PRELIMINARY DESIGN OF THE CALDECOTT 4TH BORE

Bhaskar B. Thapa, Michael T. McRae, Johannes V. Greunen,
Jacobs Associates, San Francisco

2007 RETC Proceedings

ABSTRACT

This paper describes the preliminary design of the fourth bore of the Caldecott Tunnels along State Route 24 in Oakland, California. The paper focus is on two major aspects of the project: initial support system design and seismic design. The initial support system was designed based on the New Austrian Tunneling Method (NATM). NATM provides the required flexibility to accommodate the variable ground conditions in the weak, folded, and faulted rock along the alignment. The seismic design provides a final lining system that meets serviceability requirements for a 1,500 year seismic event with an estimated peak ground acceleration of 1.2g.

INTRODUCTION

Project Background

The existing Caldecott Tunnels consist of three bores along State Route 24 (SR 24) through the Berkeley Hills in Oakland, California. The California Department of Transportation (Caltrans) and the Contra Costa Transportation Authority (CCTA) propose to address congestion on SR 24 near the existing Caldecott Tunnels by constructing a fourth bore that will provide two additional lanes. The length of the proposed fourth bore is 1,036 meters (3,399 feet). The project will include short sections of cut-and cover tunnel at each portal; seven cross-passageway tunnels between the fourth bore and the existing third bore; and a new Operations and Control Building.

The fourth bore includes two 3.6-meter (12-foot) traffic lanes and two shoulder areas that are 3 meters and 0.6 meter (10 feet and 2 feet) wide. The horseshoe-shaped mined tunnel is 15 meters (50 feet) wide and 9.7 meters (32 feet) high. The tunnel includes a jet fan ventilation system, a wet standpipe fire protection system, and various operation and control systems including CCTV monitoring, heat and pollutant sensors, and traffic monitoring systems.

GEOLOGY

Major Geologic Formations and Structure

The geology of the alignment is characterized by northwest-striking, steeply-dipping, and locally overturned marine and non-marine sedimentary rocks of the Middle to Late Miocene age. The western end of the alignment traverses marine shale and sandstone of the Sobrante Formation. The Sobrante Formation includes the

First Shale, Portal Sandstone, and Shaly Sandstone geologic units as identified by Page (1950). The middle section of the alignment traverses chert, shale, and sandstone of the Claremont Formation. The Claremont Formation includes the Preliminary Chert, Second Sandstone, and Claremont Chert and Shale geologic units (Page , 1950). The eastern end of the alignment traverses non-marine claystone, siltstone, sandstone, and conglomerate of the Orinda Formation. Major formations and geologic units within these formations are shown Figure 1.

