifts, with timber lagging and tiebacks installed as indicated on the drawings as excavation proceeds. As tiebacks are installed they would be tested, stressed, and locked off. Upon reaching the total depth of the excavation, a mud slab or working slab may be installed as required.

6.2 South Portal

Given the expected wall loading at the south portal, robust walls would be required for excavation support. Secant pile and slurry walls are suitable wall types which also would render the excavation support system watertight. Of the two, slurry walls appear preferable given the depth of wall installation, which would be in the order of 130 feet or greater. At this depth range, the equipment used to install secant piles may not be able to maintain pile overlap, unless the pile diameters are substantially oversized. Therefore, slurry panel walls have been chosen as the wall type for preliminary design concept at the south portal. Given the depth of the excavation, this is a wall type that has been used routinely.

Analysis results indicate that a slurry wall at the south portal would be required to be approximately 4 feet thick, with 1% reinforcing per face. Tiebacks would be installed through these walls; the vertical and horizontal tieback spacing used would be similar to that for the soldier piles at the north portal: 8-foot horizontal spacing and a typical 12-foot vertical spacing. The preliminary design concept shows the tiebacks arranged to be at a 15 degree angle from the horizontal in the zone with the alluvium and the fine-grained Puente Formation. In the fine grained Puente Formation, tieback loads are expected to be in the range of 350 kips to 500 kips, when long term loading is considered on these walls. Bond lengths on the order of 60 feet may be required to achieve these capacities. The more heavily loaded tieback anchors may be of the single-borehole multi -anchor variety, in which the bond length of the anchor bar/strands are developed in a staggered fashion within the same drill hole to maximize capacity with a minimum drill length.

At locations with the saturated fill and alluvium overlaying the rock-like Puente Formation, analysis results indicate that the tieback loads are expected to be in the range of 180 kips to 315 kips. In this zone, the tiebacks have been arranged to be at a 45 degree angle, so that the tiebacks reach the rock with a shorter drill length. Bond length in rock is expected to be on the order of 30 feet. As the bond zone in the sandstone is shorter, this arrangement reduces total tieback length.

These two different tieback scenarios at the south portal are indicated on Drawings S4.04 and S4.05. Further geotechnical investigation is required in order to determine where the transition between the two rock types and support scenarios occur; however, the concepts for both scenarios have been conceptually developed.

After the installation of the slurry panels, ground improvement as described below would be performed at the headwall, and then excavation of the portal would begin. Excavation would be performed in lifts: excavation to approximately 2 to 3 feet below a planned tieback level would be completed, and then the tiebacks would be installed, tested, stressed, and locked off.

The rock-like fractured and jointed portion of the Puente Formation, if encountered at the base, would require base grouting (permeation or pressure grouting) in order to seal off the joints and fractures from groundwater inflows. The base grouting would be performed in advance of the excavation. Grout types for base grouting could be microfine cement grout, a chemical grout of the polyurethane, colloidal silica, acrylate type, or a combination of these grout types. Note that since the formation grouted is a hard (though fractured and jointed), rock-like sandstone, uplift pressures on the grouted rock zone are likely not a significant concern. If they become a design concern, then the walls can be extended into a zone of the rock, below which it is essentially watertight.

6.3 Portal Headwall Support

As explained in the design considerations, no structural steel elements can be present within the zone through which the TBM would excavate. Additionally, GRP reinforcing for walls and GRP soil nails do not appear feasible at this time. To meet these requirements, a gravity wall concept could be used for the portal headwall in the zones over which the TBM would excavate. To achieve this, a substantial mass of soil at the portal headwall would be improved to the point that it becomes self-supporting. Several methods were considered to achieve this improvement, including jet grouting, deep soil mixing, and ground replacement using slurry wall or drilled shaft equipment. The method selected should be feasible in the ground types that exist at the north and south portals and should also provide the required strength after improvement.

Jet grouting and deep soil mixing are both feasible at the north portal based on depth and anticipated geologic conditions. Jet grouting essentially erodes the soil and mixes the cuttings in-situ with cement slurry to form stabilized columns. These columns overlap or interlock and can be designed to act as a gravity wall. Soil mixing uses continuous auger flights to also mix the soil with a cement slurry to form interlocking columns. At the south portal adjacent to the headwall, the ground consists of the fill/alluvium overlaying the hard, clay-like Puente Formation, and jet grouting is not feasible in the clay-like Puente Formation. Soil mixing or ground replacement using slurry wall and/or drilled shaft equipment appears to be the most suitable method at the south portal to achieve the ground improvement.

At this time, the preliminary design concept for ground improvement consists of large-diameter ground improvement columns that are interlocked with panels behind the headwalls at both the north and south portals. A total zone of 60 feet behind the headwall is shown to be improved. The improvement zone is the full height of the wall and covers the zone through which the TBM would excavate. Drawings S4.06 in Attachments A and B show this type of ground improvement. The required strength (unconfined compressive type strength) of the mass should be in the range of 500 to 1000 psi. Additional analyses will be required during subsequent design stages in order to refine the design strength. When detailed geotechnical information becomes available, the methods listed here can be further investigated in terms of constructability and required design strengths.

At the North portal, as shown on the drawings, the ground has to be filled prior to tunneling. Pending detailed environmental investigations to determine any environmental impacted soil or groundwater, the excavation material may be used to fill that area. The fill that is used would be the granular type materials that are obtained from the north portal excavation; cohesive portions of the excavated soils would not be suitable for re-use. During future design phases, feasibility of the re-use of material obtained from the north portal as fill will be investigated in greater detail using data from additional geotechnical investigations.

6.4 TBM Thrust Frame

A TBM thrust frame would be necessary at both portals, and it typically would be supported by a reinforced concrete base slab at the bottom of the portal excavation. This slab would be of substantial thickness in order to provide the required horizontal and vertical reaction to the TBM frame. This slab can be a temporary slab that is subsequently demolished or the permanent base slab needed for the permanent structure with some modifications, based on the requirements for final use. Preliminarily it is assumed that the base slab would be in the range of 8 to 10 feet thick.

6.5 Environmental Considerations for Wall Construction and Ground Improvement

6.5.1 Wall Construction Spoils and Surface Runoff

Slurry wall and soldier pile construction, as well as ground improvement methods would generate spoils. The amount of spoils generated is variable; some methods such as jet grouting and slurry walls tend to produce more spoils than others such as soil mixing and soldier pile construction. Spoils produced from wall construction would require appropriate environmental testing prior to disposal. Testing procedures, requirements and specifications that will be followed by the contractor will be established during future design phases. Potential disposal sites for this material have been identified in the Environmental Document.

Maintaining the surface water quality and drainage via a temporary erosion and sediment control system will also be an important environmental consideration. This will typically be achieved by the contractor via an erosion and sediment control system for the duration of construction. This system will be required to comply with all requirements of the California Storm Water Pollution Prevention Plan and the California Water Pollution Prevention Plan.

6.5.2 Dust, Noise and Vibration Abatement

Construction equipment such as drill rigs, cranes, and batch plants may be required for slurry wall construction and jet grouting (alternatively, existing batch plants in the metropolitan area could be used for this work). This equipment and activities may generate dust, noise and vibrations. To control dust, the contractor will be required to have in place a dust control plan that meets or exceeds South Coast Air Quality Management District (SCAQMD) requirements. The contractor will also be required to monitor noise and vibrations and limit these to below City Ordinance Limits. Sound walls may be used to control noise. More information on this is provided in the Environmental Document.

6.6 Wall Monitoring and Instrumentation

Wall monitoring and instrumentation would be required for the portal walls. The instrumentation is expected to include primarily inclinometers to measure wall movement, surface settlement points and structure settlement points (as applicable) to monitor settlement as well as piezometers to monitor groundwater levels during excavation and for the duration of construction. At the south portal, since long term wall load is a concern, the instrumentation may additionally include strain gauges and load cells on a percentage of the anchors, bracing and wall reinforcing elements to monitor changes in wall loads with time. The exact type and number of instruments will be determined in future design stages.

7 Recommendations for Future Design Evaluations

The preliminary design concepts presented herein should be advanced in future stages of the design. Additional field and laboratory investigations will be required to perform more advanced design of the excavation support systems. Recommendations for future design evaluations include, but are not limited to, the following:

- Additional geotechnical investigations to better define the thickness and extents of the various formations and the groundwater table and to identify the presence of naturally-occurring gas.

- Permeability testing for determining dewatering feasibility and base grouting design in the rock-like Puente Formation at the south portal.
- Environmental testing to determine requirements for groundwater disposal or treatment from potential dewatering.
- Pressuremeter testing to determine soil modulus and K_0 at both the north and south portals. At the south portal, in the fine-grained Puente Formation, the pressuremeter testing should include creep measurements to estimate long-term modulus and strength parameters. These tests can also be used to estimate bond stress values for tiebacks.
- Laboratory testing of samples obtained from the fine-grained Puente Formation to determine the shear strength, modulus, and creep under short- and long-term conditions. These tests can also be used to estimate bond stress values for tiebacks.
- Routine testing such as moisture density and grain size tests to help refine the evaluation of the feasibility of various ground improvement methods at the portal headwall.
- Establish seismic design parameters for the excavation support walls during construction of the tunnels.
- Determine construction staging/sequencing to better understand interface with permanent works, such as the cut-and-cover tunnels.
- In the design and construction planning for the portal excavations, requirements for controlling ground movements and preventing damage to structures should be evaluated in conjunction with selecting a method of excavation support (including its stiffness), the excavation sequencing, and monitoring requirements.

8 References

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CivilTech. 2012. Shoring Version 8. A Program for Limit Equilibrium Analysis of Retaining Walls.

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9 Revision Log

Revision 0	September 30, 2013	Internal Review
Revision 1	October 11, 2013	Metro/Caltrans Review
Revision 2	May 27, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

Twin-Bore Drawings

Metro

Caltrans

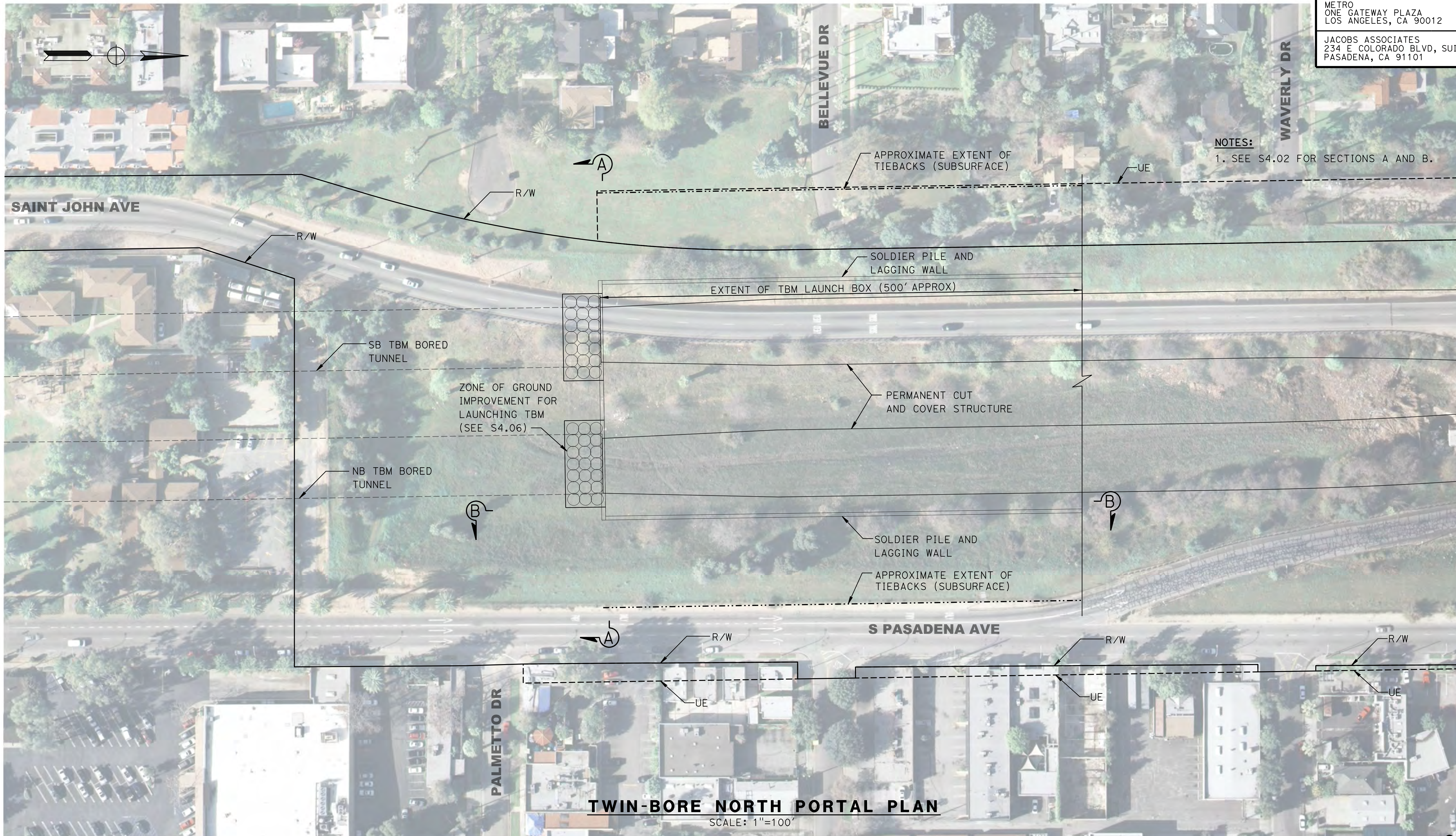
PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

NOTES:
1. SEE S4.02 FOR SECTIONS A AND B.



TWIN-BORE NORTH PORTAL PLAN
SCALE: 1"=100'

S4.01

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY
NORTH PORTAL PLAN

BRIDGE NO. TBD
UNIT:
SCALE: 1"=100'
PROJECT NUMBER & PHASE:
CONTRACT NO.: X

DESIGN OVERSIGHT
SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S401.dgn

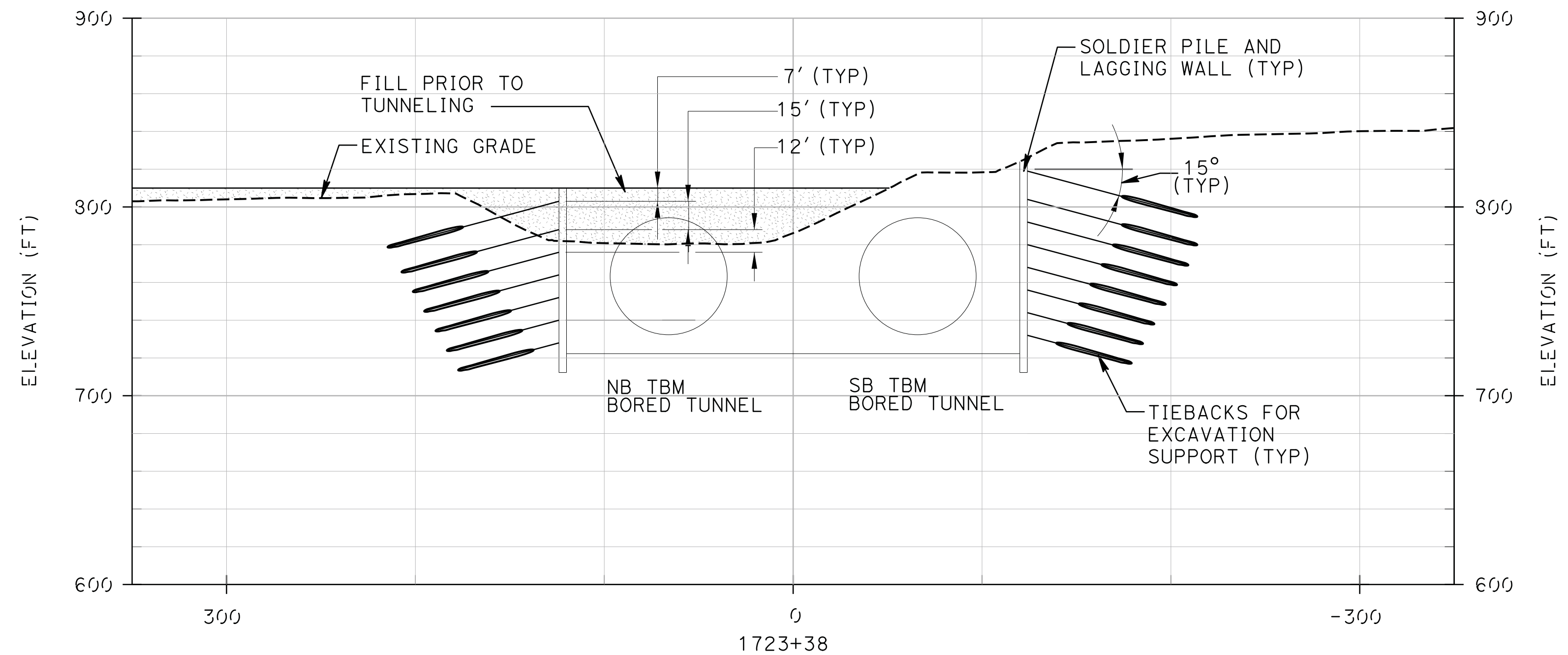
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 10:40

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



SECTION - F-7 NORTH TBM PORTAL

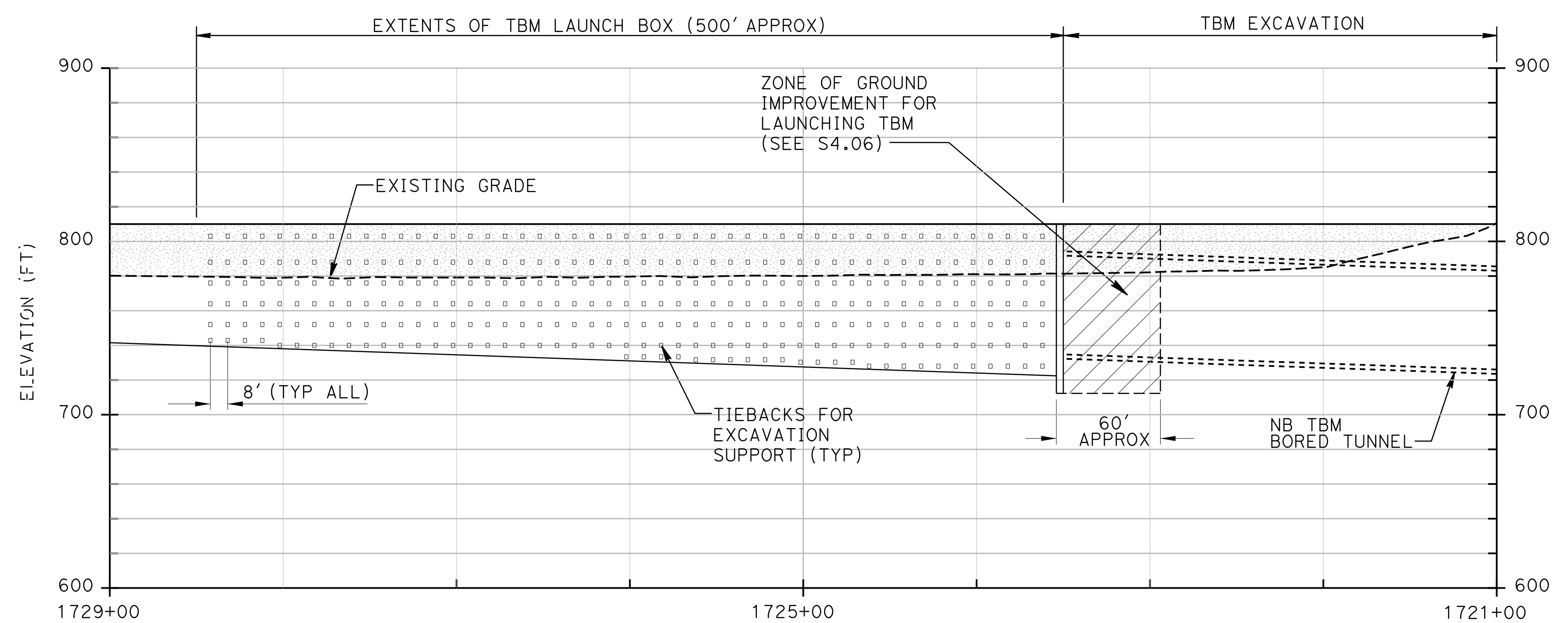
A

S4.01

SCALE: 1"=100'

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 40 FEET (APPROX).