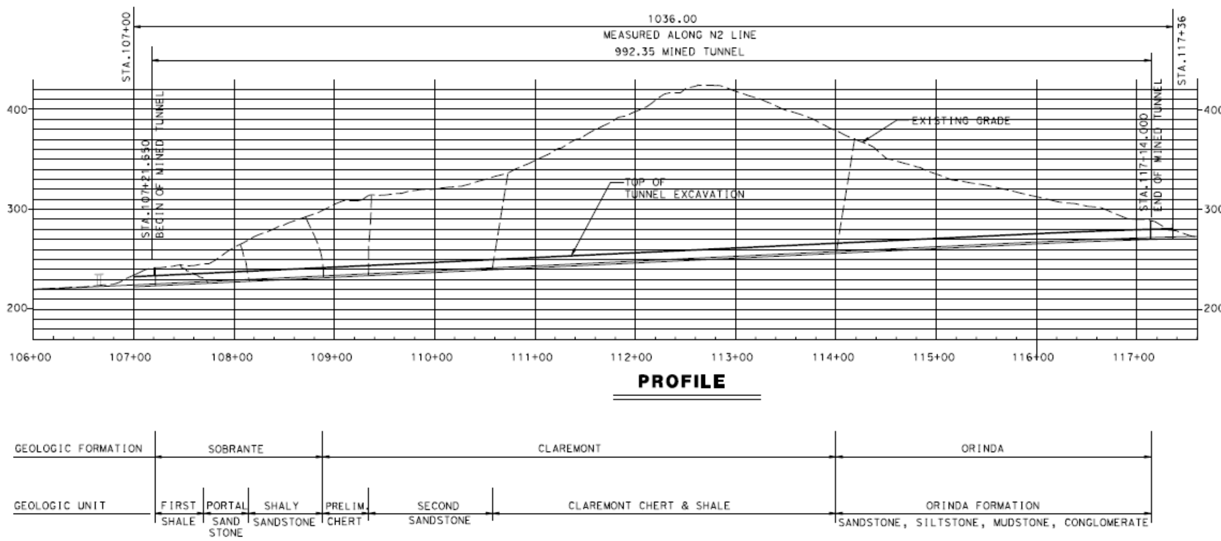

Figure 1. Geologic Formations and Geologic Units

The regional geological structure of the project area has been characterized as part of the western, locally overturned limb of a broad northwest-trending syncline, the axis of which lies east of the project area. The fourth bore alignment will encounter four major inactive faults, which occur at the contacts between geologic units. These faults strike northwesterly and perpendicular to the tunnel alignment. In addition to the major faults, many other weakness zones will be encountered away from the major faults, such as smaller-scale faults, shears, and crushed zones.

West of the fault contact between the Preliminary Chert and Shale and the Second Sandstone, the bedding encountered in the fourth bore generally dips predominantly toward the northeast. East of this fault contact, the bedding dips to the southwest. Several joint sets occur within each geologic unit, and random joints occur in almost any orientation in all geologic units. Intrusive sandstone dikes and hydrothermally altered diabase dikes occur most frequently in the Claremont Chert and Shale, but may also be encountered less frequently in other geologic units. The structure of the rock mass units along reaches of the alignment varies from being blocky in the best ground, down to a disintegrated or crushed condition in the poorest quality rock. Average values of unconfined compressive strengths varies from 9.6 MPa (1400 psi) to 48.5 MPa (7000 psi) in the various geologic units along the alignment Rock Mass Ratings (Bieniawski 1989) vary between 20 and 65 along the alignment.

Seismicity

The San Francisco Bay Region is considered one of the more seismically active regions of the world, based on its record of historical earthquakes and its position astride the tectonic boundary between the North American and Pacific plates. During the past 160 years, faults within this plate boundary zone have produced numerous small-magnitude ($M<6$), and more than a dozen moderate- to large-magnitude ($M>6$), earthquakes affecting the region. Major faults that comprise the 80-kilometer-wide plate boundary within the San Francisco Bay Region include the San Gregorio, San Andreas, Hayward, and Calaveras faults.

The active Hayward fault, located 1.4 kilometers (0.9 miles) west of the Caldecott Tunnel, is the closest regional fault to the project site. The southern segment of the Hayward fault produced the 1868 Haywards earthquake of estimated magnitude 6.8 that was accompanied by 30 to 35 kilometers (19 to 22 miles) of surface faulting. The most recent large earthquake on the northern segment of the Hayward fault is estimated to have occurred between 1640 and 1776. Presently, both segments of the Hayward fault exhibit tectonic fault creep at rates of up to 1 centimeter/year (0.4 inches/year).

GROUND CLASSIFICATION

The design and construction of the fourth bore is based on the sequential excavation method (SEM), also called the New Austrian Tunneling Method (NATM). The ground classification process was twofold: identification and characterization of Rock Mass Types (RMT) along the alignment having similar mechanical characteristics, and identification of ground classes based on similarity of anticipated ground behaviors of each RMT in response to excavation.

The identification of RMTs was based on the distribution of geological characteristics and relevant geotechnical parameters. The alignment was divided into RMTs based primarily on lithology, fracture density, discontinuity properties and unconfined compressive strength (UCS). Mechanical properties were determined for each of the RMTs along the alignment and ground behaviors were evaluated considering the identified boundary conditions.

The RMTs were then grouped into four ground classes based on the similarity of anticipated ground behaviors in response to excavation. An appropriate support category was then developed for each ground class. For example, Ground Class 1 comprises all RMTs along the alignment that require Support Category I. Ground Class 2 correlates to Support Category II, and so on. Individually, the support categories address sets of similar ground behaviors, and as a whole they address all anticipated ground behaviors along the alignment. The ground classes were the basis of design for the initial support categories.

Ground Classes

The actual ground classes along the alignment will be determined during construction based on probe drilling ahead of the lead drift, geologic mapping of the tunnel, and tunnel monitoring. The ground classes encompass a broad range of rock properties as shown in Table 1.

Ground Class	Rock Mass Description
1	Blocky rock masses with poor to good discontinuity conditions and weak to medium strong intact unconfined compressive strength
2	Very blocky to blocky/disturbed/seamy rock masses with poor to good discontinuity surface conditions and weak to medium strong intact rock unconfined compressive strength
3	Blocky/disturbed/seamy rock masses with poor discontinuity surfaces, or disintegrated rock with poor discontinuity conditions
4	Disintegrated First Shale rock mass at the west portal with poor to fair discontinuity conditions and weak to very weak intact rock strength.

Table 1: Ground Classes

EXCAVATION AND INITIAL SUPPORT

NATM excavation sequences and support designs were developed for the four support categories that correspond to the four ground classes described above. This section describes:

- The excavation sequence
- The main analyses performed to determine support element requirements within each support category
- The major support elements and support selection considerations used to develop the support categories
- Construction monitoring to be used during application of the designed support categories

Excavation Sequence

The overall excavation and support sequence consists of a top heading and bench. The top heading excavation will be accomplished using a single drift with a sloping core for face support. The bench excavation will be done in one or two stages depending on the support category and the lag maintained between the top heading and bench. A minimum lag between the top heading and bench is required to ensure equilibrium of the top heading under biaxial loading before additional loading is introduced as a result of the excavation of bench drifts. Drift advance length is primarily controlled by anticipated ground stand-up time and the size of the drift.

Analysis for Determination of Support Requirements

Convergence-confinement analyses were performed using Fast Lagrangian Analysis of Continua (FLAC 5.0, Itasca, 2005) to determine the required thickness of shotcrete lining and the length of rock dowels in the four support categories. The FLAC analyses simulated the excavation sequence and installation of perimeter rock dowels and several lifts of shotcrete. In addition, the models simulated the strength/stiffness gain of the shotcrete with time. The models were used to estimate the moments and thrusts that develop in the shotcrete lining and these results were plotted on thrust-moment interaction diagrams to verify that the loads are less than the capacity of the lining (see Figure 2).

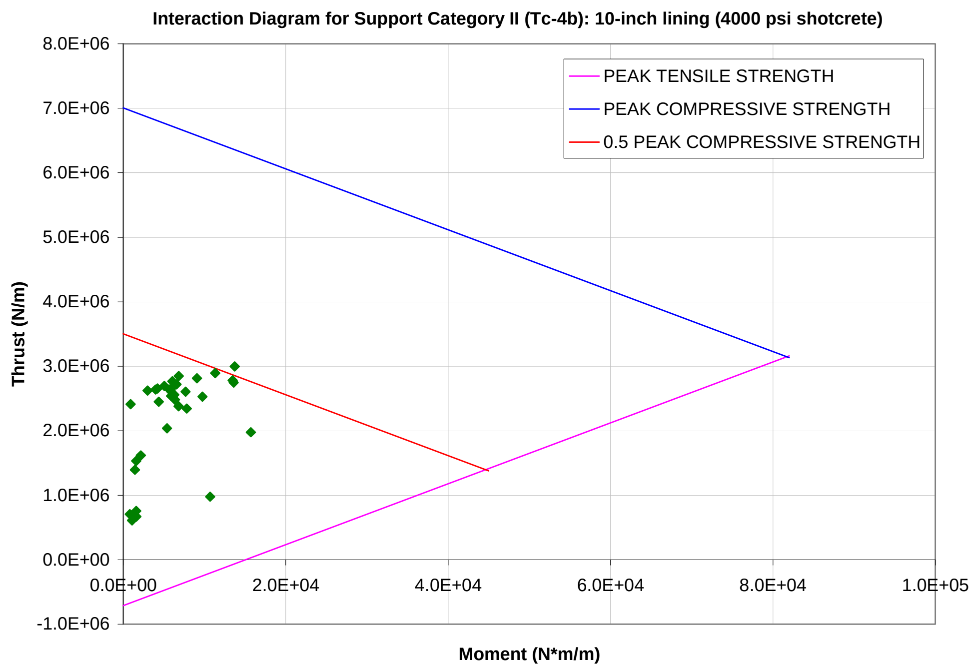

Figure 2. Moment – Thrust Diagram

The FLAC models incorporated an elastic-plastic material model to simulate the inelastic behavior of FRS. Each beam element was assigned a tensile and compressive strength, which, along with the section geometry, defines an interaction diagram. At each time step, the axial forces and moments are computed for each beam element and these forces and moments are compared to the capacity envelope. If the axial forces and moments fall outside the interaction diagram, the axial forces and moments are adjusted to return the values to the capacity envelope. This approach effectively limits the tensile stress that will develop in the shotcrete lining and permits plastic rotations and deformations to develop if the tensile capacity is reached. To assure that the section remains structurally viable, rotations that develop in the lining are plotted on thrust-curvature diagrams to verify that they are within allowable limits (see Figure 3).