SECTION - F-7 NORTH TBM PORTAL EAST WALL

B

S4.01

SCALE: 1"=100'

S4.02

DESIGNED BY	M. HARIHARAN	DATE 9-20-13
DRAWN BY	J. TOLES	DATE 9-20-13
CHECKED BY	M. LAWRENCE	DATE 9-20-13
APPROVED	S. DUBNEWYCH	DATE 9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

NORTH PORTAL SECTIONS

BRIDGE NO. TBD	UNIT:
SCALE: 1"=100'	PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

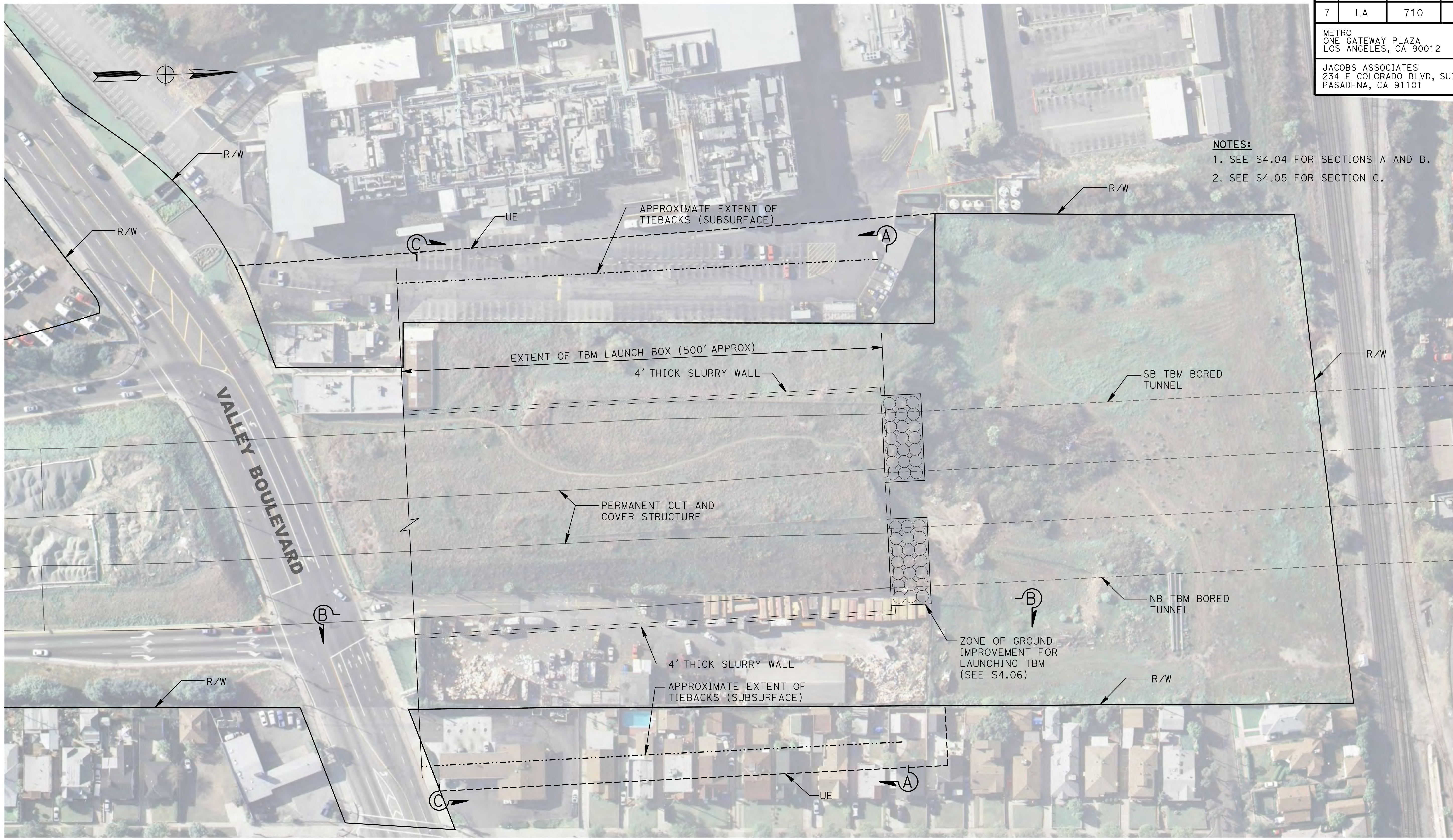
SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



NOTES:
1. SEE S4.04 FOR SECTIONS A AND B.
2. SEE S4.05 FOR SECTION C.

TWIN-BORE SOUTH PORTAL PLAN

SCALE: 1"=100'

S4.03

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY
SOUTH PORTAL PLAN

BRIDGE NO. TBD

UNIT:

SCALE: 1"=100'

PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S403.dgn

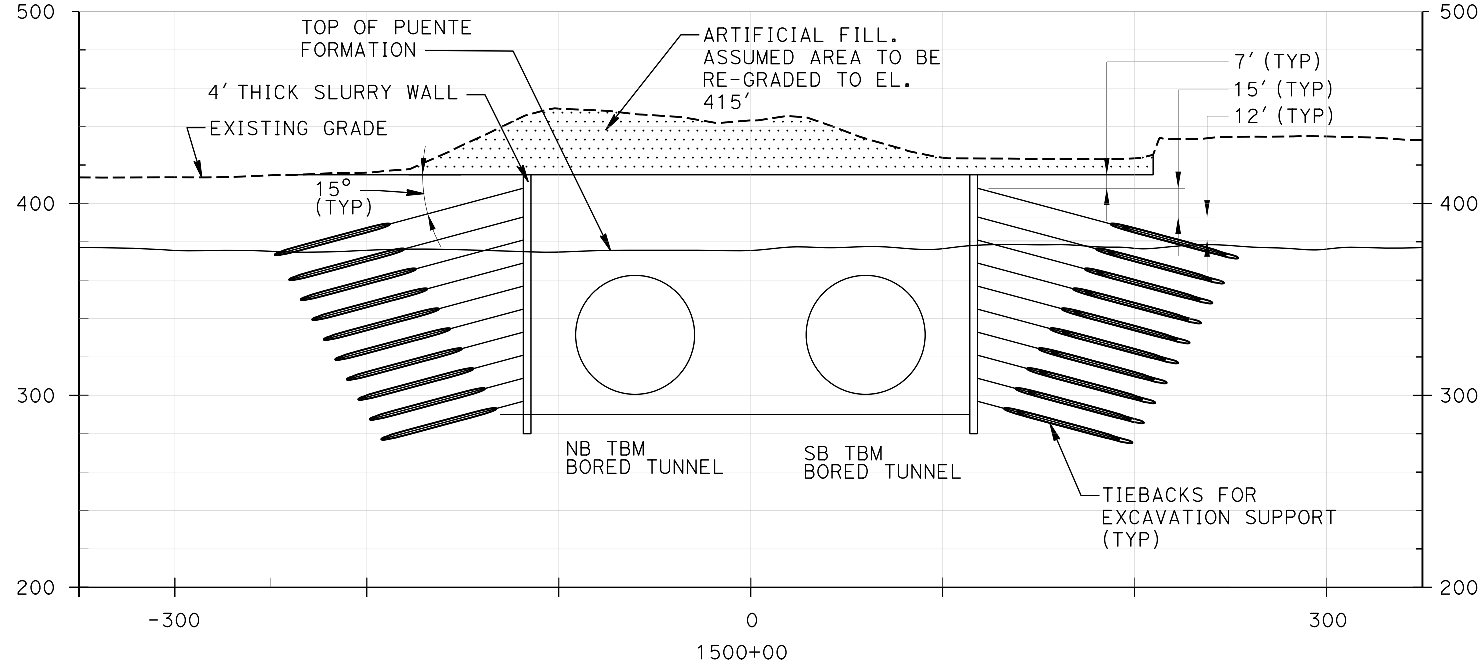
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 11:14

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

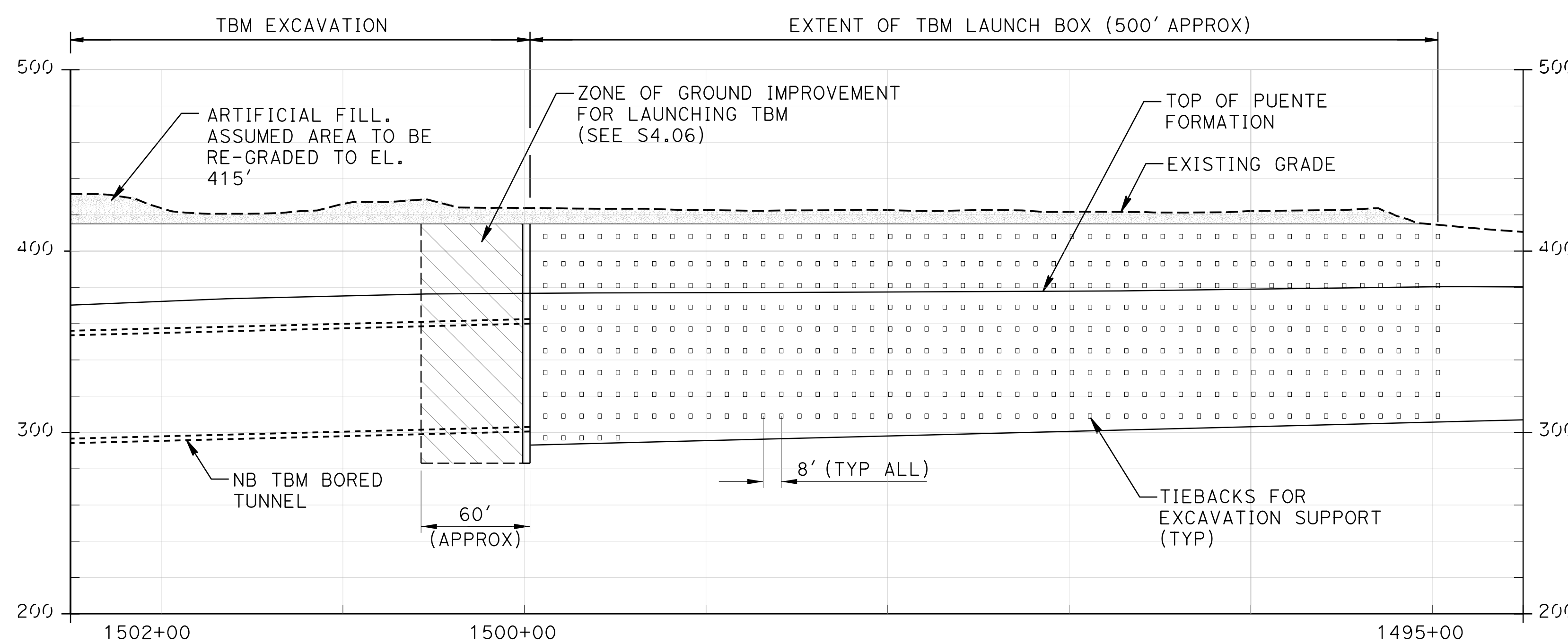
METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



SECTION - F-7 SOUTH TBM PORTAL (A) S4.03

SCALE: 1"=100'



SECTION - F-7 SOUTH TBM PORTAL EAST WALL (B) S4.03

SCALE: 1"=100'

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 60 FEET (APPROX).

S4.04

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

SOUTH PORTAL SECTIONS

BRIDGE NO. TBD | UNIT:

SCALE: 1"=100' | PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S404.dgn

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 13:19

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

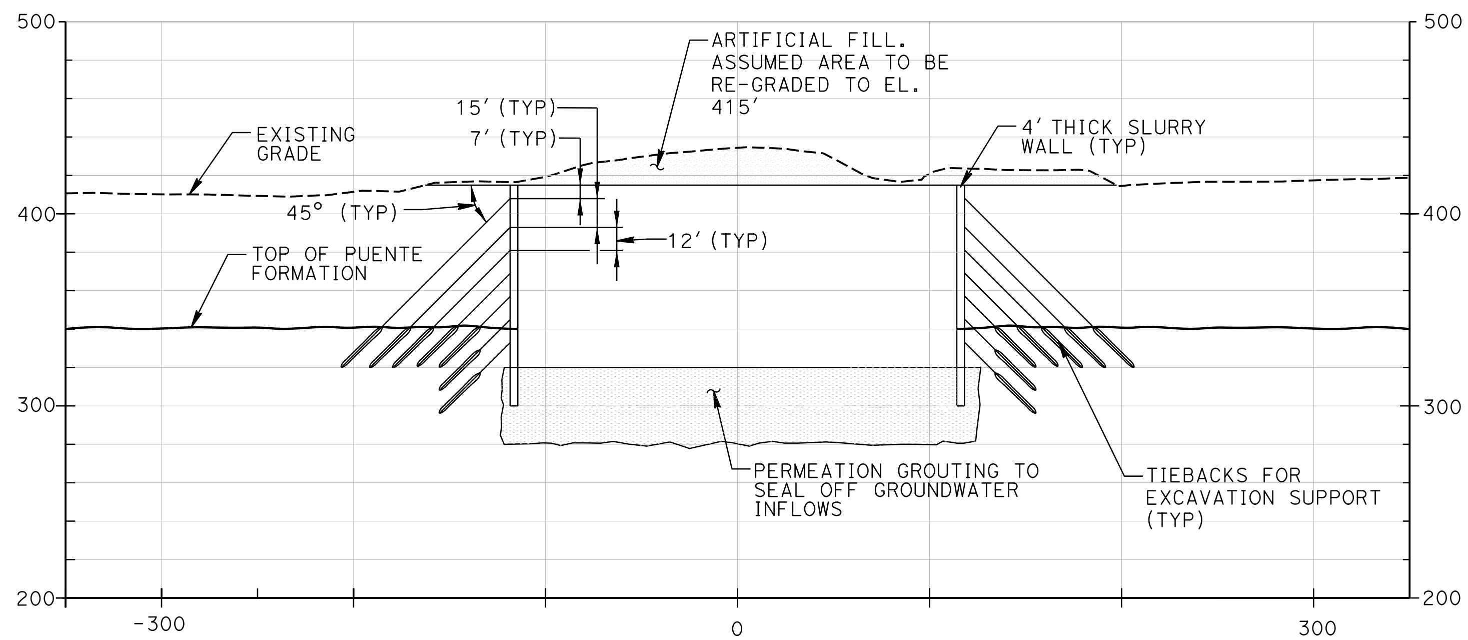
METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 30 FEET (APPROX).



SECTION - F-7 SOUTH TBM PORTAL

SCALE: 1"=100'

C
S4.03

S4.05

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

SOUTH PORTAL SECTION

BRIDGE NO. TBD
UNIT:
SCALE: 1"=100'
PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S405.dgn

CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 11:09

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

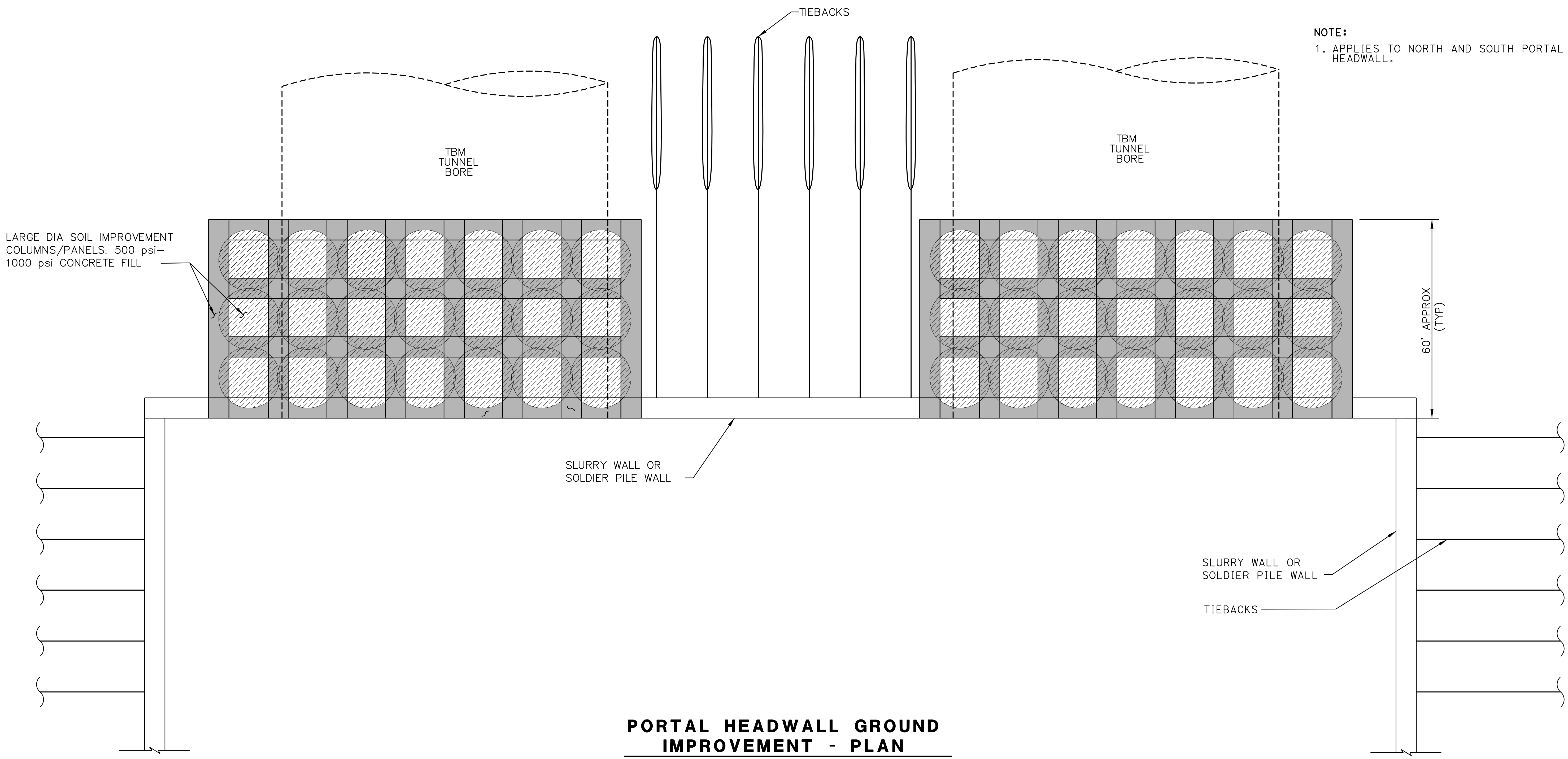
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

NOTE:

1. APPLIES TO NORTH AND SOUTH PORTAL HEADWALL.



PORTAL HEADWALL GROUND IMPROVEMENT - PLAN

SCALE: 1"=20'

S4.06

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

PORTAL HEADWALL

BRIDGE NO. TBD
UNIT:
SCALE: AS SHOWN
PROJECT NUMBER & PHASE:
CONTRACT NO.: X

DESIGN OVERSIGHT
SIGN OFF DATE

Attachment B

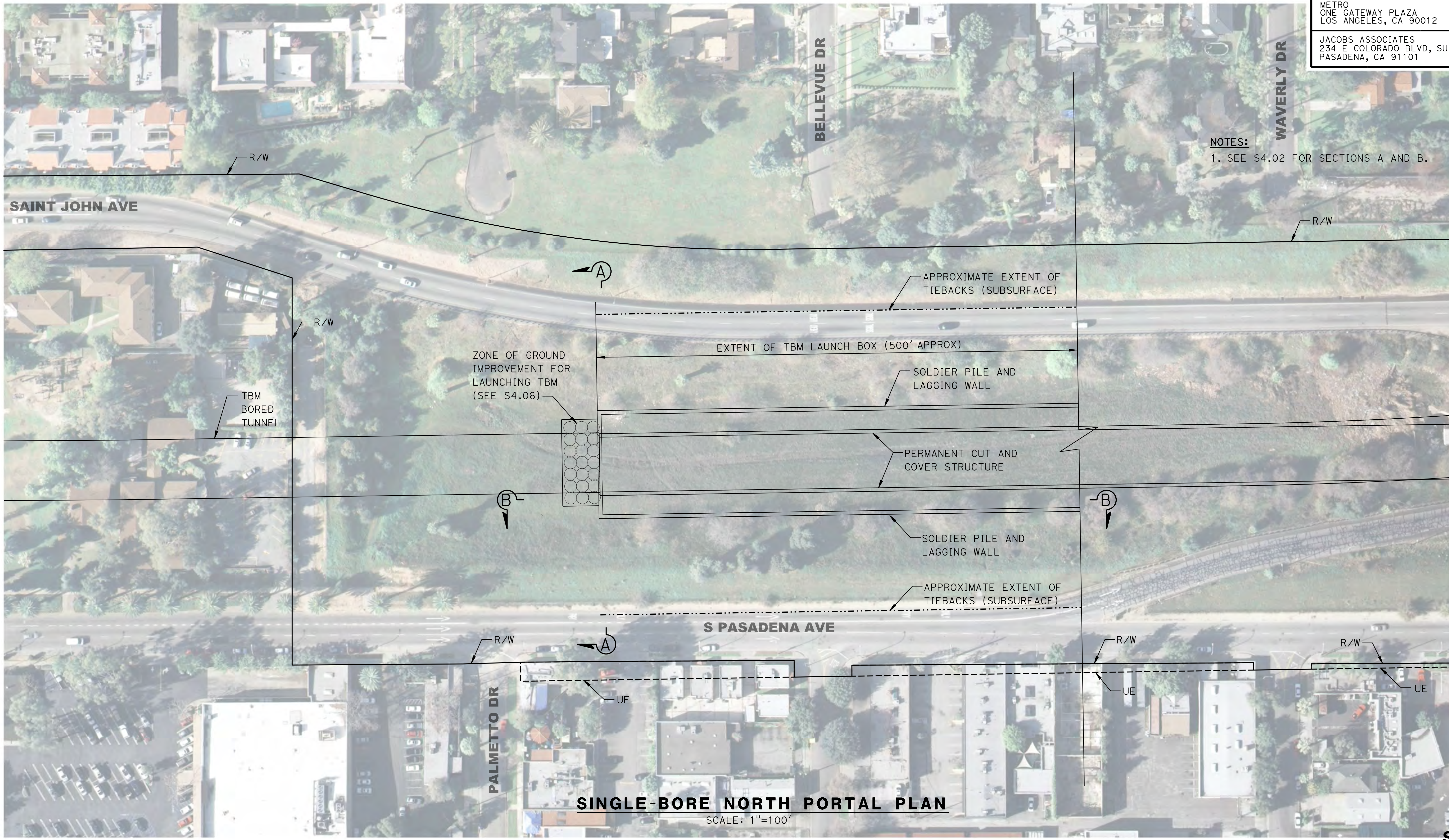
Single-Bore Drawings

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



NOTES:

1. SEE S4.02 FOR SECTIONS A AND B.

SINGLE-BORE NORTH PORTAL PLAN

SCALE: 1"=100'

S4.01

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY
NORTH PORTAL PLAN

BRIDGE NO. TBD	UNIT:
SCALE: 1"=100'	PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

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CONTRACT NO.: X

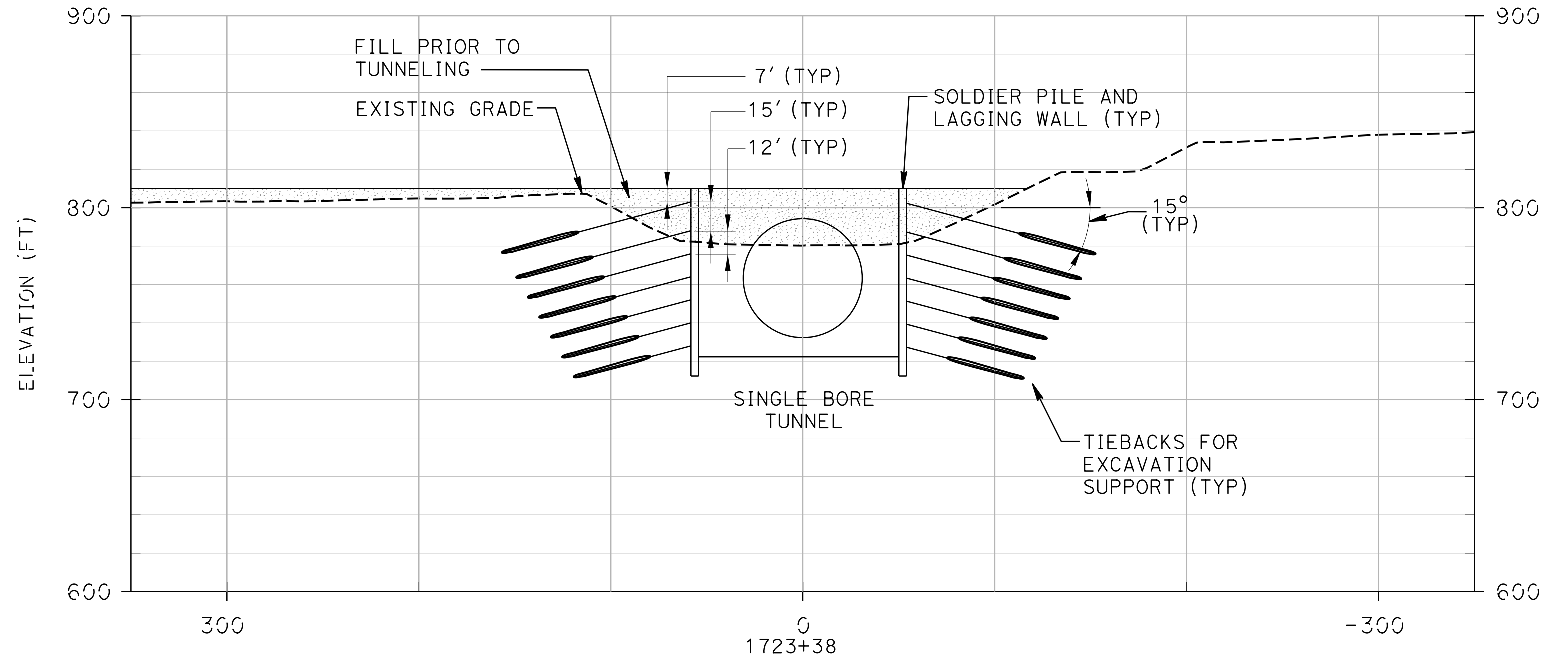
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 12:53

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

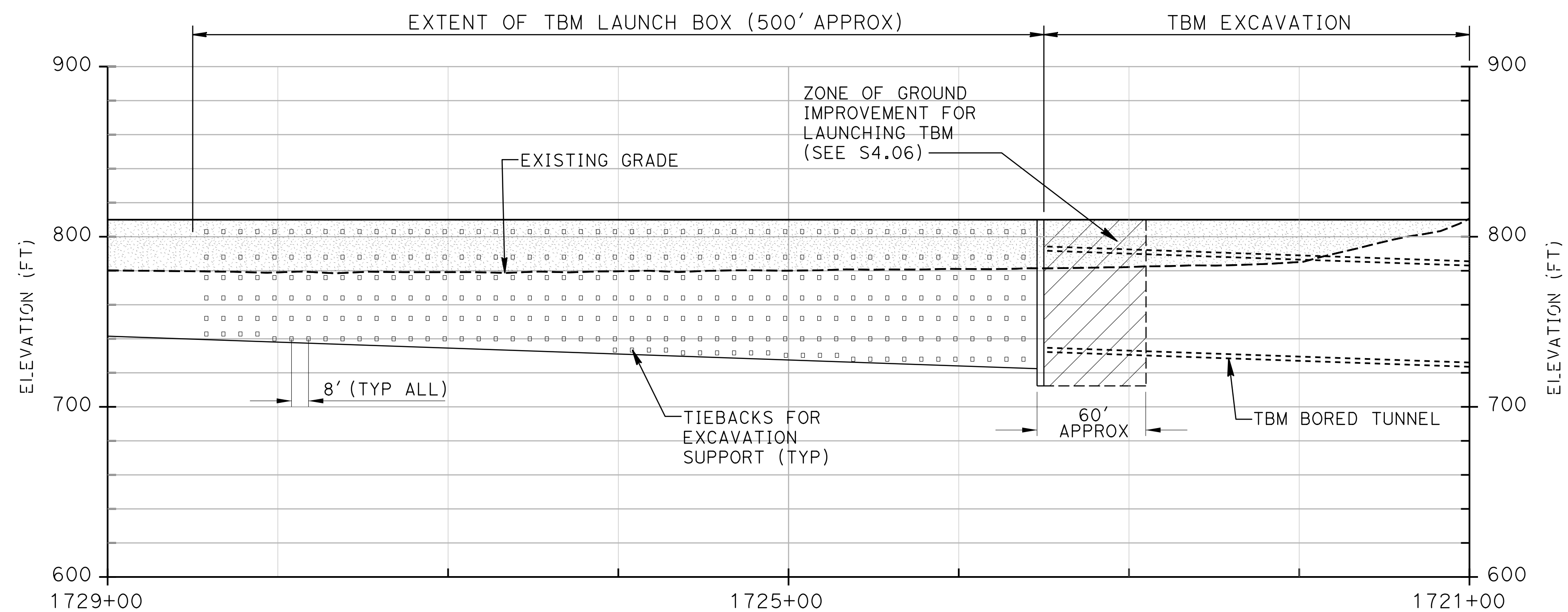


SECTION - SINGLE BORE
NORTH TBM PORTAL (A) S4.01

SCALE: 1"=100'

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 40 FEET (APPROX).



SECTION - SINGLE BORE
NORTH TBM PORTAL (EAST WALL) (B) S4.01

SCALE: 1"=100'

S4.02

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

NORTH PORTAL SECTIONS

BRIDGE NO. TBD | UNIT:

SCALE: 1"=100' | PROJECT NUMBER & PHASE:

CONTRACT NO.: X

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S402.dgn

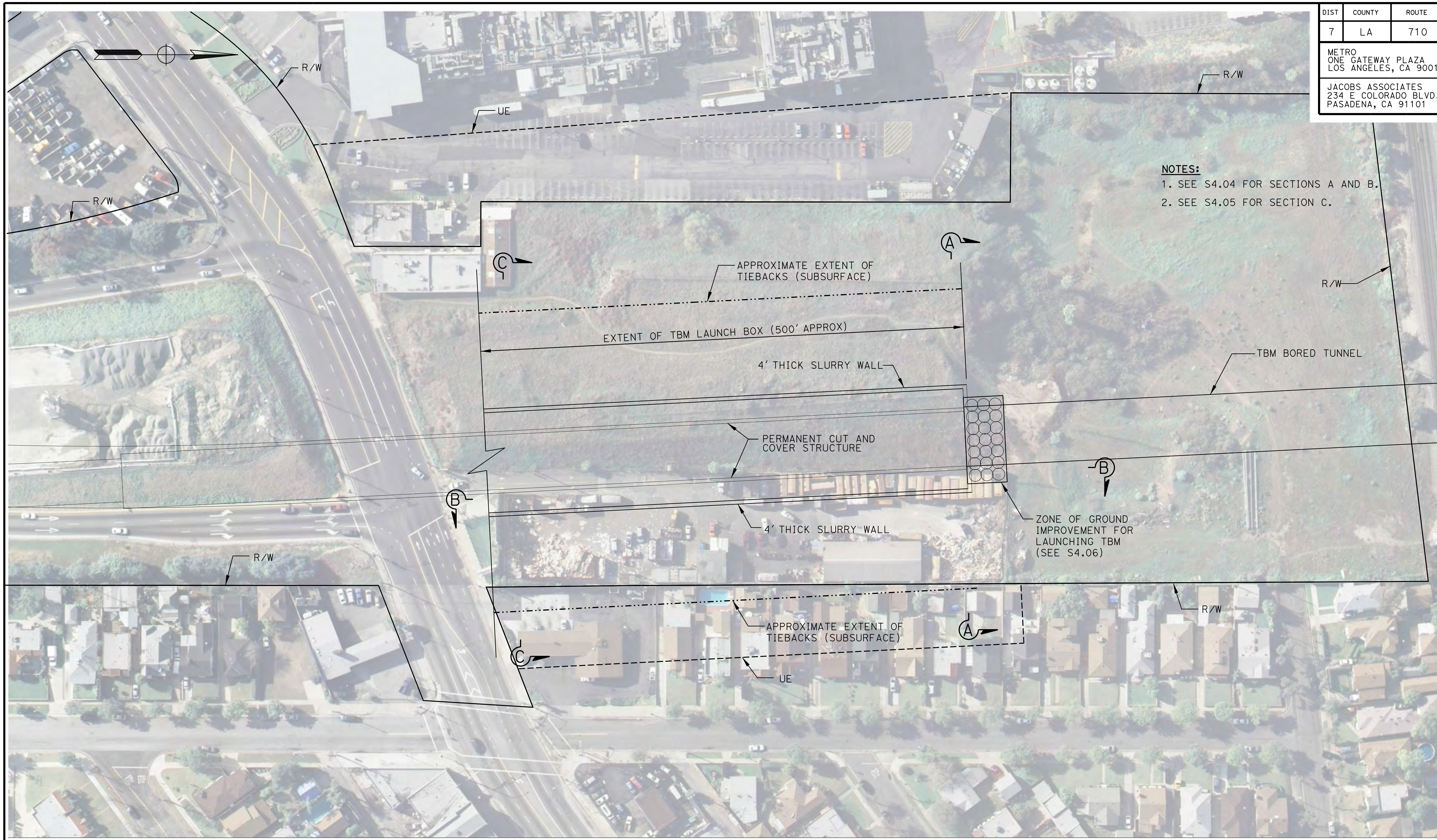
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 11:20

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



SINGLE-BORE SOUTH PORTAL PLAN

SCALE: 1"=100'

S4.03

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	JAMES TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

SOUTH PORTAL PLAN

BRIDGE NO. TBD	UNIT:
SCALE: 1"=100'	PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S403.dgn

CONTRACT NO.: X

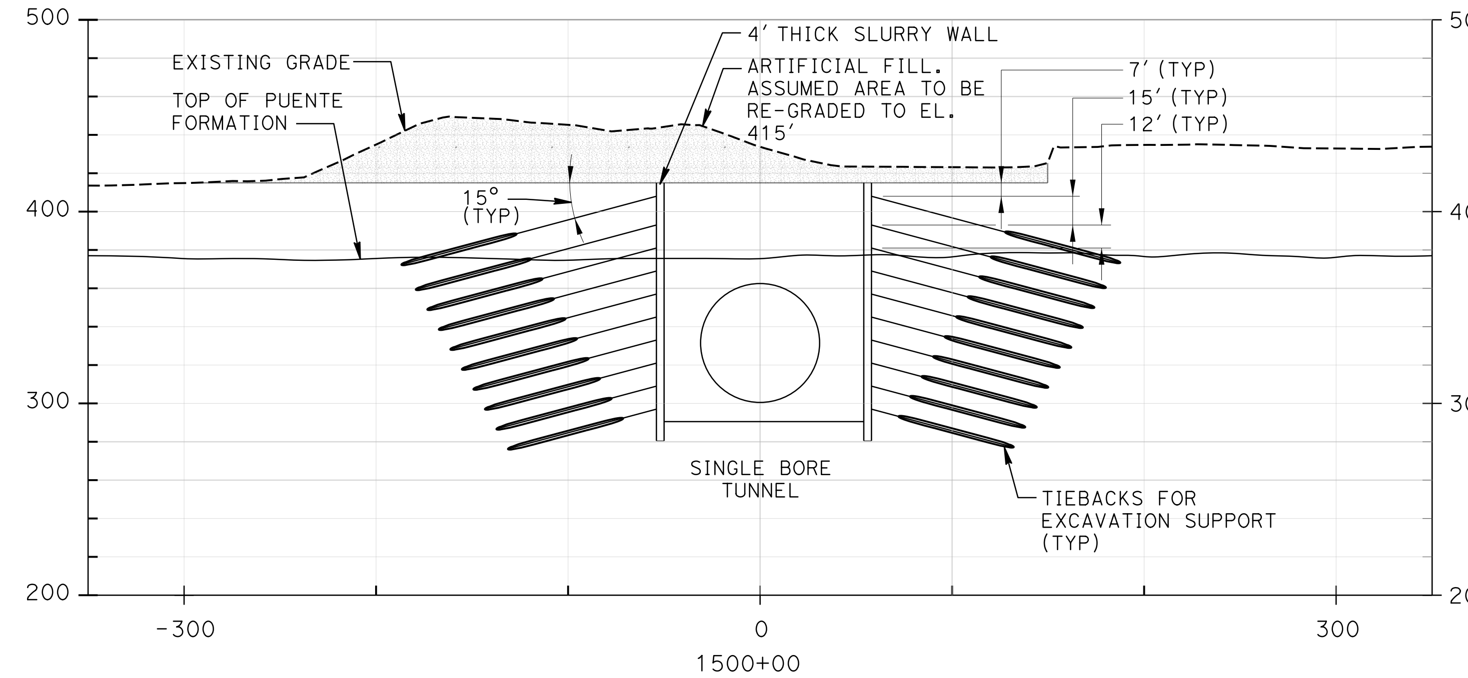
USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 11:21

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101



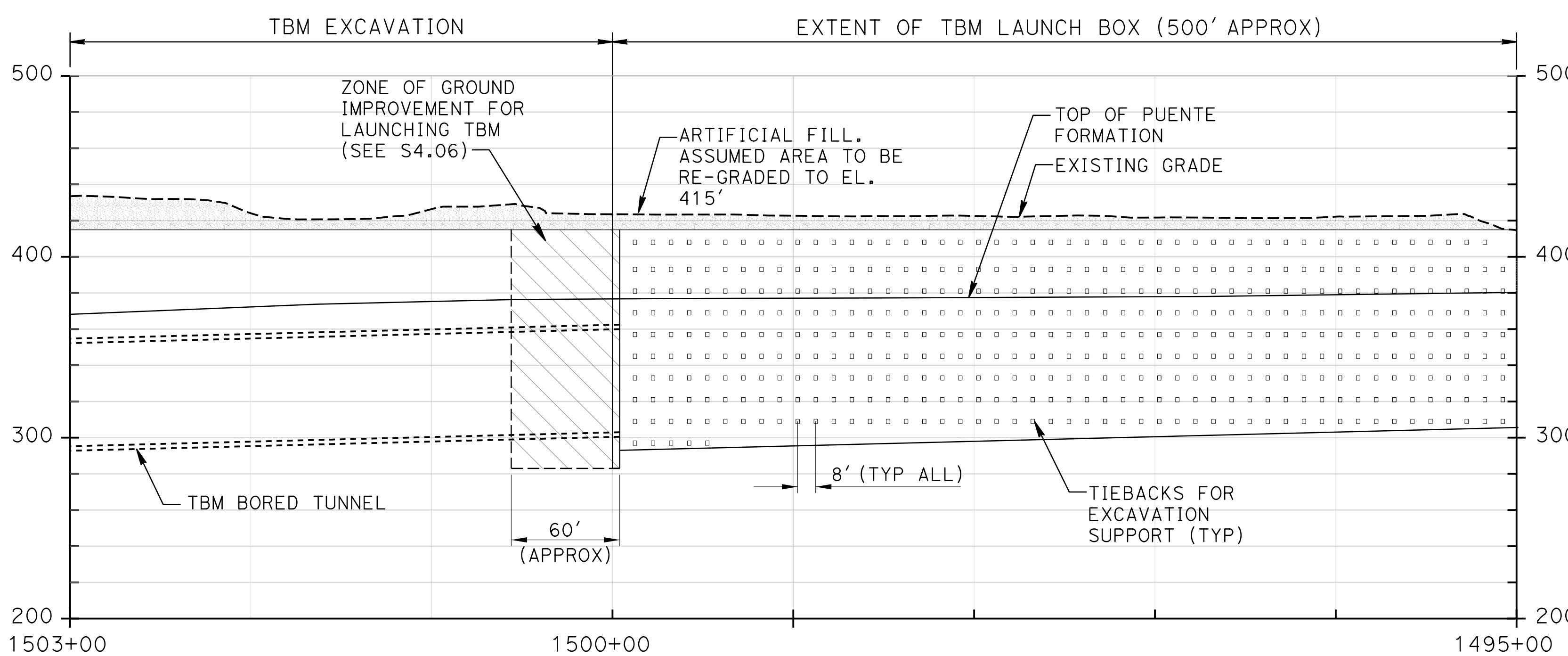
SECTION - F-7 SINGLE BORE SOUTH TBM PORTAL

SCALE: 1"=100'

A S4.03

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 60 FEET (APPROX).



SECTION - F-7 SINGLE BORE SOUTH TBM PORTAL (EAST WALL)

SCALE: 1"=100'

B S4.03

S4.04

DESIGNED BY	M. HARIHARAN	DATE 9-20-13	X PROJECT ENGINEER	**SR 710 NORTH STUDY**
DRAWN BY	J. TOLES	DATE 9-20-13		**SOUTH PORTAL SECTIONS**
CHECKED BY	M. LAWRENCE	DATE 9-20-13		BRIDGE NO. TBD / UNIT:
APPROVED	S. DUBNEWYCH	DATE 9-20-13		SCALE: 1"=100' / PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S404.dgn

CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 13:20

PREPARED FOR THE STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

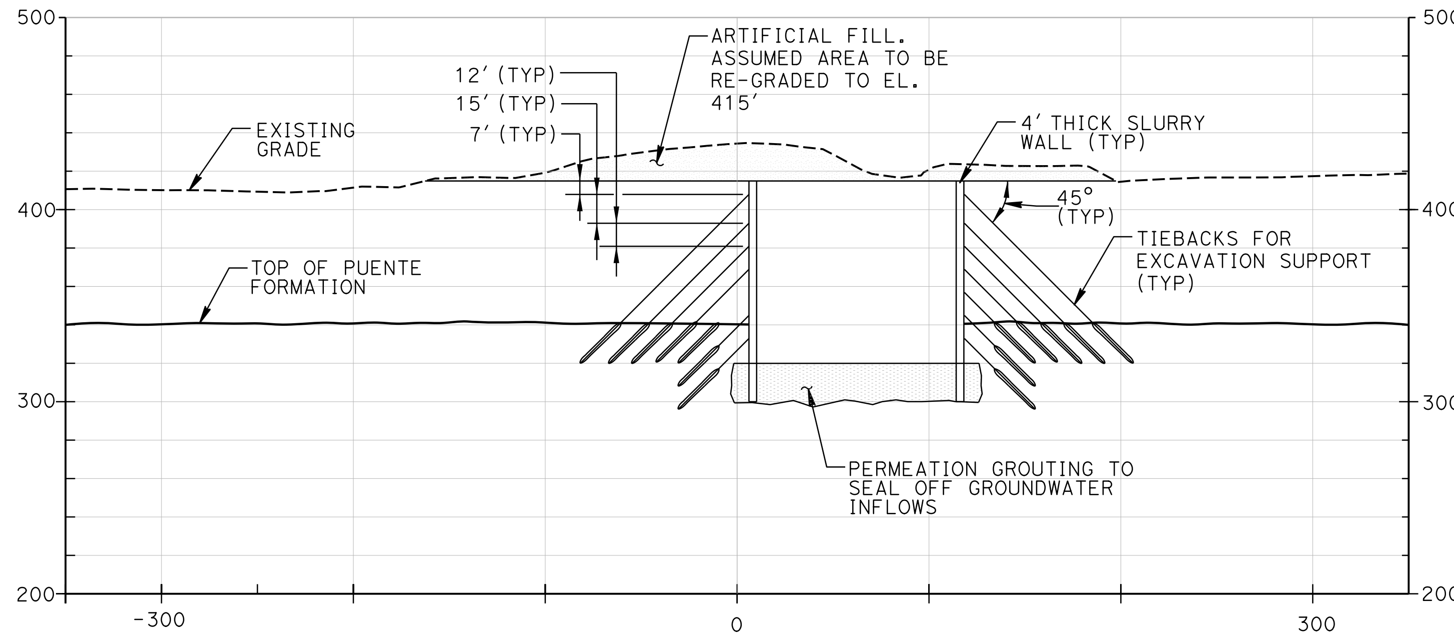
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

NOTES

1. TIEBACK BOND LENGTH SHOWN IS 30 FEET (APPROX).



SECTION - F-7 SINGLE BORE
SOUTH TBM PORTAL

SCALE: 1"=100'

C
S4.03

S4.05

DESIGNED BY	M. HARIHARAN	DATE 9-20-13	X	SR 710 NORTH STUDY
DRAWN BY	J. TOLES	DATE 9-20-13	PROJECT ENGINEER	SOUTH PORTAL SECTION
CHECKED BY	M. LAWRENCE	DATE 9-20-13		BRIDGE NO. TBD / UNIT:
APPROVED	S. DUBNEWYCH	DATE 9-20-13		SCALE: 1"=100' / PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

ADVANCE PLANNING STUDY SHEET (ENGLISH) (REV. 7/16/10)