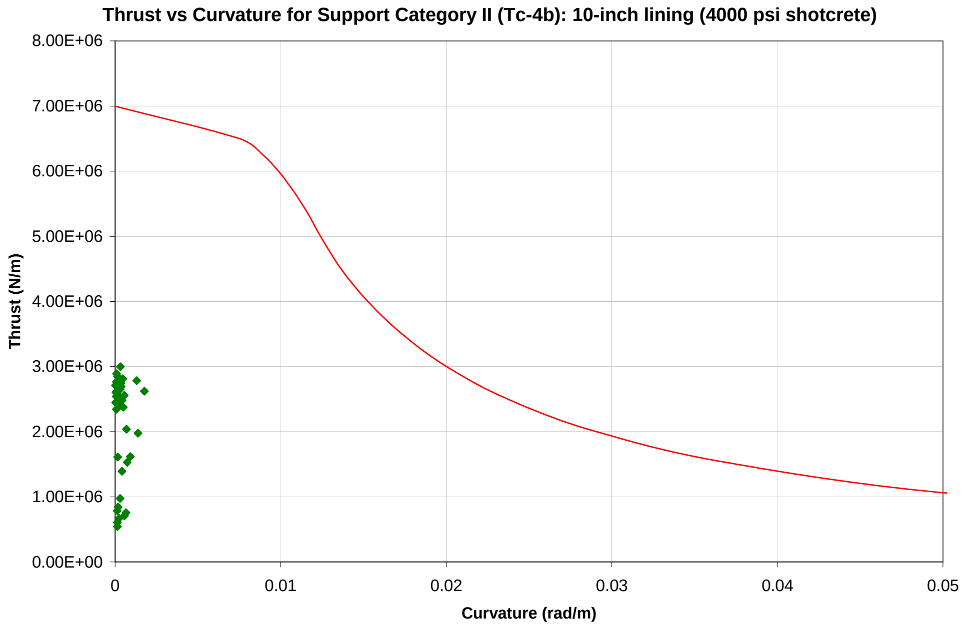

Figure 3. Thrust - Curvature Diagram

The elastic modulus of shotcrete used in the FLAC analyses varied from 5,000 to 15,000 MPa (725 to 2175 ksi) to account for creep in early age shotcrete (John and Mattle, 2003). The lining is initially assigned an elastic modulus of 5,000 MPa (725 ksi) immediately after installation. The elastic modulus is gradually increased to 15,000 MPa (2175 ksi) for hardened shotcrete, as the excavation progresses, to simulate the stiffness increase of the shotcrete as it increases with time.

FLAC3D models of the full NATM excavation and support operation (see Figure 4) in each support category were used to estimate the amount of relaxation that occurs in the ground ahead of drift headings, evaluate face stability, estimate the required bench lags, and evaluate spile performance. The methodology used to evaluate relaxation ahead of the face is shown on Figure 5. Results from the FLAC3D analyses were also used to cross-check FLAC2D results. Finally, keyblock analyses were performed to determine support requirements to prevent block failure at the tunnel face and along the perimeter.