FILE => 53-XXXX-u-S405.dgn

CONTRACT NO.: X

USERNAME => pwsvc DATE PLOTTED => 30-JAN-2015 TIME PLOTTED => 11:20

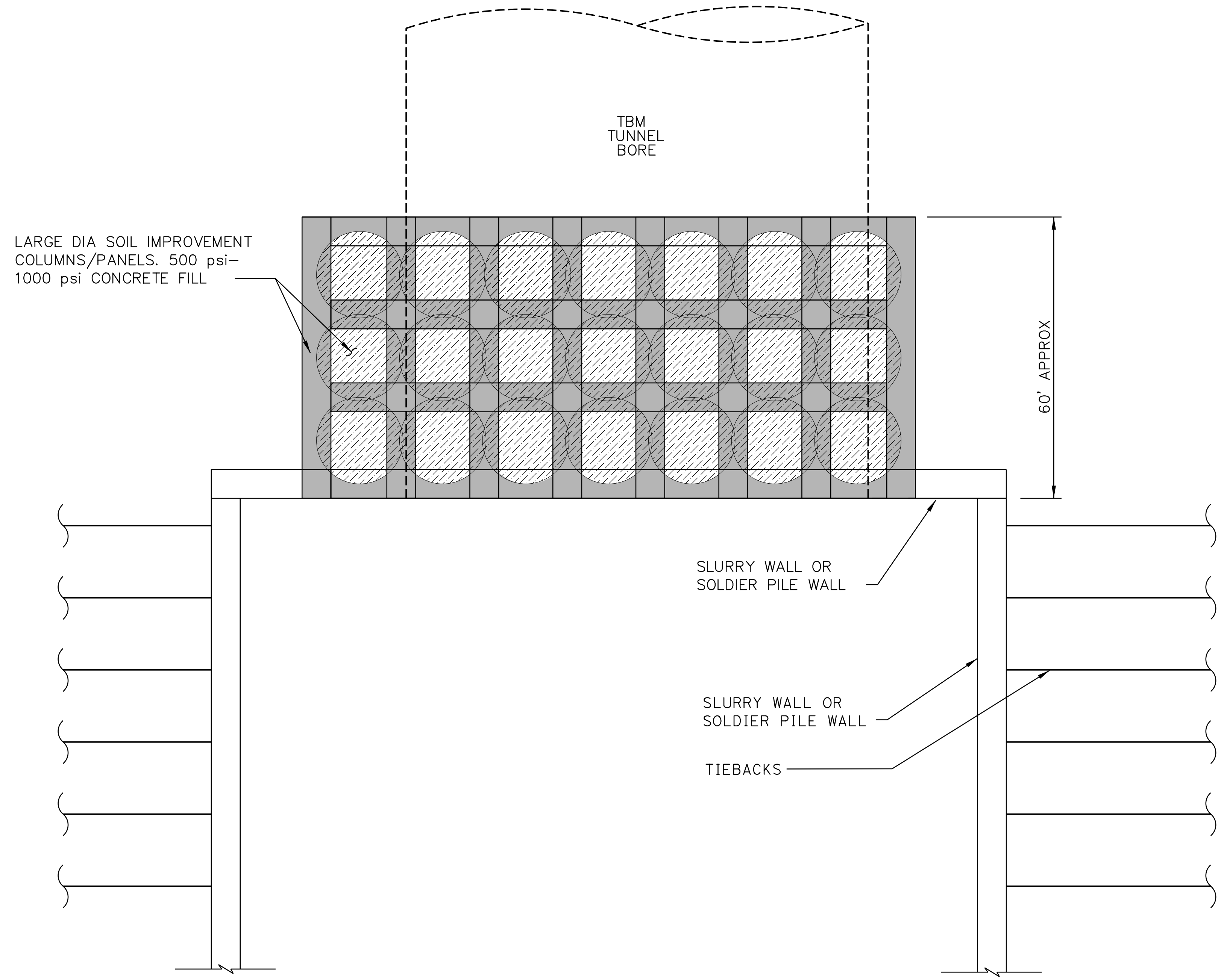
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT
7	LA	710	

METRO
ONE GATEWAY PLAZA
LOS ANGELES, CA 90012

JACOBS ASSOCIATES
234 E COLORADO BLVD, SUITE 400
PASADENA, CA 91101

NOTE:

1. APPLIES TO NORTH AND SOUTH PORTAL HEADWALL.



PORTAL HEADWALL GROUND IMPROVEMENT - PLAN

SCALE: 1"=20'

S4.06

DESIGNED BY	M. HARIHARAN	DATE	9-20-13
DRAWN BY	J. TOLES	DATE	9-20-13
CHECKED BY	M. LAWRENCE	DATE	9-20-13
APPROVED	S. DUBNEWYCH	DATE	9-20-13

X
PROJECT ENGINEER

SR 710 NORTH STUDY

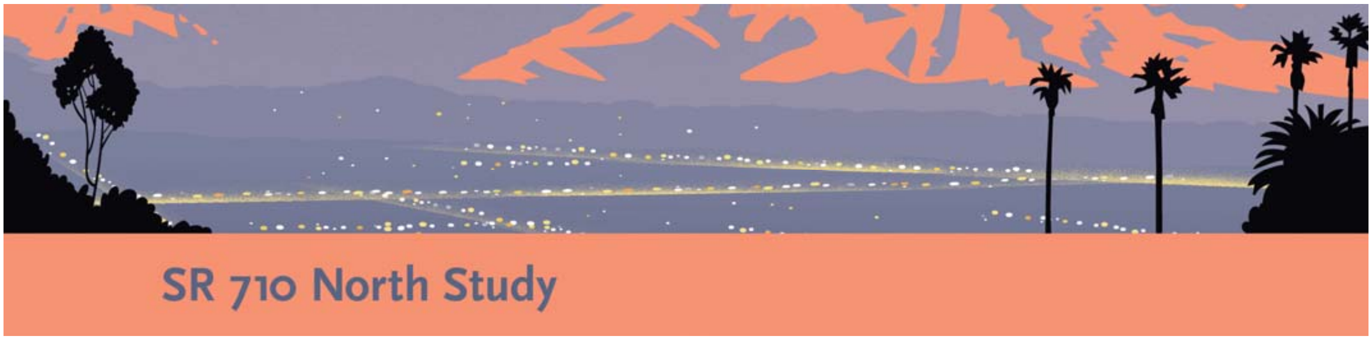
PORTAL HEADWALL

BRIDGE NO. TBD
UNIT:
SCALE: AS SHOWN
PROJECT NUMBER & PHASE:

DESIGN OVERSIGHT

SIGN OFF DATE

Appendix J
TM-8 Preliminary Design Concepts for the LRT Station and Portal Excavation Support Systems



TECHNICAL MEMORANDUM 8

Preliminary Design Concepts for the LRT Portal and Station Excavation Support Systems

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.

1.2 Task Description and Scope

This technical memorandum (TM) describes the preliminary design concepts developed for the excavation support systems for the south construction portal and underground station excavations of the LRT Alternative. The LRT tunnels are expected to be mined using two tunnel boring machines (TBMs) launched from the south portal, and the portal would also be used to stage construction activities for the tunneling work. The final, permanent structures to be constructed within this portal's excavation (i.e., the transition from the cut and cover tunnel to the at-grade rail) are beyond the scope of this TM and are not discussed herein.

Four underground stations are expected to be excavated using cut and cover techniques in advance of the TBM arrival at each location; it is common that the stations are excavated first but it's not necessary and would depend on project requirements. Along the alignment, the TBMs would break into the south end of each station, be walked through the station excavation, and recommence tunneling at the north end of the station. The alignment terminates at the Fillmore Station, and the TBMs would be retrieved from that location. Drawings for the portal and station excavations are presented in Attachment A.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2.0 Portal and Station Geometry

The LRT Alternative geometry affecting the construction portal and underground stations is described below and is what the preliminary design concepts in this TM are based on. As the vertical and/or horizontal alignment of the tunnel is optimized in future phases of the work, the design concepts should be re-evaluated and refined.

2.1 Portal

The concept for the portal excavation is shown on Drawings Y-303 and Y-304 (Attachment A) in plan and section. As shown in the section (Drawing Y-304), the height of the excavation at the headwall is approximately 50 feet. The side walls of the temporary excavation would be of a similar height near the headwall, and would decrease in height to the south as the excavation becomes shallower. The portal would ramp down from the existing ground surface to gain enough cover to launch the TBMs; both TBMs for the LRT alternative would be launched at the headwall of this portal.

2.2 Stations

A typical station, the Alhambra Station, is shown on Drawings Y-301 and Y-302 (Attachment A) in plan and section. Other stations, including Huntington, South Pasadena, and Fillmore, are shown on drawings Y-305, 306, and 307, respectively, in plan. All four stations are located predominantly within the limits of city streets, and while the footprint of each station varies based on operational needs, the basic configuration is similar for all four stations. Two of the four stations would be larger than a typical station because additional length for either a crossover or tail tracks are expected to be excavated immediately adjacent to them. The station excavations are approximately 80 to 100 feet deep and 80 feet wide, with the length of each excavation varying from 400 feet for a standard station up to over 1,000 feet if there is a crossover/tail track excavation adjacent to it. It is expected that the stations would be excavated prior to the TBMs reaching each station, and therefore the TBMs would need to break into and out of each station at the north and south walls (the headwalls) of each station.

3.0 Anticipated Geotechnical Conditions

Anticipated geotechnical conditions were evaluated based on geological data contained in the Preliminary Geotechnical Report (CH2M HILL, 2014), and are detailed in the following sections. In addition to what is detailed below, it should be noted that there is a low to moderate potential of encountering naturally occurring oil and/or gas, most likely within that Puente Formation, along the underground portions of the LRT Alternative.
As more geotechnical information becomes available in future phases of this study, the groundwater levels and geologic contacts should be better refined, and the vertical alignment should be optimized based on the updated information.

3.1 Portal

Figure 1 shows a generalized geologic profile through the portal area. It consists of approximately 50 to 70 feet of alluvium overlying the Puente Formation siltstone/claystone. The alluvium at this location consists of interlayered sandy silts, silty sands, clayey sands and lean and fat clay layers. The granular layers are medium dense to dense, and the cohesive layers are generally stiff. The Puente Formation at the south portal is a weak to very weak siltstone/claystone with UCS values in the range of 100 to 200 psi and short-term undrained shear strengths in the range of 75 to 120 psi; however, the LRT portal would be excavated entirely in the alluvium. The water table at this location is approximately 25 to 40 feet below grade, and would likely be encountered at the deeper portions of the excavation, for example near the headwall.

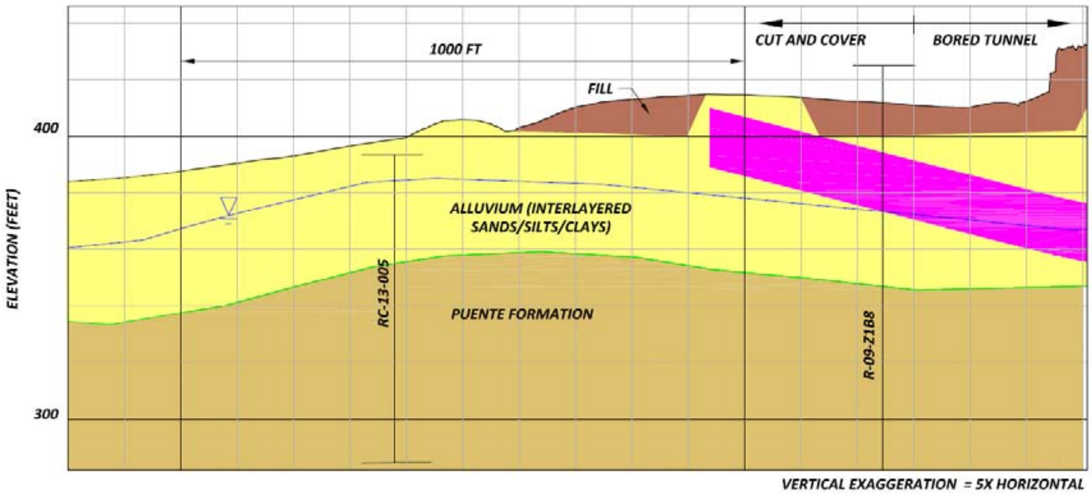


Figure 1: Generalized Geologic Profile at the LRT Portal

3.2 Stations

Figures 2, 3, 4 and 5 show a generalized geologic profile at each of the four underground stations.

3.2.1 Alhambra Station. The Alhambra Station is expected to be predominantly in alluvium, with approximately 20 feet at the bottom of the excavation in the Fernando Formation. Based on Boring R-09-Z4B4, the alluvium in this area consists predominantly of silty sand (SM) with poorly-graded sand (SP) and sandy silty clay (CL-ML) interlayers. The granular layers are dense to very dense and the cohesive layers are stiff to very stiff. The Fernando Formation at this location is a siltstone/claystone and is highly weathered for the upper 40 feet, after which the weathering changes to moderate. No testing has been performed at this location on the Fernando Formation, but the highly weathered nature of the Fernando Formation would likely produce soil-like behavior. Groundwater is expected in the bottom 20 to 30 feet of this station’s excavation (CH2M HILL, 2014).

3.2.2 Huntington Station. The Huntington Station is expected to be excavated through an area with a sloping alluvium/rock contact. The thickness of the alluvium ranges from approximately 30 feet at the southern end of the station to 80 feet (the total depth of the excavation) at the northern end. The alluvium is underlain by the Topanga Formation, and groundwater is expected approximately 40 feet below the base of the excavation. Based on boring A-13-008 (drilled in the alluvium) the alluvium in this area consists of interlayered silty sand (SM) and well-graded sands (SW-SM), and the sands are dense to very dense. Based on boring A-13-021 which was drilled in the Topanga Formation near the proposed Huntington Station, the bedrock consists of siltstone with some sandstone and claystone interbeds. SPT N values were obtained in the rock up to approximately 85 feet depth below grade, after which the spoon refusal was obtained; these SPT N values were in the range of 50 to 100 blows per foot. Laboratory strength tests have not been performed on rock cores near this location.

3.2.3 South Pasadena Station. The South Pasadena Station is expected to be excavated entirely in alluvium situated above the groundwater table. The alluvium in this area consists of predominantly of poorly-graded sands (SP) with clay and gravel inclusions (SP-SC) and silty sand (SM) interlayers. The sands are medium dense to dense and the groundwater in this area ranges from approximately 20 to 40 feet below the base of the excavation.

3.2.4 Fillmore Station. The Fillmore Station is expected to be excavated entirely in alluvium situated above the groundwater table. The alluvium in this area consists of interlayered poorly-graded sands (SP), silty sands (SM), and sandy gravels (GP). A few sandy silt (ML) interlayers are also present. The sands are dense to very dense, and the sandy silts are hard. The groundwater in this area is expected to range from approximately 40 to 50 feet below the base of the excavation.

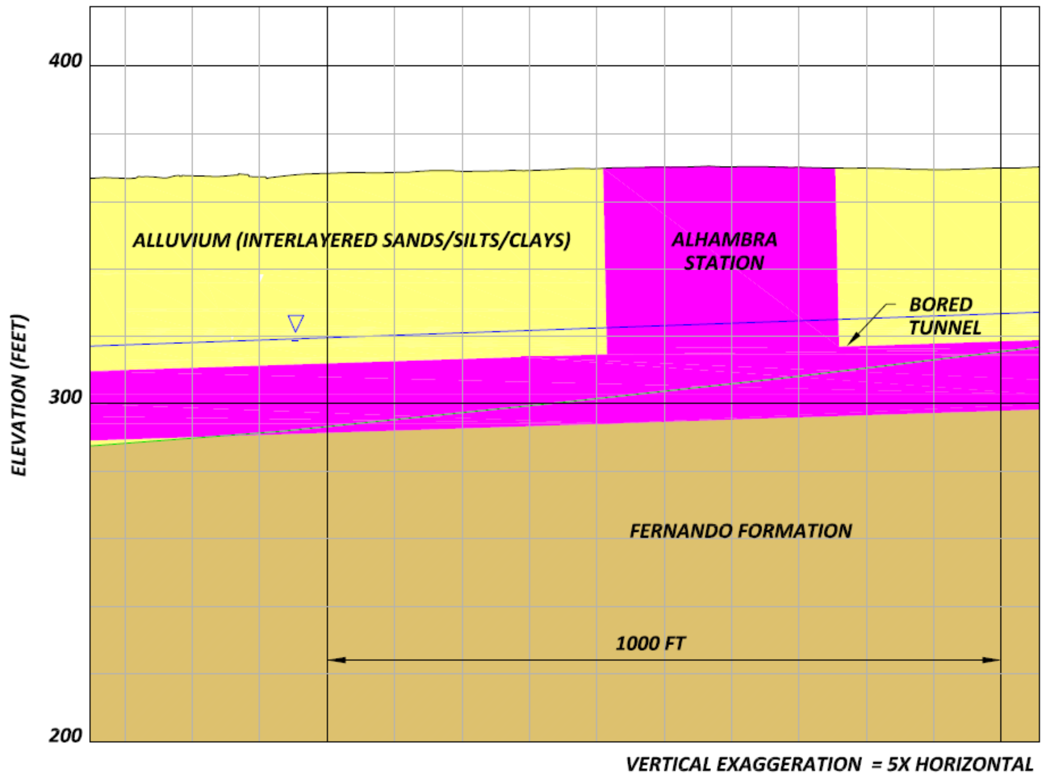


Figure 2: Generalized Geologic Profile at the Alhambra Station

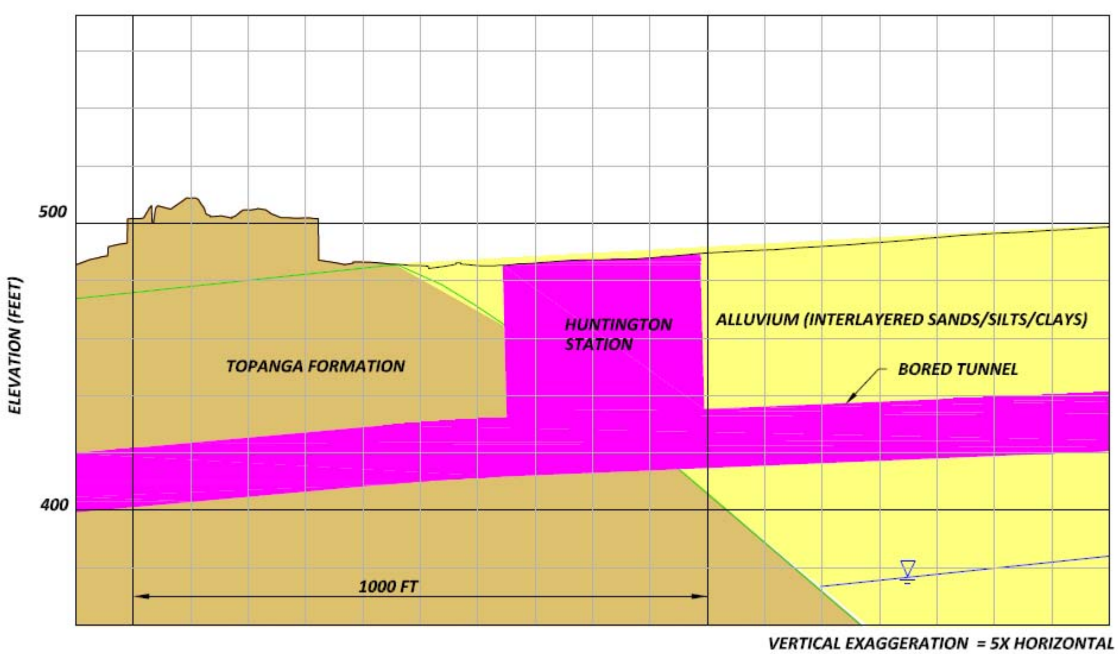


Figure 3: Generalized Geologic Profile at the Huntington Station

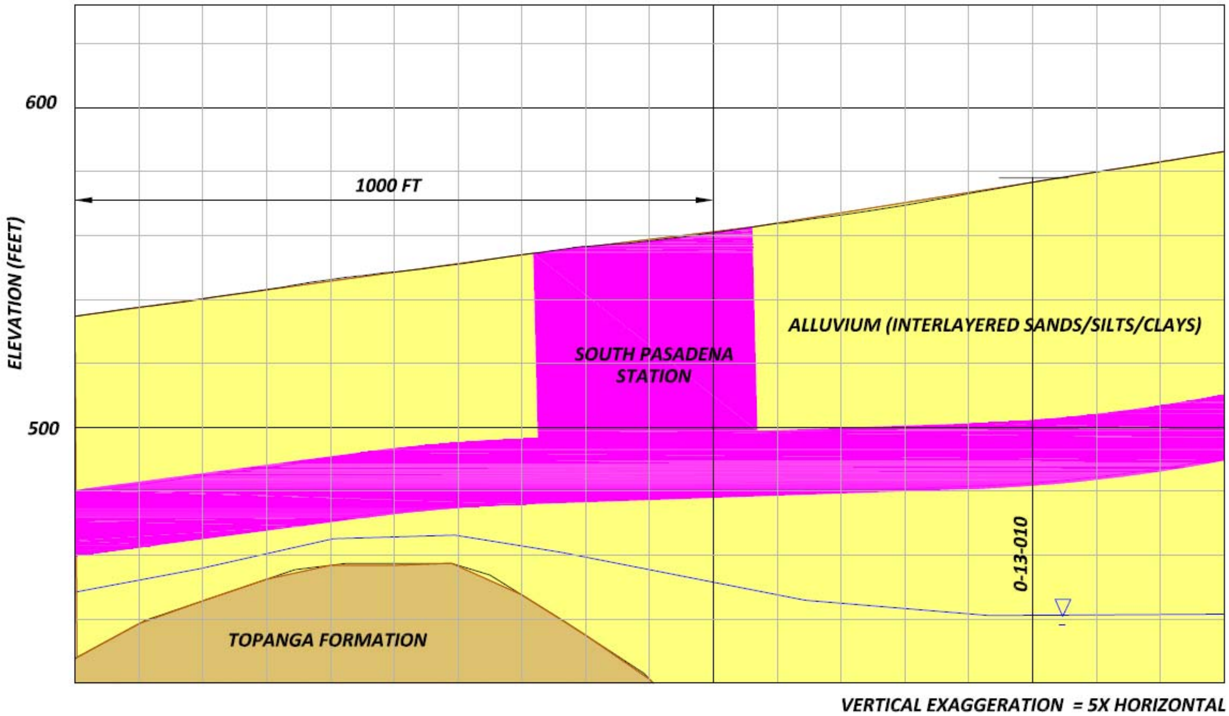


Figure 4: Generalized Geologic Profile at the South Pasadena Station

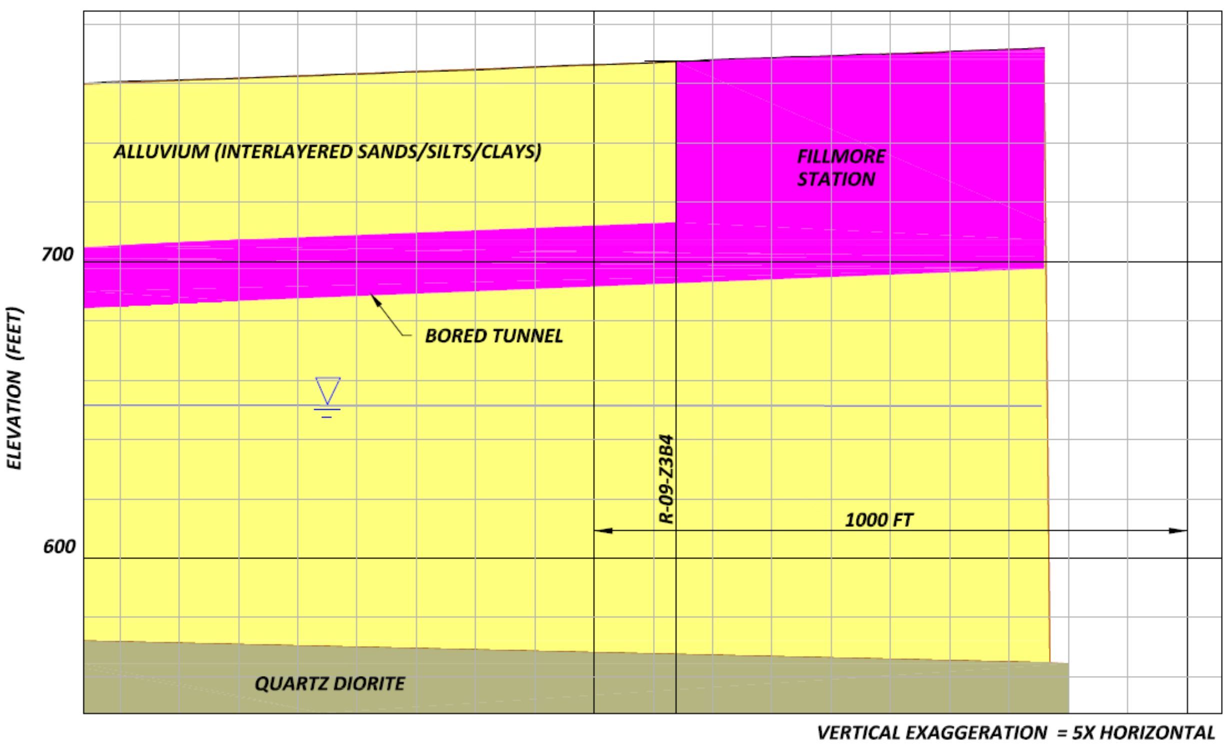


Figure 5: Generalized Geologic Profile at the Fillmore Station

4 Design Considerations

4.1 Portal

At the portal, the depth of excavation is approximately 40 to 50 feet (refer to drawing Y-304 in Attachment A). Geotechnical conditions indicate alluvial soils within the excavated height and that groundwater could potentially be encountered in the deeper portions of the excavation. e.g., near the portal headwall. As one moves away from the headwall to the south, the excavation becomes shallower and therefore the water table would be below the excavation base. The main issue would be preventing the alluvium from sloughing into the excavation, and a secondary issue would be controlling groundwater near the portal headwall. Either a continuous wall system or a wall system such as soldier piles and lagging can be used for this purpose. In a soldier pile and lagging wall system excessive sloughing is prevented by limiting the excavation lifts to no more than 4 to 5 feet and excavating the portal footprint in limited areas at a time. Limited dewatering with wells may be used for groundwater control as necessary near the portal headwall.