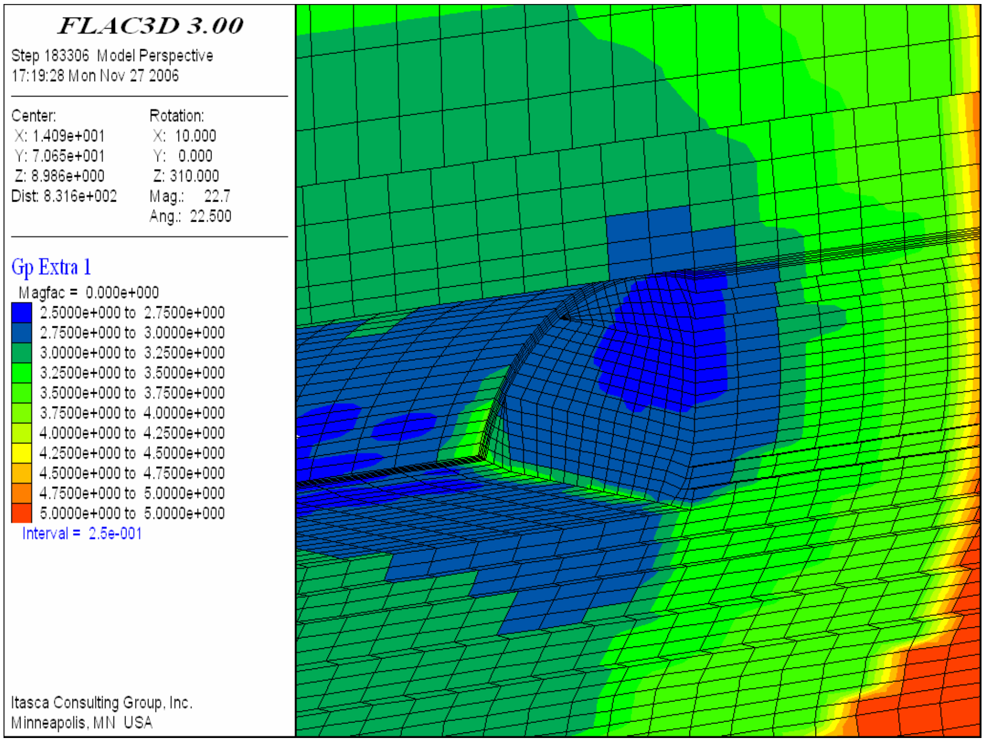

Figure 4. Face Stability Evaluation with FLAC3D

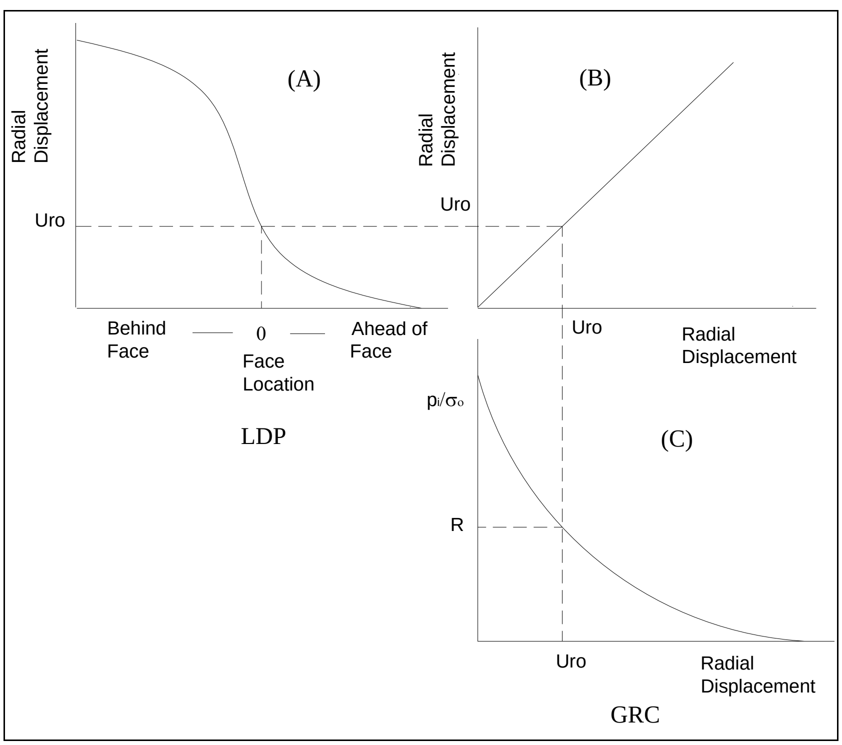

Figure 5. Schematic Illustration of Estimation of Ground Relaxation Factor Using FLAC3D Results

Initial Support Design

Support elements planned include plain and fiber-reinforced shotcrete (FRS), lattice girders, fast-setting cement-grouted rock dowels, fiberglass rock dowels, self-drilling and grouted spiles, injection spiles, and self-drilling grouted pipe spiles. The FRS will have a strength of 27 MPa (4,000 psi) in all support categories. Self-drilling spiles are used because drill holes are expected to be unstable where spiles are required.