4.2 Stations

At the four underground stations, the depth of the excavations range from approximately 80 to 100 feet, and the geotechnical conditions indicate that they would be excavated wholly in alluvial soils or in a combination of alluvial soils and weak sedimentary rock. The Alhambra Station has groundwater in the bottom 20 or so feet of the excavation. All stations are excavated in an urban setting in the public right-of-way with buildings and structures immediately adjacent to the excavations. Four primary design considerations for the station excavation support systems include:

- Preventing caving and sloughing during excavation, which could lead to settlement-induced damage of nearby buildings, utilities, and other existing facilities.
- Installing temporary wall systems that would not deflect or move excessively to mitigate settlement-induced damage of nearby buildings, utilities, and other existing facilities.
- Dewatering where necessary if non-watertight excavation support methods are used.
- Maintaining street traffic over the excavations during excavation and construction of the permanent works.

Another important issue would be temporary and permanent utility relocations that would be required before the excavation support systems are installed; however, this consideration is not within the scope of this memorandum.

4.3 Headwall Support

At the headwalls of the four stations and at the portal, neither steel reinforcing nor steel piles can be used where the TBMs would penetrate the walls. In addition, the headwalls cannot be supported with tiebacks, since a TBM has to mine through this zone (behind the headwall). Given the small span of the wall over the tunnel bores (approximately 22 feet), a variety of headwall support methods which do not have steel support elements could be feasible. These range from self-supporting headwalls (gravity walls) to secant/slurry walls with fiber glass (GRP) reinforcing over the tunnel zone.

5 Recommended Preliminary Design Details

The following sections detail the preliminary design concepts used for this phase of the study. The details presented below are expected to change in future phases of this project as more geotechnical information becomes available and/or when the alignment is optimized.

5.1 Portal

Several wall types can be considered for the portal excavation support system including continuous wall systems, soldier piles and lagging, and soil mix walls (auger or cutter soil mix). These wall types can be installed to the depths required fairly easily. Among these, a soldier pile and timber lagging wall supported with tiebacks appears most suitable from a constructability and structural design standpoint.

For the wall depth, which is in the 40-foot range, this is also an economical wall type. This wall type would also satisfy the design considerations set forth in the above section. These walls are not watertight; therefore, a limited dewatering effort may be necessary for groundwater control. Soldier piles in this design concept are expected to be of steel HP14 sections spaced laterally at 8 feet on center. Tiebacks can be installed through the soldier pile itself, eliminating the need for walers. The horizontal tieback spacing is then the same as the pile spacing. Tieback loads are expected to be in the range of 100 kips for the arrangement shown on Drawing Y-304, with a bond length of approximately 30 feet.

5.2 Stations

A key design consideration for the station excavations is allowing traffic over the excavations. This requires the excavation support walls to support a decking system and places substantial vertical loads on the walls in addition to the lateral wall loads. Wall types that are suitable for this loading are soldier pile and lagging walls, soil mix walls, and secant pile walls. Soldier piles walls have been successfully used for past Metro projects under similar loading conditions and therefore have been selected as the wall type at this conceptual design level. These walls would also satisfy the design considerations with respect to limiting settlement from soil sloughing and wall movements.

Soldier piles are expected to be of steel W24 sections at 8 feet on center with timber lagging. Tiebacks or internal bracing can be used for lateral wall support. Currently, the drawings in Attachment A show that the excavation is supported using tiebacks alone; however, internal bracing may be used in case of utility conflicts and right-of-way issues. Tiebacks can be installed through the soldier pile itself, and therefore the horizontal tieback spacing is the same as the pile spacing, which is 8 feet. Tieback loads are expected to be in the range of 200 to 300 kips for the arrangement shown on Drawing Y-302, with a bond length of approximately 40 feet. Internal bracing, if used, would consist of walers and struts. Walers are expected to be W36 size wide flange beams and struts are expected to be 24- to 36-inch-diameter steel pipe sections.

5.3 TBM Thrust Frame

A TBM thrust frame would be necessary at the portal and the three stations from which the TBMs would mine (it would not be required at the Fillmore Station as the TBMs would be removed from that station). The frame typically would be supported by a reinforced concrete base slab at the bottom of the portal or station excavation. This slab would be of substantial thickness in order to provide the required horizontal and vertical reaction to the TBM frame. This slab can be a temporary slab that is subsequently demolished or the permanent base slab needed for the permanent structure with some modifications, based on the requirements for final use. Preliminarily it is assumed that the base slab would be in the range of 4 to 6 feet thick.

5.4 Headwall Support

As explained in the design considerations, no structural steel elements can be present within the zone through which the TBMs would excavate. To meet these requirements, a gravity wall concept could be used for the portal and station headwalls in the zones through which the TBMs would excavate. To achieve this, a substantial mass of soil at the headwall could be improved to the point that it becomes self-supporting. Several methods were considered to achieve this self-supporting improvement, including chemical grouting, jet grouting, and deep soil mixing. Soil nailing with glass-reinforced plastic (GRP) soil nails was also considered on its own and in conjunction with the above methods. The method selected should be feasible in the ground types that exist at the portals and stations and should also provide the required strength after improvement.

Chemical (sodium silicate) grouting is expected to be feasible at several locations except those where the alluvium is too fine-grained to allow the sodium silicate to penetrate. Use of this method would require detailed information regarding grain size distribution at each of the headwall locations. Jet grouting and deep soil mixing are both feasible based on depth and anticipated geologic conditions at the portal and stations. Jet grouting essentially erodes the soil and mixes the cuttings in-situ with cement slurry to form stabilized columns. These columns overlap or interlock and can be designed to act as a gravity wall. Soil mixing uses continuous auger flights to also mix the soil with a cement slurry to form interlocking columns. Soil mixing has a disadvantage that it cannot be installed at locations where utilities are present, since the auger flights are of large size and continuous.

At the headwall locations, an additional 60- to 80-foot zone at each side of the station excavation has to be further cleared of utilities to install the soil mix columns. For this reason, jet grouting would be preferred over soil mixing for headwall support at this time.

Soil nailing with a shotcrete face would work well at locations where the ground has sufficient cohesion (i.e., standup time long enough to excavate and install the shotcrete face support). However, soil nailing may not work well at locations where the ground has little cohesion. Similar to the chemical grouting, use of this method would require detailed information regarding soil composition at each of the headwall locations.

For the portal and stations, fairly wide zones of jet grouting would be required based on the soil type and depth of the excavations. One method to reduce the thickness of the improvement zone is to use GRP soil nails in conjunction with these methods, so that the headwall acts as a semi-gravity wall, deriving some support from its weight and some from the soil nails.

At this time, the improvement shown is a jet grouted zone acting as a gravity wall. The improvement is the full height of the wall and covers the zone through which the TBM would excavate. The drawings in Attachment A show details of this type of ground improvement. The one exception is at the south headwall at the Huntington Station, which is almost entirely in the Topanga formation. Here, headwall support would likely consist of GRP soil nails/dowels and a shotcrete face.

6 Recommendations for Future Design Evaluations

The preliminary design concepts presented herein should be advanced in future stages of the design. Additional field and laboratory investigations will be required to perform more advanced design of the excavation support systems. Recommendations for future design evaluations include, but are not limited to, the following:

- Additional geotechnical investigations to better define the thickness and extents of the various formations and the ground water table and to identify the presence of naturally-occurring gas.
- Pressuremeter testing to determine soil modulus and K_0 at both the stations and portal. These tests can also be used to estimate bond stress values for tiebacks.
- Moisture density and grain size tests to evaluate the feasibility of various ground improvement methods at the portal and station headwalls.
- Establish seismic design parameters for the excavation support walls during construction of the tunnels.
- Environmental testing to determine requirements for groundwater disposal or treatment from potential dewatering.
- In the design and construction planning for the portal and station excavations, requirements for controlling ground movements and preventing damage to structures should be evaluated in conjunction with selecting a method of excavation support (including its stiffness), the excavation sequencing, and monitoring requirements.

7 Reference

CH2M HILL. 2014. *Preliminary Geotechnical Report, SR 710 North Study, Los Angeles County, California.* Prepared for Metro. March.

8 Revision Log

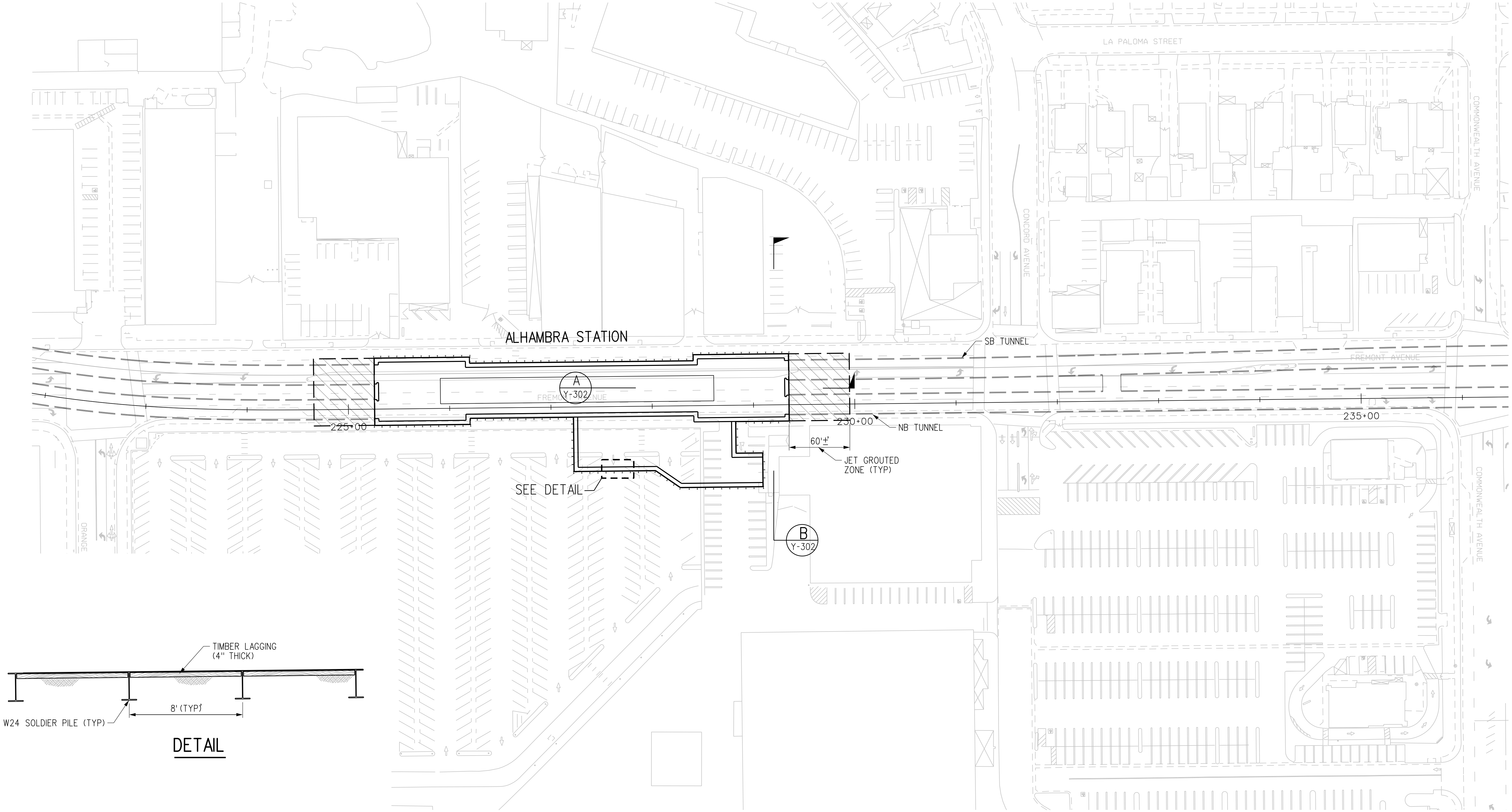
Revision 0	September 30, 2013	Internal Review
Revision 1	October 11, 2013	Metro/Caltrans Review
Revision 2	May 27, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

Drawings

Metro

Caltrans



PRELIMINARY

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THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

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DESIGNED BY M. HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TYPICAL UNDERGROUND STATION
EXCAVATION SUPPORT PLAN

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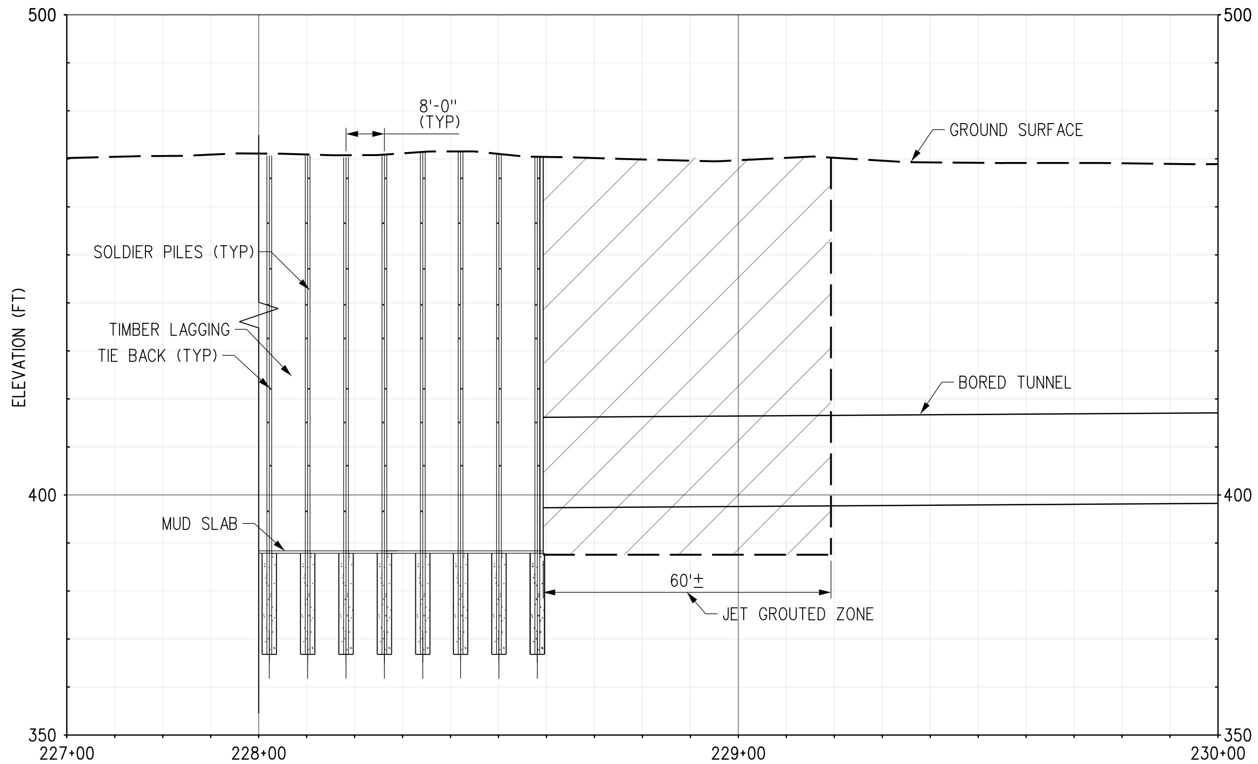
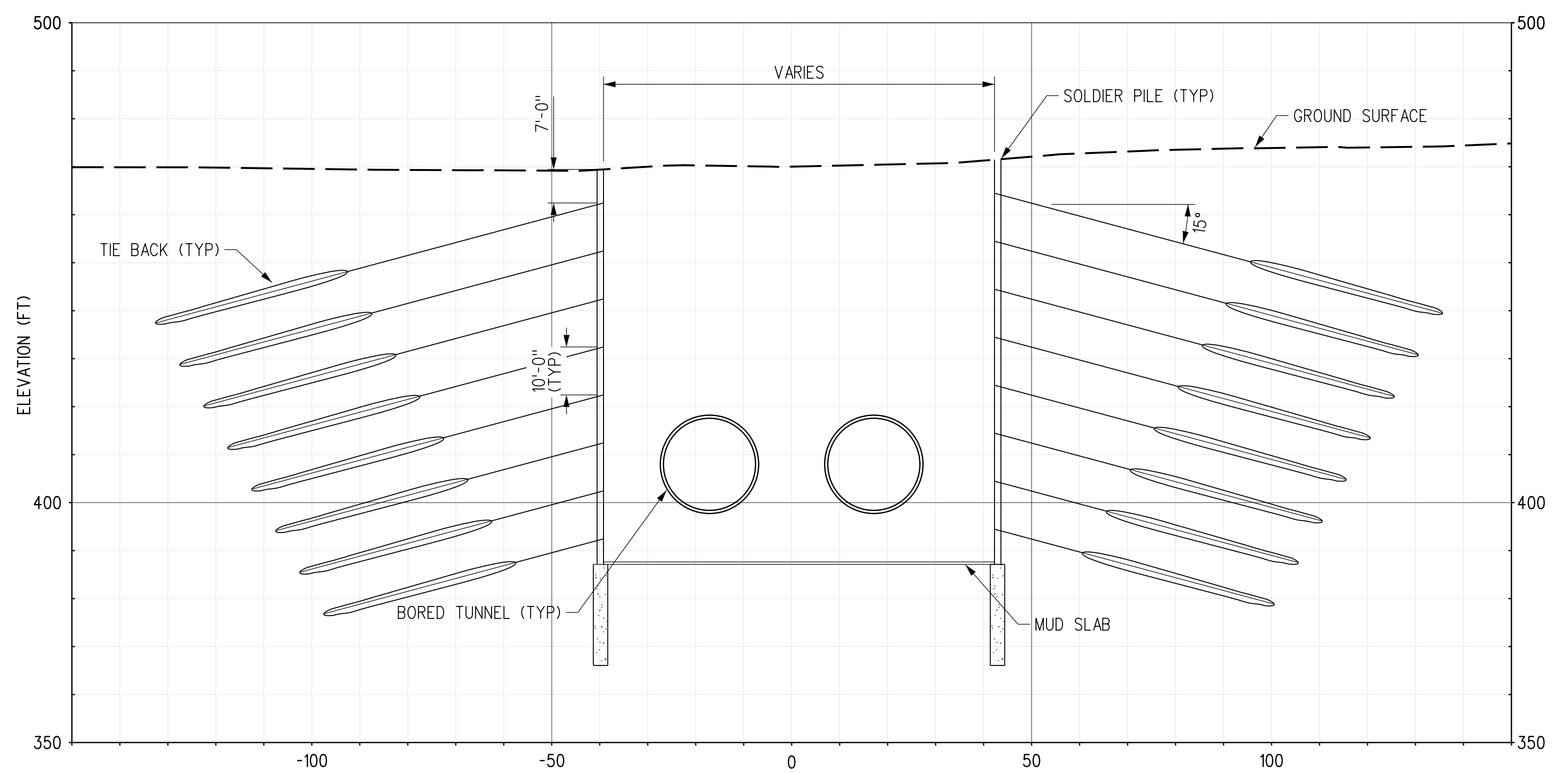


TYPICAL SECTION A (Y-302, Y-301)



TYPICAL SECTION B (Y-302, Y-301)

NOTES:

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2. TIEBACK BOND LENGTH SHOWN IS 40 FEET (APPROX.)

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PRELIMINARY

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

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-	2/7/14						METRO COMMENTS

DESIGNED BY M. HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE S. DUBNEWYCH
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TYPICAL UNDERGROUND STATION
EXCAVATION SUPPORT
SHEET 2 OF 2

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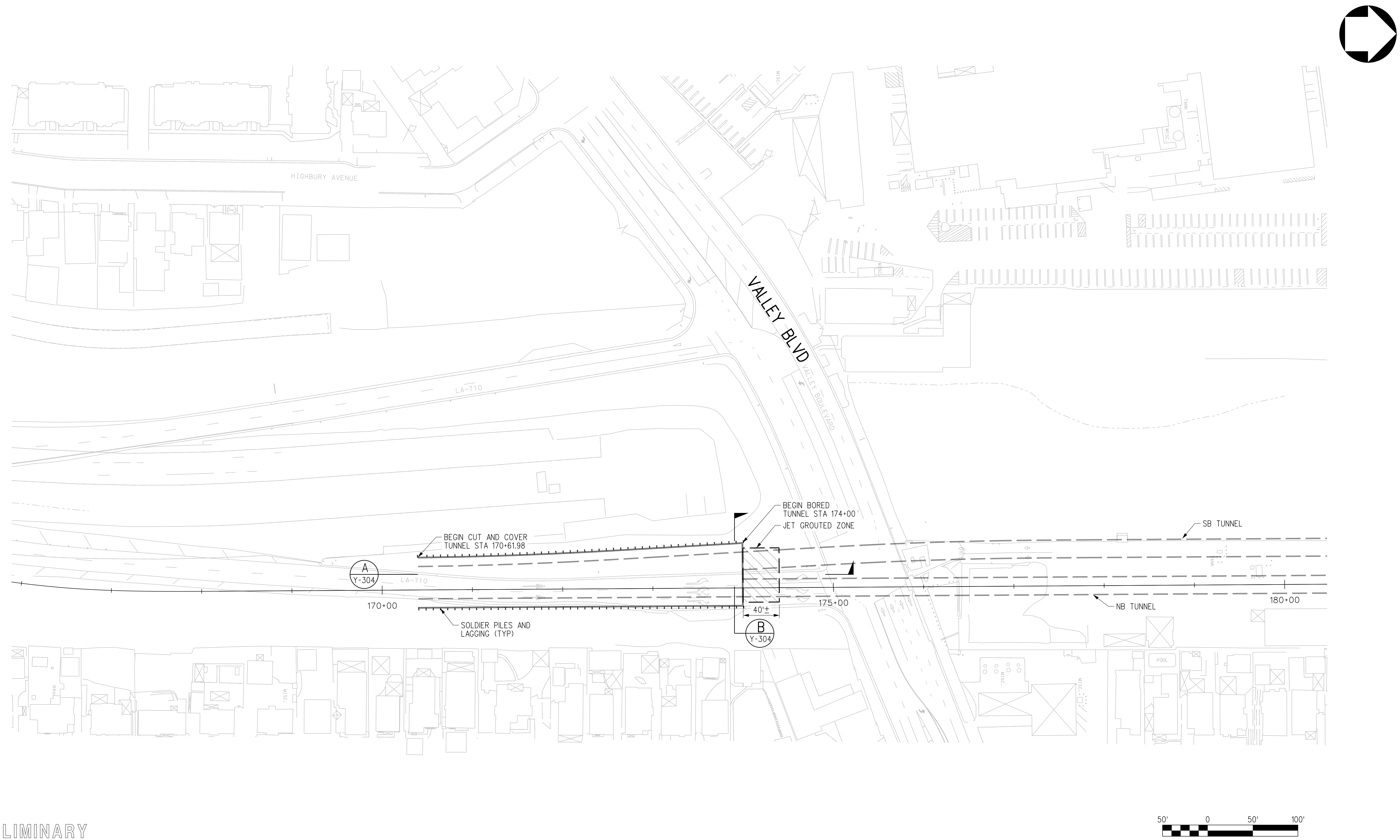

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DESIGNED BY M. HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TBM LAUNCH PORTAL
EXCAVATION SUPPORT

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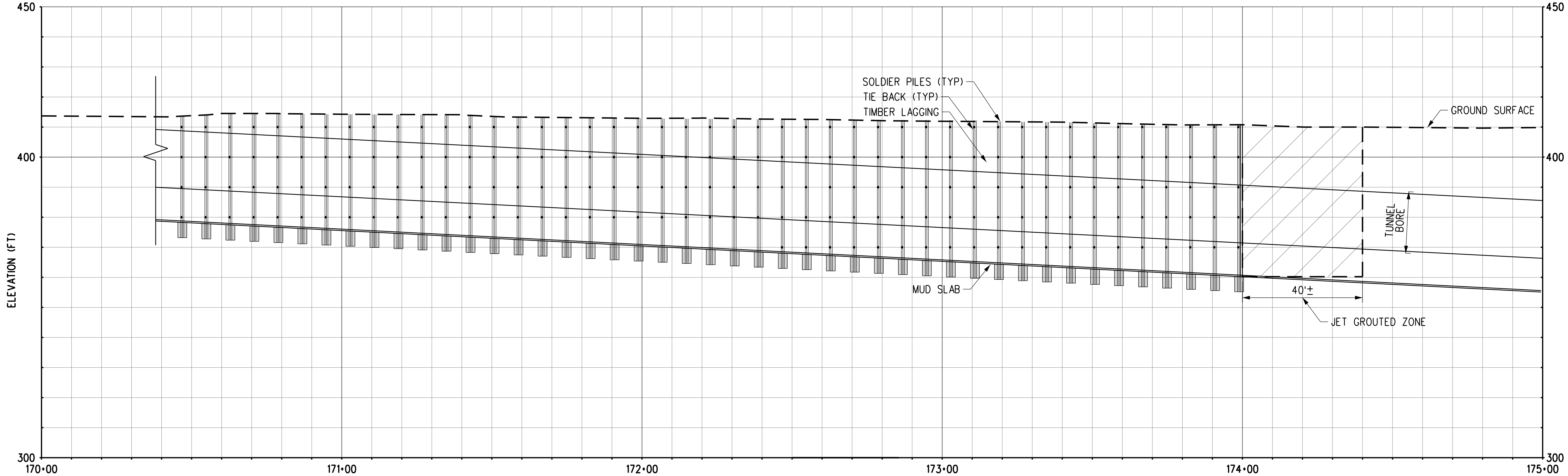
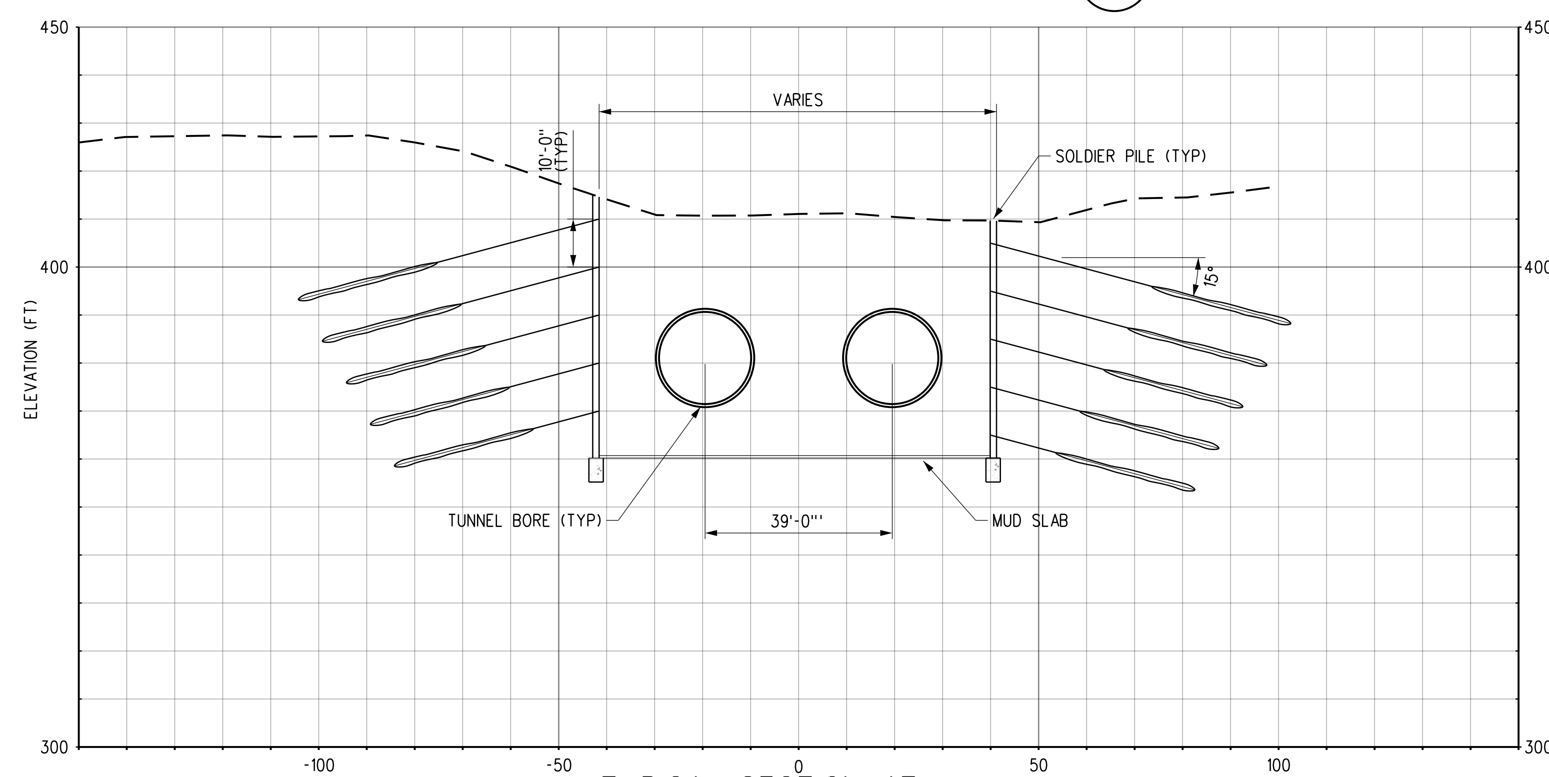


TYPICAL SECTION AT
PORTAL SIDEWALL

A Y-304 Y-303



TYPICAL SECTION AT
PORTAL HEADWALL

B Y-304 Y-303

NOTES:

1. THE TEMPORARY SUPPORT SHOWN IS SCHEMATIC ONLY.
2. TIEBACK BOND LENGTH SHOWN IS 30 FEET (APPROX.)

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PRELIMINARY

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REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

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DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
TBM LAUNCH PORTAL
EXCAVATION SUPPORT SECTIONS

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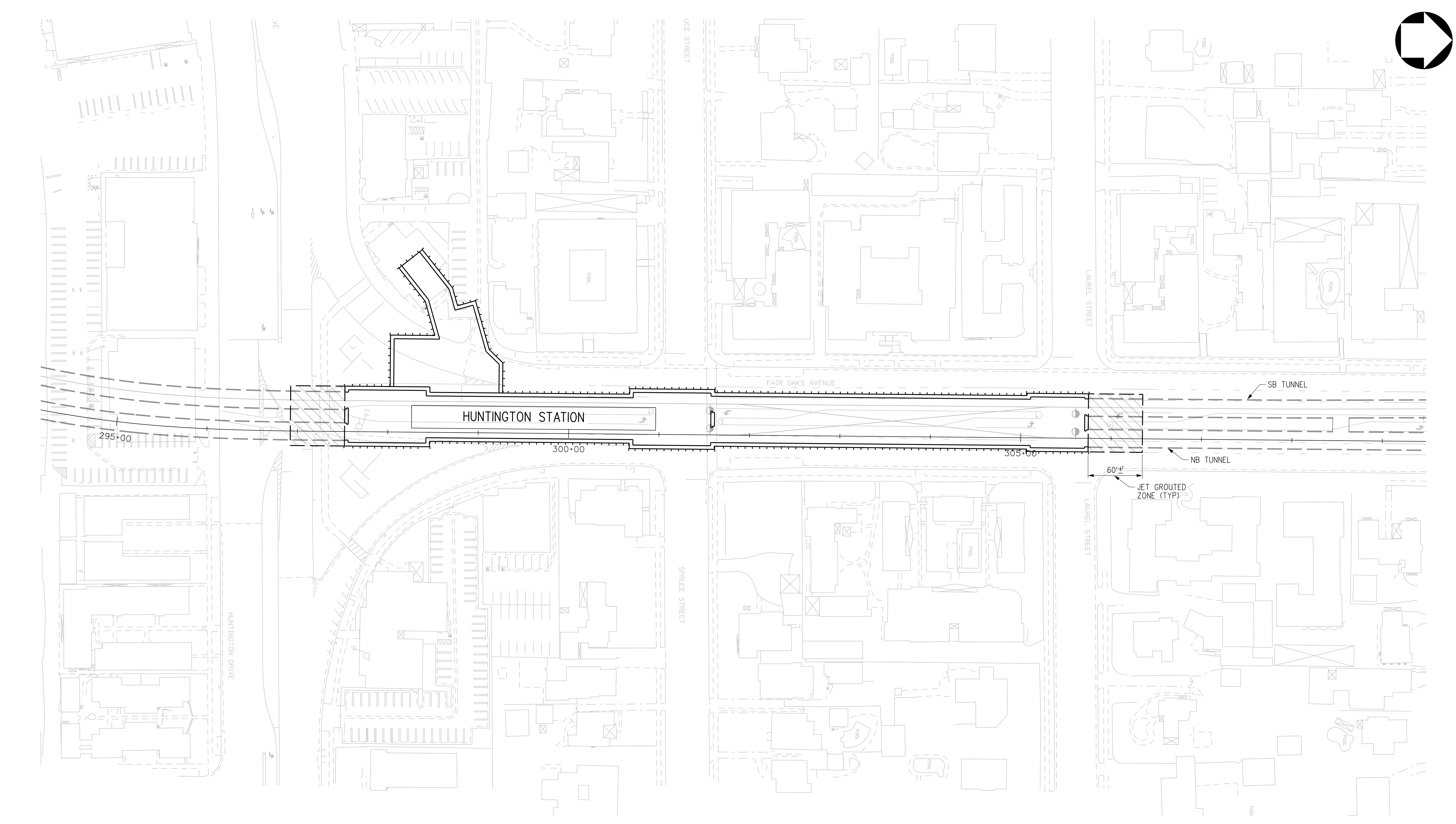

PRELIMINARY

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CHECKED BY M. LAWRENCE
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
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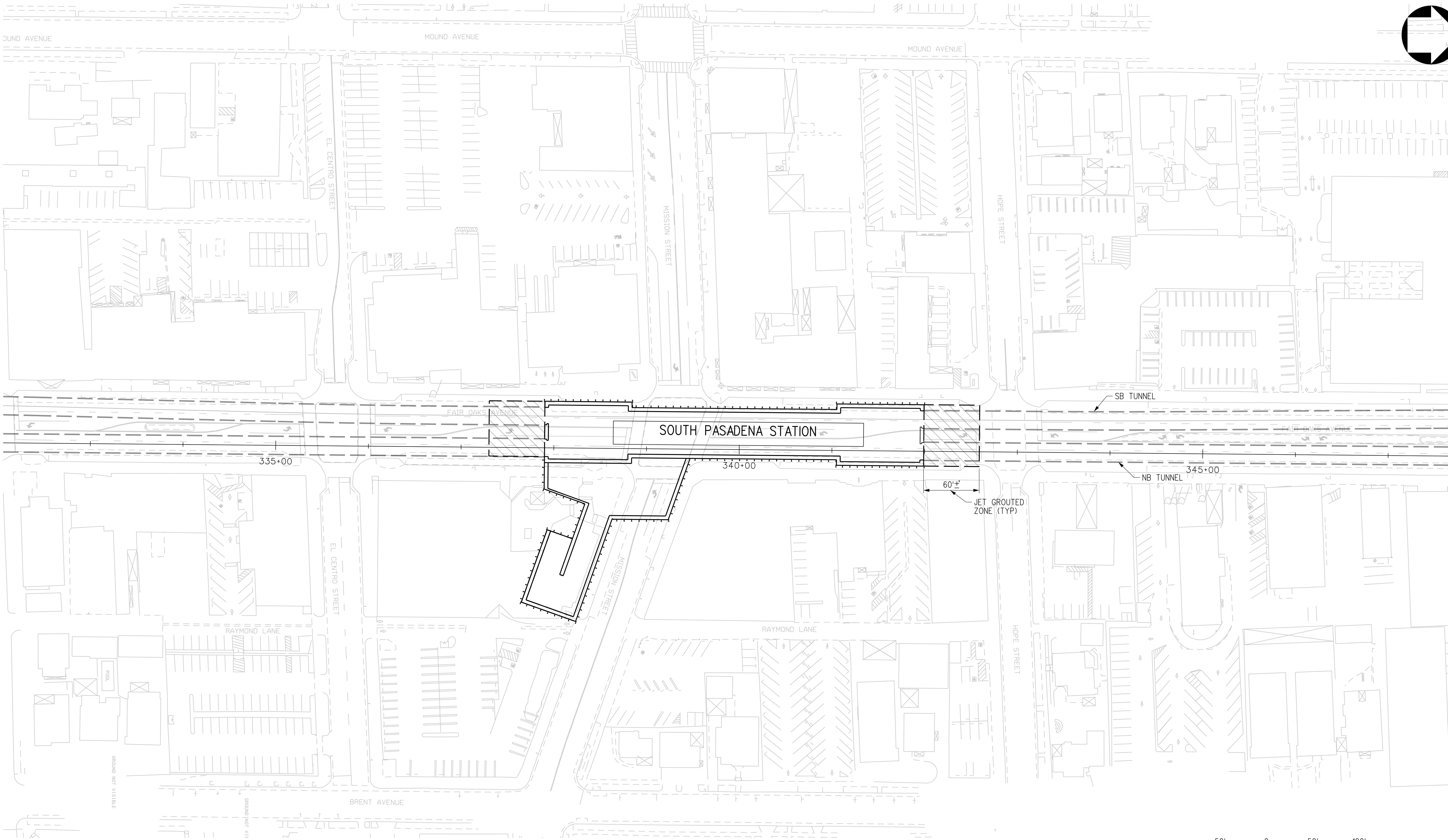

PRELIMINARY

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DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
SOUTH PASADENA STATION
EXCAVATION SUPPORT PLAN

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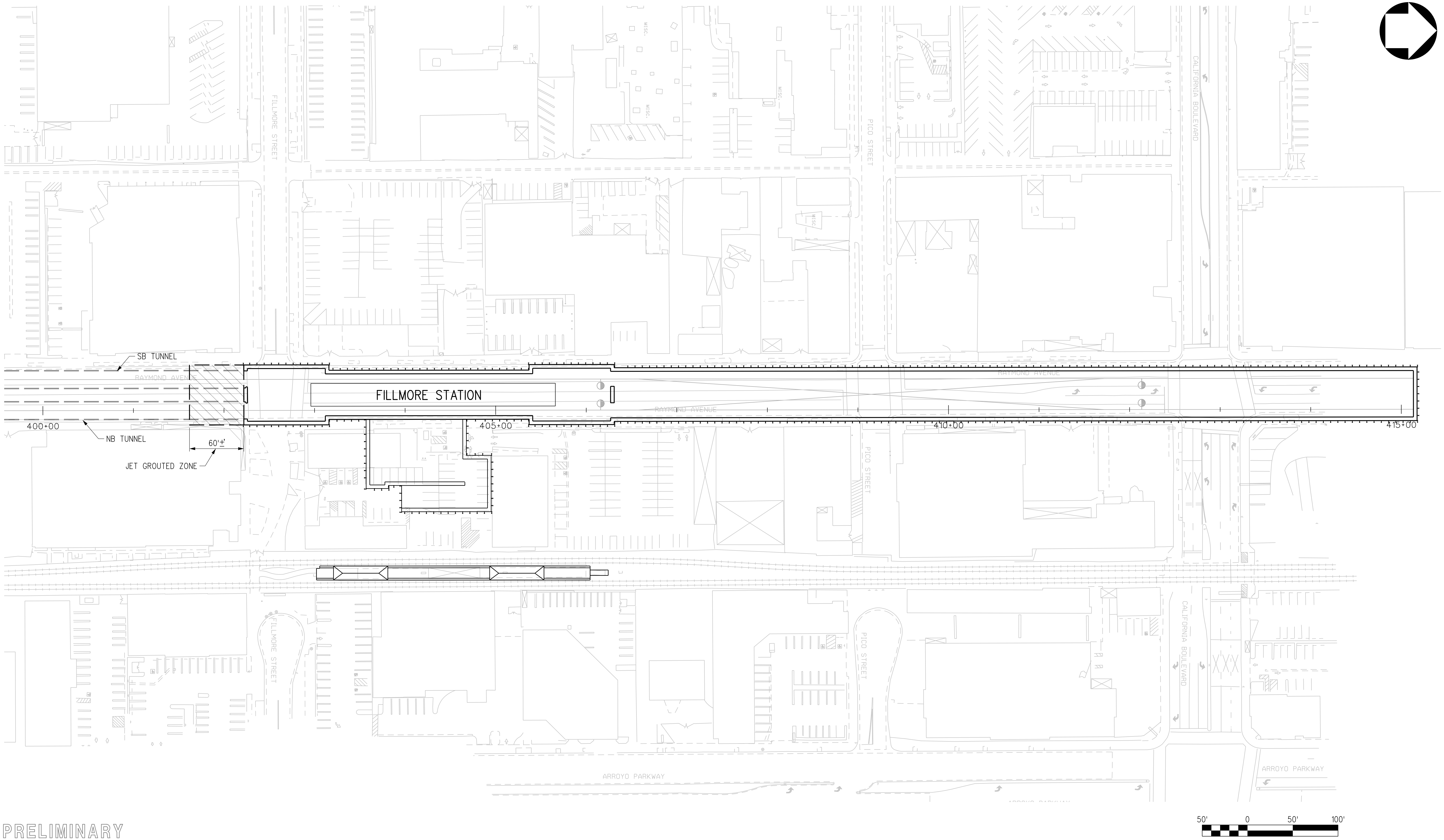

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA.