Rock reinforcement, consisting of cement-grouted dowels, will be used in all support categories. In some of the more adverse rock , self-drilling cement-grouted dowels will be required. The top heading tunnel perimeter support will be installed within three rounds of the working face.

Spiles will be installed as part of Support Categories 1 through 3 so as to minimize overbreak and maintain the excavation profile line. In Support Category 4 that includes a pipe canopy, the excavation profile is flared along the inclination of the pipe canopy, to provide adequate clearance for the drill boom used to install the pipes.

Drainage holes and probe holes will be drilled ahead of the top heading to control the impact of water inflows on the stability of the ground around the tunnel excavation, and to identify adverse ground conditions ahead of the face.

Shotcrete thicknesses for the four support categories range from 203 millimeter (8 inch) to 304 millimeter (12 inch). Rock reinforcement will consist of 4-meter-long (13 foot) and 6-meter-long (20 foot) rock dowels. A sloping core is used for face support in all support categories. Systematic pre-support and lattice girders will be required in Support Categories 2-4. Maximum advance lengths for the four support categories range from 1 meter (3 feet) to 1.8 meter (6 foot).

Monitoring

An initial support and ground response monitoring program will be implemented during construction to verify that the performance of the initial support systems is within the anticipated range. The monitoring data will also be used as supplemental information to facilitate the selection of appropriate support categories during construction, and to help determine where additional support measures are needed. The monitoring instruments will measure displacements and loading of the shotcrete lining at points around the perimeter of the tunnel, and will monitor ground movements within the rock mass in the rock pillar between the third and fourth bores near the portals as well as within the slopes adjacent to the portals.

The tunnel monitoring program will use monitoring bolts and pressure cells to measure deflections and stresses in the shotcrete lining. Sets of deflection points and load cells will be installed together at defined points around the circumference of the shotcrete lining. Five monitoring types, defining the instrument locations along the tunnel lining in a cross-sectional view, will be used for the range of support categories along the main tunnel alignment and in the cross-passageways.

FINAL LINING AND SEISMIC DESIGN

Final Lining System

The Caldecott fourth bore uses a double lining system consisting of an initial support system (discussed above) and a cast-in-place reinforced concrete final lining (Figure 6). A waterproofing membrane with a geotextile backing layer for drainage will be installed between the initial support and the final lining. The initial support system is designed to carry the ground loads that develop during construction while the cast-in-place reinforced concrete final lining is designed to carry long-term ground loads and any additional loads due to interior finishes or equipment anchored to it. The final lining will also accommodate seismic deformations and provide a durable and sound tunnel lining.