REV	DATE	BY	APP	REG NO	EXPIRES	SEAL HOLDER	DESCRIPTION

DESIGNED BY M. HARIHARAN
DRAWN BY W. OSTERMANN
CHECKED BY M. LAWRENCE
IN CHARGE
DATE 8/12/13

Metro
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY

SR 710 NORTH STUDY
ADVANCED CONCEPTUAL DESIGN
FILLMORE STATION
EXCAVATION SUPPORT PLAN

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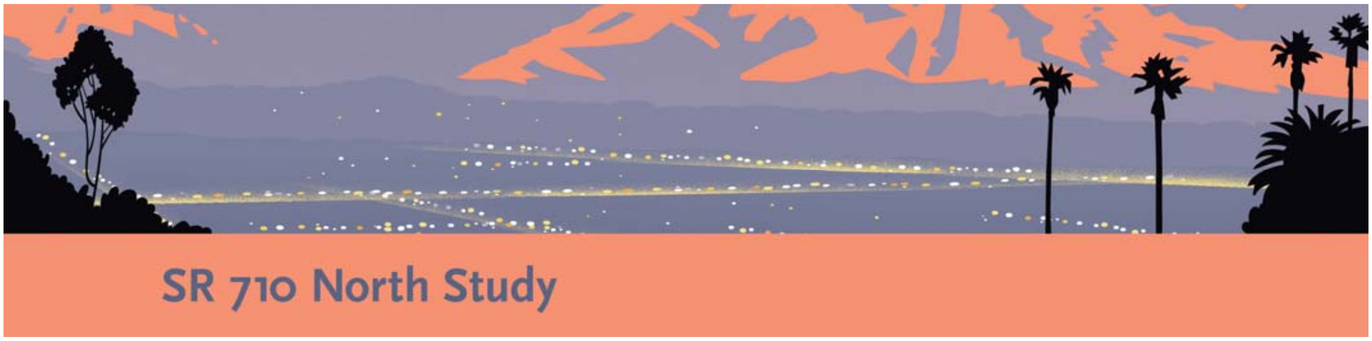
Appendix K
TM-9 Handling and Disposal of Excavated Materials



TECHNICAL MEMORANDUM 9

Handling and Disposal of Excavated Materials

PREPARED FOR:	Metro
COPY TO:	Caltrans
PREPARED BY:	Jacobs Associates/CH2M HILL
DATE:	August 22, 2014
PROJECT NUMBER:	428908

1 Introduction

1.1 Project Description

As part of the State Route (SR) 710 North Study, five alternatives are being evaluated as part of an ongoing environmental documentation process. The proposed alternatives include the No Build Alternative, the Transportation System Management/Transportation Demand Management (TSM/TDM) Alternative, the Bus Rapid Transit (BRT) Alternative, the Light Rail Transit (LRT) Alternative, and the Freeway Tunnel Alternative. The Freeway Tunnel and LRT Alternatives will involve tunnels for significant distances over their alignments.

- The LRT Alternative would include passenger rail operated along a dedicated guideway, similar to other Metro light rail lines. The LRT alignment is approximately 7.5 mi long, with 3 mi of aerial segments and 4.5 mi of bored tunnel segments. The LRT Alternative would begin at an aerial station on Mednik Avenue adjacent to the existing East Los Angeles Civic Center Station on the Metro Gold Line and continues north to end at an underground station beneath Raymond Avenue adjacent to the existing Fillmore Station on the Metro Gold Line. Two directional tunnels are proposed with tunnel diameters approximately 20 feet each. Seven stations would be located along the LRT alignment; of these, the Alhambra Station, the Huntington Station, the South Pasadena Station, and the Fillmore Station would be underground stations.
- The alignment for the Freeway Tunnel Alternative starts at the existing southern stub of SR 710 in Alhambra, just north of I-10, and connects to the existing northern stub of SR 710, south of the I-210/SR 134 interchange in Pasadena. The Freeway Tunnel Alternative has two design variations: a twin-bore tunnel and a single-bore tunnel. The twin-bore tunnel variation is approximately 6.3 mi long, with 4.2 mi of bored tunnel, 0.7 mi of cut-and-cover tunnel, and 1.4 mi of at-grade segments. The twin-bore tunnel variation would consist of two side-by-side tunnels (one northbound, one southbound), each tunnel of which would have two levels. Each tunnel would consist of two lanes of traffic on each level, traveling in one direction, for a total of four lanes in each tunnel. Each bored tunnel would have an outside diameter of approximately 60 feet. The single-bore tunnel design variation is similar in length and diameter; however, the single-bore tunnel variation would consist of one tunnel with two levels. The northbound traffic would traverse the upper level, and the southbound traffic would traverse the lower level.

1.2 Scope

This technical memorandum (TM) describes various aspects of handling and disposal of excavated materials for the FreewayTunnel and LRT Alternatives. The freeway and LRT bored tunnels are expected to be mined with tunnel boring machines (TBMs), and the excavated material generated from the tunneling operations would be removed from the tunnels at the portals. In addition to the bored tunnels, excavation of portals, cross passages, and LRT stations would also generate excavated material. The purpose of this TM is to:

- Evaluate anticipated properties (including bulking factors) for excavated material based on the geologic formations along the tunnel alignments,
- Estimate approximate quantities of excavated material that would be generated from the tunnel, portal, and LRT station excavations for each alternative,
- Discuss the anticipated excavated material conditions from TBM tunneling operations and the handling of excavated material at the work areas, and
- Determine the approximate volume of excavated material generated each work day and the associated number of truck trips per day at each of the portal areas during TBM tunnel excavation for each alternative. CH2M HILL has identified potential disposal sites for the excavated material that is generated from the excavation of the tunnels and related elements, and that is detailed in the environmental documentation.

This TM focuses on excavated material as a result of the excavation of the tunnel components listed above; disposal and/or treatment of water and groundwater as a result of excavation or construction activities is not discussed herein, but is discussed in the environmental documentation for this study.

1.3 Purpose

The purpose of this TM is to document preliminary design evaluations and concepts developed in support of the environmental documentation for the SR 710 North Study. These concepts also serve as a basis for preliminary construction cost estimate, developed for the tunnel sections.

The preliminary design concepts presented in this TM will be considered in subsequent environmental studies, and in many cases represent one feasible or likely option. Other options should be explored, and the concepts should be taken to a further level of refinement, in future phases of this study if either of the bored tunnel alternatives is carried further.

2 Anticipated Geologic Conditions

The anticipated geotechnical conditions along the tunnel alignments are indicated on the geologic profiles along the proposed alignments for the alternatives provided in Attachment A. The geologic units and their properties were evaluated based on data contained in the preliminary geotechnical report and the geologic profiles prepared by CH2M HILL (2014). The geologic units anticipated to be encountered during the excavation of the alternatives include Alluvium, Fernando Formation, Puente Formation, Topanga Formation, and Basement Complex Rocks.

2.1 Alluvium

Young and Old Quaternary alluvial materials (Alluvium) are encountered within the study area. The Alluvium, which overlies the bedrock, ranges approximately from 0 to 280 feet thick along the freeway alignment and 0 to 300 feet thick along the LRT alignment. The alluvial materials consist of interbedded lenses and/or discontinuous layers of fine-grained soil (clay and silt) and coarse grained materials (sand and gravel) that include a wide range of soil types. Cobble-size rocks are common locally within the Alluvium, and some boulders may be scattered locally throughout the unit. The alluvial soils generally increase in strength with depth. The consistency of the fine-grained soil encountered in the borings ranged from soft to hard; while the density of the coarse-grained materials encountered ranged from loose to very dense. Typical total in-situ unit weights for the Alluvium range from 110 to 150 pcf.

2.2 Fernando Formation

Fernando Formation consists primarily of low-strength, dark gray to black, massive (unbedded), marine claystone and siltstone. The rock mass is slightly to very slightly fractured, moderately to slightly weathered, extremely weak to very weak, and very soft. Typical total in-situ unit weights for the Fernando Formation range from 110 to 140 pcf.

2.3 Puente Formation

Puente Formation is comprised predominantly of sandstone, shale, diatomaceous siltstone/shale, and siltstone members. The sandstone member consists predominantly of thin to thick bedded fine-grained, very weak to strong sandstone and silty sandstone with scattered laminations to thick interbeds of siltstone and shale. Individual beds and intervals of these rocks are friable, weakly cemented, and susceptible to softening in the presence of water, but other beds are strongly cemented. The shale member consists predominantly of thinly bedded to laminated and fissile shales with thin interbeds to laminations of fine-grained sandstone and siltstones. The diatomaceous siltstone/shale member is represented by thin-bedded to laminated diatomaceous siltstones. Finally, the siltstone member generally consists of thin-bedded to laminated, extremely weak to medium strong siltstones with medium to thick interbeds to laminations of fine-grained sandstone. Typical total in-situ unit weights for the Puente Formation range from 110 to 160 pcf.

2.4 Topanga Formation

Topanga Formation includes a lower siltstone member, a middle sandstone member, and an upper conglomerate/breccia member.

- The siltstone unit consists of thinly bedded to laminated and fissile siltstones and shales, with fine- to coarse-grained sandstone interbeds. At the anticipated depth of the Freeway tunnels, the Topanga Formation might consist of well-bedded, slightly weathered, extremely weak to medium strong, and slightly to moderately fractured rock.
- The sandstone unit consists of well-bedded, medium- to coarse-grained sandstone with thin interbeds and laminations of fine-grained sandstone, siltstone, and/or shale with some conglomerate beds. The sandstone unit at the anticipated depth of the Freeway Tunnels is slightly weathered to fresh, extremely weak to strong, friable to low strength, and unfractured to slightly fractured. Individual beds and intervals of these rocks are friable, weakly cemented, and susceptible to softening in the presence of water.
- The conglomerate/breccia member generally consists of hard, well-rounded to subangular rock fragments derived from the basement complex of the San Gabriel and Verdugo mountains. Rock fragments of the Topanga Formation are commonly within an uncemented, friable, sandy matrix that allows the hard fragments to be broken out of the matrix with little difficulty. Individual beds and intervals of these rocks are friable, weakly cemented, and susceptible to softening in the presence of water.

The rocks of the Topanga Formation tend to be coarser-grained north of the Raymond fault. Typical total in-situ unit weights for the Topanga Formation range from 110 to 160 pcf.

2.5 Basement Complex Rocks

Basement Complex Rocks comprise a wide suite of lithologies, including diorite, monzonite, quartz diorite, quartz monzonite, and gneissic diorite. Regardless of the variable lithologies, these rocks have similar engineering properties. The Basement Complex Rocks expected to be encountered along the Freeway Tunnel alignment are highly fractured. The fracture density is commonly greater than 10 fractures per foot, and the rock quality designation (RQDs) are generally zero and rarely greater than 10 percent, indicating very poor quality rock. Basement Complex Rocks are not anticipated to be encountered along the LRT tunnel (refer to Attachment A). Typical total in-situ unit weights for the Basement Complex Rocks range from 100 to 160 pcf

2.6 Bulking Factors

Excavated material is typically measured in “bulked” volume because the volume of the material once excavated is larger than the volume of the same amount of material when in place and undisturbed. The amount of the volume increase is determined by a bulking factor. Bulking factors have been estimated for each geologic unit using published references for excavation volumes, including Toll (1993) and Caterpillar Performance Handbook (1996), as well as engineering judgment based on prior experience with similar materials. The unit weights and bulking factors used in this TM are summarized in Table 1. The bulking factors may vary based on the method of excavation, but it is expected that they would be similar to, and not much larger than, those listed in Table 1 given the information currently available. The bulking factors should be re-evaluated when more geotechnical information is available.

TABLE 1
Formation Unit Weights and Bulking Factors

Geologic Unit	Range of In Situ Unit Weights (pcf)	Assumed In Situ Unit Weight (pcf)	Bulking Factor Range	Assumed Bulking Factor
Alluvium	110 - 150	125	1.05 – 1.4	1.3
Fernando Formation	110 - 140	136	1.5 – 1.6	1.6
Puente Formation	110 - 160	134	1.4 – 1.65	1.6
Topanga Formation	110 - 160	134	1.4 – 1.65	1.6
Basement Rock	100 - 160	158	1.5 – 1.7	1.6

3 Tunnel Alternative Description

3.1 Freeway Tunnel Alternative

There are several elements of the Freeway Tunnel Alternative that would generate excavated material in completing the required excavations. The following sections describe each element and the assumptions made in estimating the volume of excavated material generated for this alternative. The assumtions presented herein is based on the current vertical and hotizontal tunnel alignment, and is subject to change as the alignments are optimized in future stages of the study.

3.1.1 Bored Tunnels. The Freeway Tunnel Alternative has two variations - either a single- or twin-bore tunnel both 22,340 feet in length; the outside diameter of the final lining is expected to be be 58.5 feet. The excavated diameter of the tunnel would be slightly larger to account for overcut and TBM shield thickness, making the diameter of the excavated tunnel approximately 60 feet, which would be used in estimating excavated material volumes (Jacobs Associates, 2014c).

Each tunnel bore is expected to be driven with two TBMs based on the understanding of the current schedule demands, with one starting at the south portal and one at the north portal and meeting in the middle for each bore. This would require four TBMs total for the twin-bore alternative, and two TBMs total for the single-bore alternative. For the purposes of this TM, it is assumed that each TBM would mine half of each bore–the excavated material from the north reach would be generated at the north portal and the excavated material from the south reach would be generated at the south portal.

A generalized profile of the Freeway Tunnel Alternative provided by CH2M HILL (2014) was used to evaluate the percentage of each geologic unit that would be present along the freeway tunnel at the approximate tunnel depth (refer to Attachment A). The approximate distribution of the various geologic units is summarized in Table 2. The lengths shown are per tunnel bore, and would be doubled for the twin-bore option.

TABLE 2
Summary of Geologic Unit Expected in the Freeway Tunnel Alternative (per Tunnel Bore)

Geologic Unit	North Reach Length (ft)	%	South Reach Length (ft)	%
Alluvium	4,620	42%	0	0%
Fernando Formation	0	0%	3,650	33%
Puente Formation	0	0%	3,550	32%
Topanga Formation	4,950	44%	3,970	35%
Basement Rock	1,600	14%	0	0%
Total	11,170	100%	11,170	100%

3.1.2 Cross Passages. In addition to the bored tunnels of the Freeway Tunnel Alternative, six pairs of emergency vehicle cross passages (twelve total) would be included along the twin-bore variation to connect the two tunnels, which are located approximately one tunnel diameter apart (i.e. approximately 64 feet). These cross passages would be roughly circular in shape and approximately 29 feet in diameter. They would be excavated using the Sequential Excavation Method (SEM) after the bored tunnels are excavated. The excavated material generated would be removed from the nearest portal. Therefore, it is assumed that excavated material from three pairs of cross passages would be removed from the north portal and the excavated material from the other three at the south portal. Table 3 shows a summary of the number of cross passages to be excavated through each geologic unit.

3.1.3 Construction Portals. Construction portals at the north and south ends of the tunneled portion of the Freeway Tunnel Alternative are expected to be excavated prior to the initiation of tunneling operations. These portals would be used to launch the TBM(s) and support construction activities. The roadway ramps down from the ground surface within the portal to gain cover for launching the TBM. The north portal is expected to be approximately 100 feet deep, measured at the headwall (where the tunnel starts), 240 feet wide, and 500 feet long. The south portal is expected to be approximately 130 feet deep, measured at the headwall, 230 feet wide and 500 feet long. The portal excavations gradually increase in depth from the ground surface to the headwall. The portals for the single-bore alternative are similar in shape, but the width is smaller – approximately 110 feet less. Although the portal excavations are longer than 500 feet, the excavated material volume beyond this length of the excavation will be estimated by CH2M HILL in their assessments for the design of the permanent works.

The north portal would be excavated entirely in alluvial soils and the south portal would be excavated in a mixture of alluvial soils and the Puente Formation. Refer to *Preliminary Design Concepts for Freeway Portal Excavation Support Systems* (Jacobs Associates, 2014a) for additional information on the portals for the Freeway Tunnel Alternative.

TABLE 3
Freeway Cross Passage Geologic Units

Geologic Unit	No. of Cross Passages	
	North Reach	South Reach
Alluvium	2	0
Fernando Formation	0	2
Puente Formation	0	2
Topanga Formation	4	2
Basement Rock	0	0

3.2 LRT Alternative

There are several elements of the underground portions of the LRT Alternative that would generate excavated material in completing the required excavations. The following sections describe each element and the assumptions made in estimating the volume of excavated material generated for this alternative. The assumtions presented herein is based on the current vertical and hotizontal tunnel alignment, and is subject to change as the alignments are optimized in future stages of the study.

3.2.1 Bored Tunnels. The LRT Alternative includes approximately 21,180 feet of twin-bore tunnel that are exoected to be excavated with two TBMs. The alternative also includes four underground stations which would be excavated using cut and cover techniques in advance of the TBM arrival at each location. Along the alignment, the TBMs would break into the south end of each station, be walked through the station excavation, and recommence tunneling at the north end of the station.

The outside diameter of the final lining is about 20.5 feet and the excavated diameter of the tunnel would be slightly larger to account for the cutterhead and shield thickness of the TBM. It is assumed that the diameter of the tunnel excavation would be about 14 inches greater than the diameter of the lining, making the diameter of the excavated area just over 21.5 feet, which would be assumed for the excavated material volume estimates (Jacobs Associates, 2014c). Two TBMs would be used to excavate this alternative; one for each tunnel bore. They would be launched from a portal on the south end of the alignment and terminate at a station at the north end. Therefore, all excavated material would be removed from the south portal, which would support all the mining activities.

A generalized profile of the LRT Alternative provided by CH2M HILL (2014) was used to evaluate the percentages of each geologic unit that would be encountered along the bored tunnel at the approximate tunnel depth. Because of the shallow depth of the LRT tunnels, the geology along this alignment would vary between Alluvium and the weak rock units of the Fernando, Puente, and Topanga formations. Table 4 shows the approximate distribution of geologic units along the alignment.

3.2.2 Cross Passages. In addition to the bored tunnels, twenty-six pedestrian cross passages would be excavated along the LRT tunnel alignment to connect the two tunnels, which are located approximately one tunnel diameter apart. These cross passages, which are oval- shaped and approximately 12 feet wide by 14 feet high would be excavated using the Sequential Excavation Method (SEM) and would be used for emergency egress only. Table 5 shows a summary of the number of cross passages to be excavated through each geologic unit.

3.2.3 Construction Portal. A construction portal at the south end of the tunneled portion of the LRT alternative would be excavated in advance of tunneling operations. This portal would be used to launch the two TBMs and support construction activities. The portal is expected to be approximately 50 feet deep, measured at the headwall, approximately 70 feet wide, and 350 feet long. The portal is deepest at the headwall and becomes shallower with distance away from the headwall as the base of the portal slopes upwards to meet the existing grade. The portal would be excavated entirely in alluvial soils. Refer to *Preliminary Design Concepts for LRT Portal and Station Excavation Support Systems* (Jacobs Associates, 2014b) for additional information on the portal for the LRT alternative.

TABLE 4
Summary of Geologic Units Expected Along LRT Tunnels (per Tunnel Bore)

Geologic Unit	Length per Bore	%
Alluvium	11,830	56%
Fernando Formation	700	3%
Puente Formation	3,100	15%
Topanga Formation	5,550	26%
Basement Rock	0	0%
Total	21,180	100%

TABLE 5
LRT Cross Passage Geologic Units

Geologic Unit	No. of Cross Passages
Alluvium	11
Fernando Formation	1
Puente Formation	6
Topanga Formation	8
Basement Rock	0

3.2.4 Stations. Four underground cut-and-cover stations are included in the LRT Alternative. From south to north, these are the Alhambra Station, Huntington Station, South Pasadena Station, and Fillmore Station, where the alignment terminates.

The Alhambra Station excavation is approximately 410 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet. The majority of the station excavation would be in Alluvium, with a small percentage in the Fernando Formation towards the bottom of the excavation.

The Huntington Station excavation is approximately 825 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet. The long length of this station allows space for a track crossover. The majority of the excavation would be in Alluvium, with the remainder in the Topanga Formation.

The South Pasadena Station is approximately 410 feet long and 60 feet wide in plan. The total excavation depth to the bottom would be approximately 80 feet. The station excavation would be entirely in Alluvium.

The Fillmore Station is approximately 1300 feet long and 60 feet wide in plan. The total excavation depth to bottom is approximately 80 feet. The long length of this station allows for a track crossover and tail tracks. It is assumed that the entire excavation would be performed by cut-and-cover methods, and that the TBMs would be retrieved from this excavation. The excavation would be entirely in Alluvium.

The excavated material generated from each station and crossover excavation would be removed from the construction area at that station, not from the portal at the south end of the alignment. Refer to *Preliminary Design Concepts for the LRT Portal and Station Excavation Support Systems* (Jacobs Associates, 2014b) for additional information on the underground stations for the LRT Alternative.

4 Excavated Material Volumes and Weights

The estimated volumes and weights of the excavated material to be disposed are discussed in the following sections. The results are based on the assumed excavated dimensions and anticipated geologic conditions described in the preceding sections. The bulking factors are used to estimate the volume of bulked material after calculating the bank, or in-situ, volumes.

4.1 Freeway Tunnel Alternative

Estimated excavated material quantities for the Freeway Tunnel Alternative are summarized in Table 6 below. These values include the in-situ and bulk volumes as well as the estimated weight of excavated material to be removed from each portal, separated by excavation type: bored tunnels, cross passages, and portal excavations.

TABLE 6
Summary of Estimated Freeway Tunnel Alternative Excavated Material Quantities

Excavation	In-Situ Excavated Volume (yd^3)		Bulked Excavated Volume (yd^3)		Estimated Weight (tons)	
	Twin-Bore	Single-Bore	Twin-Bore	Single-Bore	Twin-Bore	Single-Bore
Bored Tunnel – North Portal	2,339,000	1,170,000	3,453,000	1,726,000	4,223,000	2,112,000
Bored Tunnel – South Portal	2,339,000	1,170,000	3,743,000	1,872,000	4,253,000	2,126,000
Cross Passages – North Portal	11,000	NA	17,000	NA	20,000	NA
Cross Passages – South Portal	11,000	NA	18,000	NA	20,000	NA
North Portal*	384,000	164,000	500,000	213,000	648,000	277,000
South Portal*	503,000	242,000	701,000	337,000	868,000	417,000

*Note these values exclude excavation for the freeway approach and cut and cover tunnels, only the launch portal for the TBMs as described in the text.