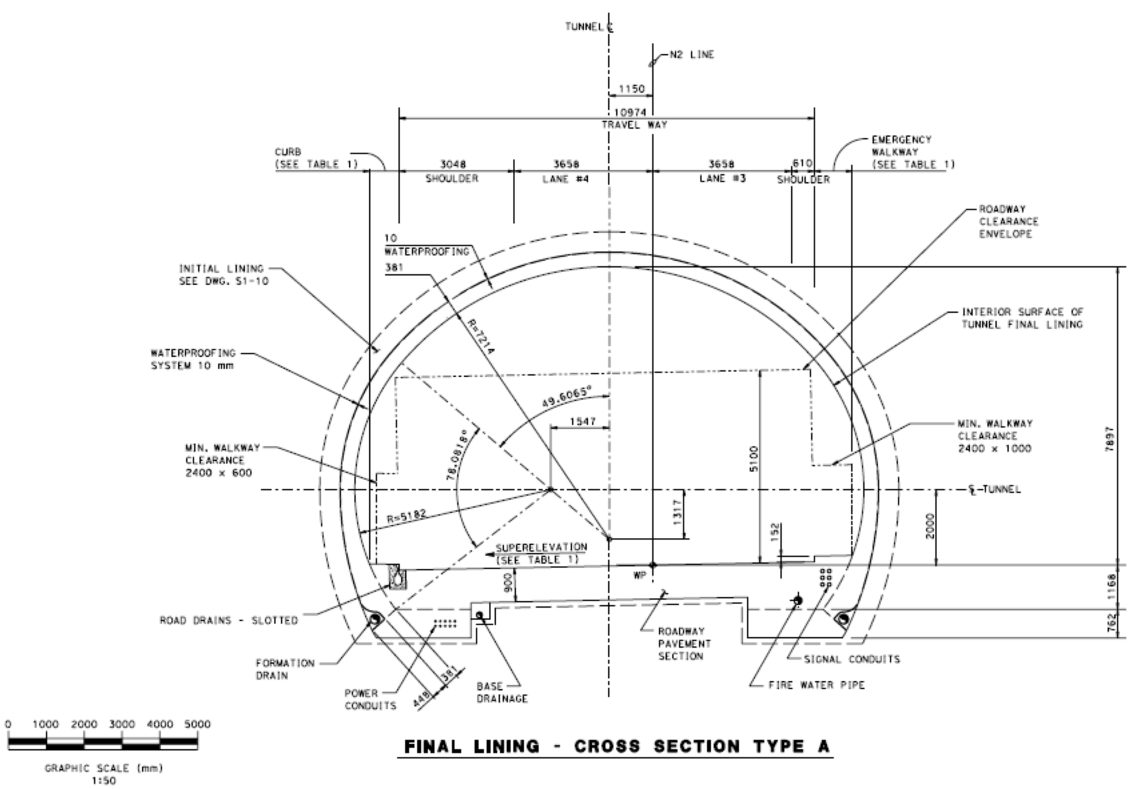

Figure 6: Final Lining

Loads and Load Combinations

The final lining will support its own dead load, ground loads from load sharing with the initial support system, rock wedge loads supported by the initial supports during construction, and seismic deformations. Two critical load conditions have been identified. The first load combination could occur during the first few years after completion of construction before any ground loads are imposed on the final lining. This load combination consists of the final lining dead load and seismic deformations. The second critical load combination combines maximum ground loads with seismic deformations.

Ground Loads from Load Sharing

The initial and final linings will function as a combined support system in the long term. Over time, after the completion of construction, a portion of the ground load carried by the initial support system will be transferred to the final lining due to deterioration of the initial support system rock dowels and shotcrete. Analyses were performed to assess the effect of the degradation of the initial support and to determine the part of the ground load that will be transferred to the final lining. The analyses assumed the dowels deteriorate completely in the long-term and that the modulus and strength of the shotcrete degrade to approximately 60% of the original design values. The initial shotcrete lining is also assumed to have no flexural capacity in the long-term due to possible deterioration of any reinforcing embedded therein. The results indicate that the final lining will attract a maximum of approximately 50% of the ground load supported by the initial lining. The final lining was conservatively designed to support 2/3 of the ground load supported by the initial lining.

Seismic Demand

General Performance Requirements. In accordance with general Caltrans practice for “important” facilities on lifeline routes such as SR 24, the seismic design for the tunnel is based on the Safety Evaluation Earthquake (SEE) and a lower-level Functional Evaluation Earthquake (FEE). The project uses a 1,500-year return period for the SEE event and a 300-year return period for the FEE event.

The performance requirements for the SEE are that the fourth bore will be open to emergency vehicle traffic within 72 hours following an SEE. Performance requirements for the FEE are that the fourth bore remains fully operational and experiences minimal, if any, damage.

Seismic Hazard Analysis. Deterministic seismic hazard analysis (DSHA) and a probabilistic seismic hazard analysis (PSHA) were used to characterize the seismic hazard at the project site (EMI, 2005). While numerous faults have been identified in the Bay Area, the Hayward fault was found to be the controlling fault because of its close proximity to the Caldecott Tunnels. Figure 7 shows uniform risk equal hazard spectra developed from the results of the PSHA and DSHA analyses.

Ground Motion Characterization and Wave Scattering Analysis. Based on site specific rock acceleration spectra, three sets of time histories were developed for each of the ground motion events (i.e. SEE and FEE). Wave scattering analyses using these time histories were performed to evaluate the effects of seismic wave propagation and to estimate the ground distortion around the tunnel lining for each time step of the input motion. These displacement time histories were used as input for the pseudo-static time history analysis described below.