4.2 LRT Alternative

Estimated excavated material quantities for the LRT Alternative are summarized in Tables 7 and 8 below. These values include the in-situ and bulk volumes as well as the estimated weight of excavated material to be removed, separated by excavation type and excavated material removal location: bored tunnels, cross passages, and south portal excavation (Table 7), and cut-and-cover station excavations (Table 8). The excavated material from bored tunnels and cross passages would be removed from the south portal, while the excavated material from each cut-and-cover station excavation would be removed at that location.

TABLE 7
Summary of Estimated LRT Alternative Excavated Material Quantities to be Removed at LRT Portal

Excavation	In-Situ Excavated Volume (yd^3)	Bulked Excavated Volume (yd^3)	Estimated Weight (tons)
Bored Tunnel	578,000	829,000	1,008,000
Cross Passages	3,000	4,000	5,000
Portal	35,000	46,000	59,000

TABLE 8
Summary of Estimated LRT Alternative Excavated Material Quantities to be Removed at Stations

Excavation	In-Situ Excavated Volume (yd^3)	Bulked Excavated Volume (yd^3)	Estimated Weight (tons)
Alhambra Station	67,000	89,000	114,000
Huntington Station	137,000	190,000	236,000
South Pasadena Station	73,000	95,000	123,000
Fillmore Station	219,000	284,000	369,000

5 Excavated Material Handling

As seen in the tables above, the majority of the excavated material generated for the two alternatives would be at the portals as a result of the bored tunnel excavation by TBM. The remainder of this TM focuses specifically on the excavated material generated from TBM tunnel excavation.

5.1 Excavated Material Consistency

The contractor's means and methods for the tunnel excavation would influence the consistency of the excavated material. It is assumed that the tunneling method for the bored tunnels is by a pressurized-face TBM. There are two basic types of pressurized-face TBMs which could be used for this project—Earth Pressure Balance (EPB) TBMs and Slurry TBMs. The principal difference between these two types of machines is in the method of stabilizing the tunnel face during excavation, and the principle difference with respect to excavated material generation is discussed herein.

In the EPB method, excavated material is excavated and conveyed using a screw conveyor and typically discharged onto a belt conveyor, whereby it is transported out of the tunnel and to the surface by means of vertical conveyors or bins. In a Slurry TBM, excavated material is circulated to the surface in a slurry suspension via pipes, where it is separated from the slurry by mechanical separators and ends up drier compared to EPB excavated material.

It should be noted that EPB excavated material is often heavily water laden, and therefore may require drying at the portal areas prior to being hauled offsite. Drying the material would require space for stockpiling and sufficient area to spread the material out so that it can be dried in a reasonable amount of time. Free water draining from the stockpiles must be collected and treated, usually with the other tunnel water, prior to disposal in accordance with regulatory and permit requirements. Figure 1 shows photos of EBP excavated material at the TBM and at a stockpile site.

Excavated material being discharged from EPB TBM

Excavated material consistency at surface after rehandling

FIGURE 1 EARTH PRESSURE BALANCE TBM EXCAVATED MATERIAL

5.2 Excavated Material Additives

Soil conditioning is an important aspect of soft ground tunnel construction, and the main objective is to improve TBM performance and to modify the ground to provide better control of the tunneling operation. The addition of suitable additives or conditioning agents in the form of foams and/or polymers may be introduced during the tunneling process to improve EPB TBM performance in several ways, as summarized below:

- Improved stability of tunnel face and better control of surface settlement.
- Improved flow of excavated material through the cutterhead.
- Reduced wear of cutterhead face plate and tools, and all parts of the excavated material removal system.
- Reduced torque and cutterhead power requirements.
- Reduced friction and heat buildup along the TBM shield.
- Improved handling of excavated material because it is formed into a suitably plastic-like mass.

It is far more cost effective to treat the soil being excavated with appropriate conditioners than to resort to time-consuming changes to equipment during tunnel excavation. In general, soil conditioning is required for EPB TBMs in all ground types, and excavated material is generally conditioned to a toothpaste-like consistency with high water and air content.

For a Slurry TBM, bentonite is mixed with water to create a slurry that is used to support the tunnel face and also to transport the excavated material to the ground surface. The excavated material is mixed with the slurry in the chamber and suspended in the slurry at a consistency that allows removal from the chamber by pumping. The pressurized slurry exerts a hydraulic pressure at the tunnel face and forms a mud cake to stabilize and seal the tunnel face. This is similar to the use of drilling muds to stabilize deep boreholes. The use of bentonite may not be required in formations with enough natural clay to develop a slurry with the proper specific gravity.

After the ground is excavated, the material is transported to the surface in the slurry through a system of pipes to a separation plant. A separation plant includes several mechanical components such as shaking sieves, hydrocyclones, and centrifuges to separate the excavated material from the slurry. After separating, the spoil from the reconditioned slurry is pumped recirculated back to the tunnel heading. Figure 2 shows excavated material from a Slurry TBM excavation.

Slurry pumped from tunnel entering separation plant **Consistency after being separated from slurry**

FIGURE 2 EXCAVATED MATERIAL FROM SLURRY TBM

The additives used for soil conditioning in EPB and Slurry TBM operations are normally specified to be non-toxic and biodegradable. When used in the concentrations recommended by the manufacturer they have been tested and shown in practice to have little impact on the customary routine of excavated soil disposal. There are natural product lines that are widely accepted and used in the tunneling industry, and therefore the soil conditioning process is not expected to introduce contamination into the excavated material; however, the chemistry of the excavated material should be taken into consideration when it is known which additives would be used during excavation.

5.3 Excavated Material Storage

Sufficient area at each portal location would be needed to stockpile material as it is excavated. Stockpiling is necessary so that water-laden excavated material can dry and also so that excavation is not slowed down because of a lack of space to place or store excavated material. While each project is different and it is ultimately up to the contractor, it is typical to provide a minimum storage capacity to stockpile a minimum of the volume of excavated material expected to be generated over a three day period of mining. More about excavated material generation rates from the bored tunnels is discussed in the following section.

6 Excavated Material Production Rates

The average amount of material generated per day is a function of the advance rate of the TBM, which depends on many factors. The total number of trucks required to complete daily trips from each of the mining areas is dependent upon the excavated material generated and the distance to the disposal location. An estimate of the number of trucks per day on individual roadways and at specific intersections would not be possible until the disposal locations to be used are finalized; however an estimate of the number of truck trips per day per portal has been performed, refer to Table 9. The number of trips is given for both average and peak advance rates; however, it should be noted that average advance rates are expected and that peak truck movements may only be experienced for short periods of time. The advance rates have been estimated, and are subject to change based on contractor's means and methods.

The following assumptions have been made to perform the trucking estimates:

- For the twin-bore alternatives, both bores would be excavated simultaneously.
- The excavation operations occur 24 hours per day.
- Only the bulked excavated material volumes generated from the TBM-driven tunnel component of the project are considered.
- Truck capacity is 16 cubic yards (bulked volume), although weight could ultimately dictate trucking requirements.
- The number of trucks estimated is for excavated material haulage only; however, there would be other trucks to the site for deliveries that are not accounted for below.

TABLE 9
Excavated Material Production Rate Summary

	Freeway		LRT
	Twin-Bore	Single Bore	
Average Advance Rate	27 ft/day/bore		46 ft/day/bore
Peak Advance Rate	40 ft/day/bore		69 ft/day/bore
Average Trips / Day / Portal	570	285	110
Peak Trips / Day / Portal	850	430	170

7 Excavated Material Disposal

CH2M HILL has indicated that potential disposal sites within 20 miles of the construction staging areas have been identified for the excavated material generated for these alternatives. These sites are detailed in the environmental documentation, and they are expected to have the capacity needed for any of these tunneled alternatives. However, since the disposal of excavated material is a significant aspect of this project due to the volume and rate of excavated material generation, the availability of these sites, hours of operation, and trucking routs are all factors that need to be considered in planning and selecting the final disposal site(s). Disposal sites could consist of landfills, quarries that need backfilling or site restoration, and also excavated material re-use from potential local projects that could benefit from having fill.

In addition to hauling excavated material by trucks from the portals, additional transportation options should be considered. For example, at the south portal, there are existing rail tracks that run alongside of Mission Road. Removal of excavated material at the south portal by rail should also be considered. Additionally, re-use of some of the excavated material should be investigated in future stages of this study. While the volument of material to be handled and disposed of may be significant, current methods and techniques are available to ensure it is performed properly and safely.

8 Limitations

The information and recommendations presented in this TM are preliminary interpretations based on limited geotechnical data and the tunnel alignments that were available when the TM was prepared. It is also assumed that the alignments of the tunneled portions of the alternatives would be optimized as the study progresses. The findings and recommendations presented in this TM should be reassessed when additional data becomes available.

9 References

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10 Revision Log

Revision 0	Ocotber 16, 2013	Internal Review
Revision 1	November 8, 2013	Metro/Caltrans Review
Revision 2	May 27, 2014	Metro/Caltrans Review
Revision 3	August 22, 2014	Incorporation into Tunnel Evaluation Report

Attachment A

Geologic Profiles

NOTES:

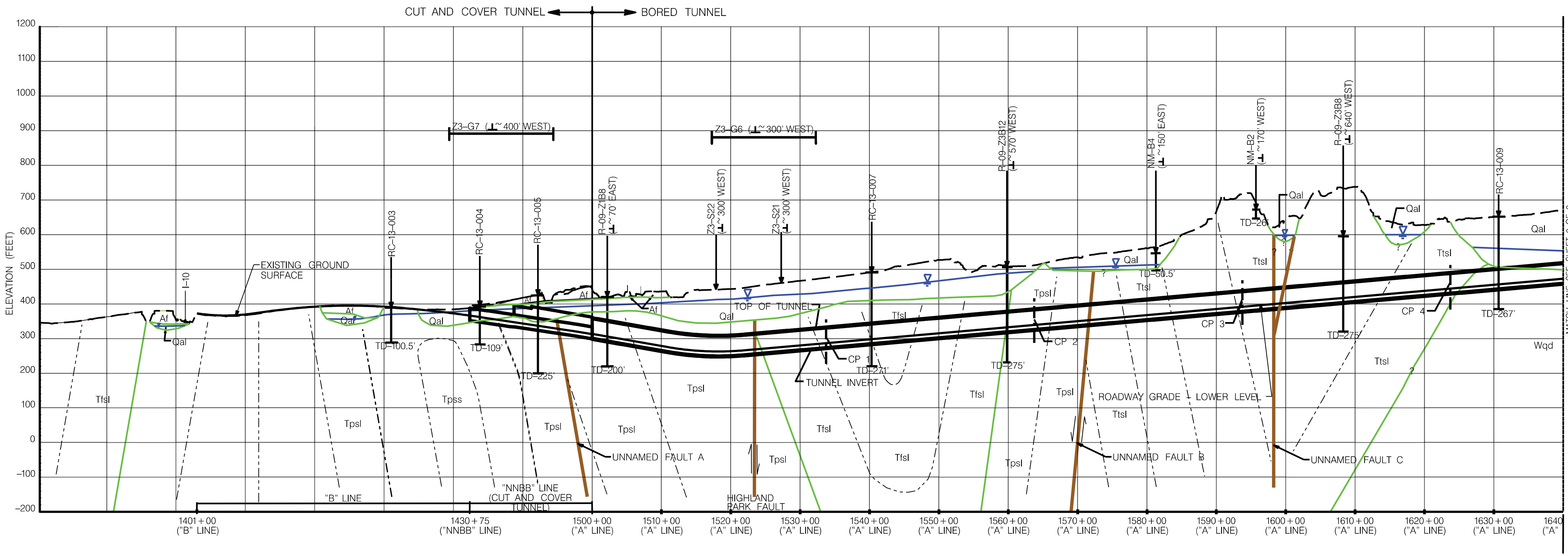
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT REPORT PREPARED FOR THE FREEWAY TUNNEL ALTERNATIVE DRAFT PRELIMINARY PROJECT PLANS BY CH2M HILL, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

A-13-020 TD-149.8' GEOTECHNICAL BOREHOLE WITH TOTAL DEPTH AND PROJECTION:
A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
R-09-Z1B8 – CH2M HILL, 2010
NM-B3 – NINYO AND MOORE, 1999
EMI-3 – EARTH MECHANICS INC, 2006
ES-2 – CALTRANS, 1974

CP – CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 1 OF 2

NOTES:

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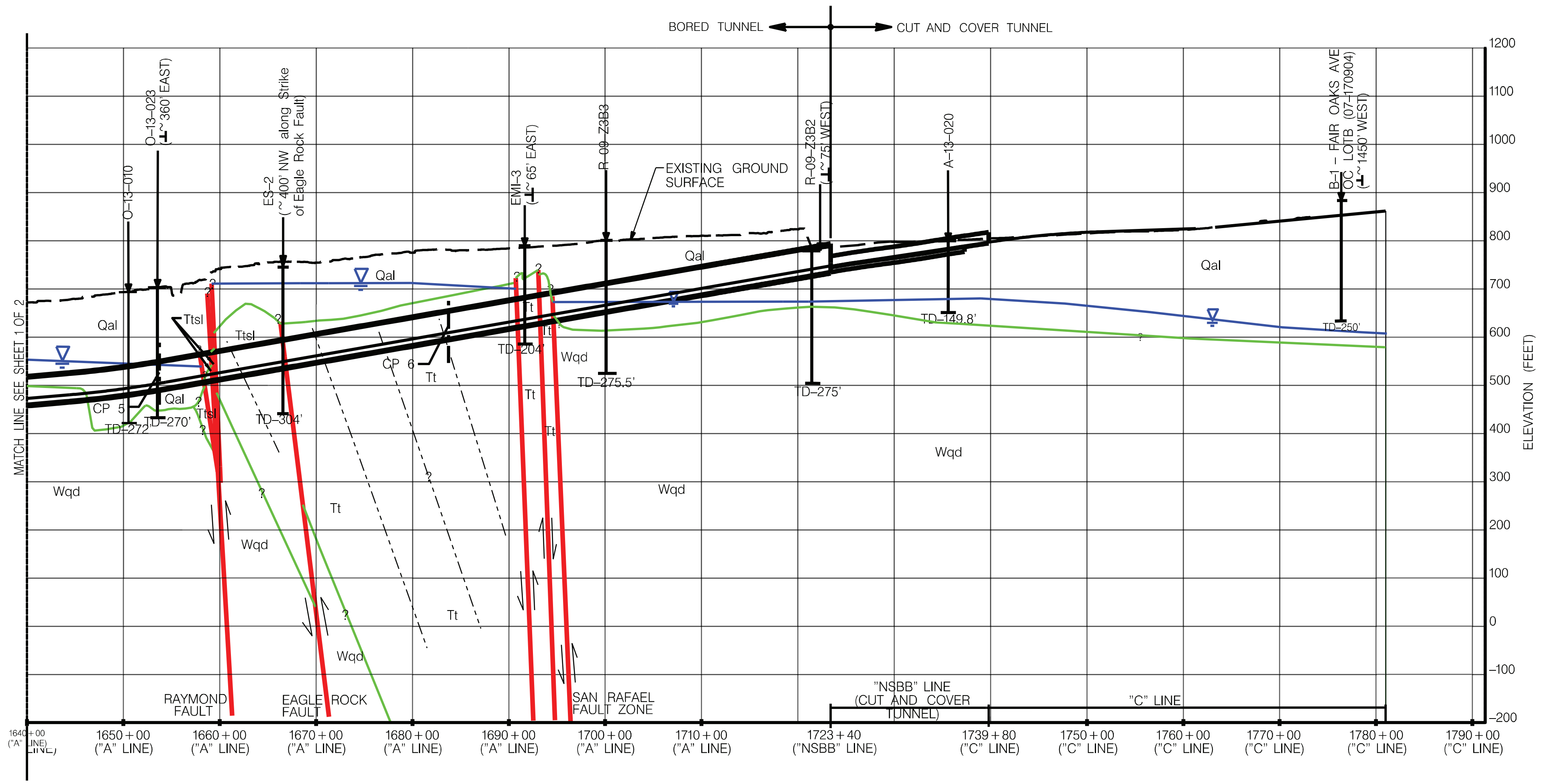
4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Freeway Tunnel Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

A-13-020 TD-149.8' GEOTECHNICAL BORHOLE WITH TOTAL DEPTH AND PROJECTION:
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ES-2 – CALTRANS, 1974

CP – CROSS PASSAGE

FREEWAY GEOLOGIC PROFILE
SHEET 2 OF 2

NOTES:

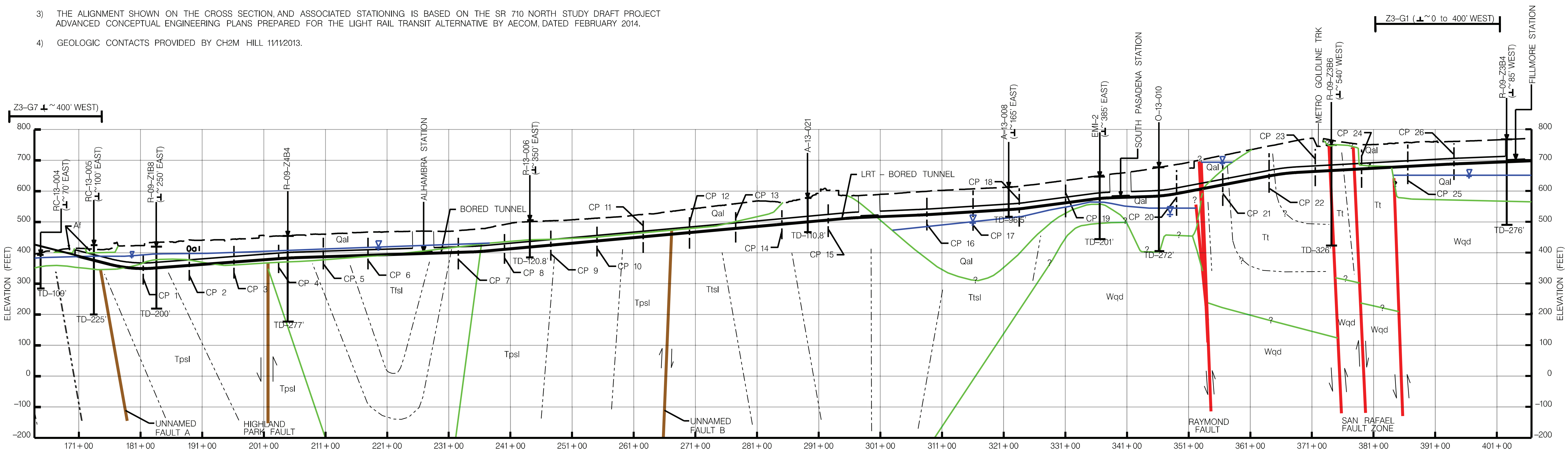
1) EXISTING PROFILE BASED ON TOPOGRAPHIC SURVEY BY WARNER ENGINEERING AND SURVEYING INC. FOR THE SR 710 NORTH STUDY. MAPPING DATUMS ARE NAD 1983 AND NAVD 1988

2) THE GEOLOGY INTERPRETED ON THIS CROSS SECTION IS APPROXIMATED, BASED ON THE GEOLOGIC SOURCES REFERENCED IN THE TEXT OF THIS REPORT AND A LIMITED NUMBER OF WIDELY SPACED BORINGS. SIGNIFICANT, ADDITIONAL DETAILED GEOLOGIC INVESTIGATION WILL BE REQUIRED TO ADEQUATELY CHARACTERIZE THE GEOLOGIC CONDITIONS ALONG THE ALIGNMENT.

3) THE ALIGNMENT SHOWN ON THE CROSS SECTION, AND ASSOCIATED STATIONING IS BASED ON THE SR 710 NORTH STUDY DRAFT PROJECT ADVANCED CONCEPTUAL ENGINEERING PLANS PREPARED FOR THE LIGHT RAIL TRANSIT ALTERNATIVE BY AECOM, DATED FEBRUARY 2014.

4) GEOLOGIC CONTACTS PROVIDED BY CH2M HILL 11/11/2013.

Geologic Cross Section
SR 710 North Study – Light Rail Transit Alternative



LEGEND

UNITS

Af	ARTIFICIAL FILL
Qal	ALLUVIAL SOIL
Tfcg	FERNANDO FORMATION, CONGLOMERATE MEMBER
Tfsl	FERNANDO FORMATION, SILTSTONE MEMBER
Tpsl	PUENTE FORMATION, SILTSTONE MEMBER
Tpss	PUENTE FORMATION, SANDSTONE MEMBER
Tt	TOPANGA FORMATION, UNDIFFERENTIATED
Ttss	TOPANGA FORMATION, SANDSTONE MEMBER
Ttcg	TOPANGA FORMATION, CONGLOMERATE MEMBER
Ttsl	TOPANGA FORMATION, SILTSTONE MEMBER
Wqd	WILSON QUARTZ DIORITE

SYMBOLS

ALL LOCATIONS ARE APPROXIMATE. QUERIES INDICATE UNCERTAINTY.

GEOLOGIC CONTACT

INACTIVE FAULT

ACTIVE OR POTENTIALLY ACTIVE FAULT

INTRAFORMATIONAL CONTACT

GENERALIZED BEDDING

ESTIMATED TOP OF GROUNDWATER TABLE

Z3-G7 SEISMIC LINE (CH2M HILL, 2010), WITH PROJECTION

A-13-020 TD-149.8' GEOTECHNICAL BORHOLE WITH TOTAL DEPTH AND PROJECTION:
- A, R, RC, O-13-001 – CH2M HILL, THIS STUDY
- R-09-Z1B8 – CH2M HILL, 2010
- NM-B3 – NINYO AND MOORE, 1999
- EMI-3 – EARTH MECHANICS INC, 2006
- ES-2 – CALTRANS, 1974

CP – CROSS PASSAGE

LRT GEOLOGIC PROFILE