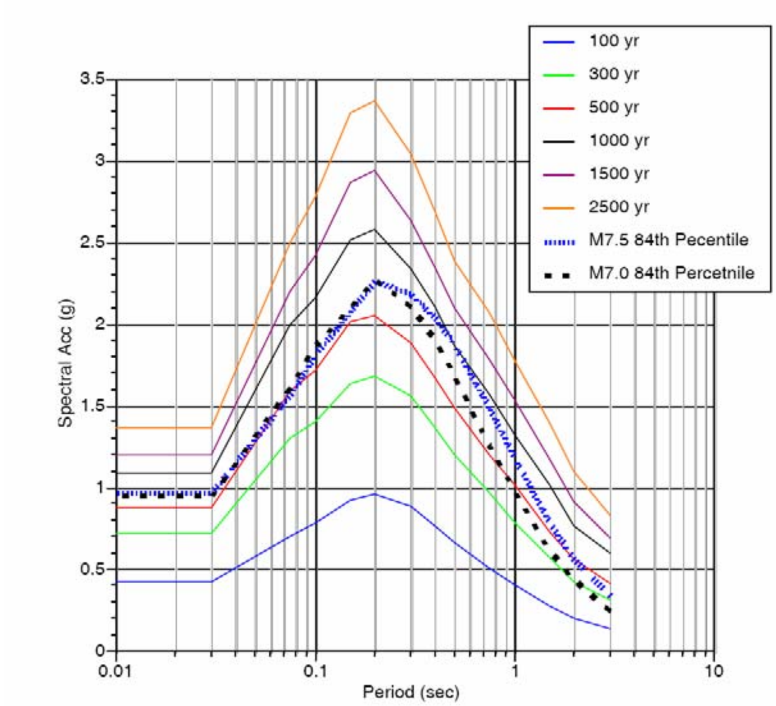

Figure 7: Uniform Risk Equal Hazard Spectra and MCE 84th Percentile Spectra at the Caldecott Fourth Bore

Tunnel Final Lining Seismic Analysis. Ground shaking and the associated ground deformations are the primary seismic design issue for the fourth bore. Three types of lining deformation were evaluated due to ground strains caused by wave propagation: longitudinal axial compression and tension, longitudinal bending, and ovaling or racking of the cross section. Two types of analyses were used to assess the behavior of the fourth bore lining due to longitudinal and racking seismic deformations. The first method uses closed-form solutions (Hashash et al., 2001 and Penzien, 1998 & 2000). The second is a state-of-the-art numerical method that uses beam-spring and beam-continuum models to perform pseudo-static time history analyses using the results of the scattering analyses described above. Two types of numerical models were used to calculate lining strains, stresses, and forces: 2-D SAP2000 (CSI, 2005) beam-spring models with nonlinear support springs (gap elements) to model ground behavior; and 2-D beam-continuum models using both FLAC (ITASCA, 2005) and ADINA (ADINA R&D Inc.) with elastic continuum elements to model ground behavior. Both methods were used to calculate strains, stresses, and forces in the fourth bore lining and cut-and-cover structures, and to ensure that the results were within acceptable stress and ductility limits.

Final Lining Design

Critical cross-sections in each support category were evaluated to determine the ability of the final lining to support the load combinations referenced above. Results of the analyses indicate that a 381 millimeter (15-inch) final lining with 35 MPa (5000 psi) concrete can support the ground loads and accommodate the seismic deformations. This final lining thickness was selected for constructability and is controlled by the thrust resulting from the ground loads in the high cover section of the alignment. Two layers of reinforcing will be used for the final lining to meet Caltrans criteria. The seismic demands, although very high, do not control the thickness of the final lining.

CONCLUSIONS

Preliminary design of the 15-meter-wide two-lane Caldecott fourth bore included identification of four ground classes that are expected along the alignment. Four corresponding excavation and initial support categories have been developed for NATM construction of the mined tunnel. Support elements include plain and fiber-reinforced shotcrete, lattice girders, fast-setting cement-grouted rock dowels, fiberglass rock dowels, self-drilling and grouted spiles, injection spiles, and self-drilling grouted pipe spiles. Shotcrete lining thickness in the four support categories ranges from 200 millimeters (8 inches) to 300 millimeters (12 inches).

The final lining will support dead load, ground loads, rock wedge loads, and seismic deformations. The design analysis shows that a 381 millimeter (15 inch) final lining with 35 MPa (5,000 psi) concrete can support the ground loads and accommodate the seismic deformations. Seismic demands do not control the thickness of the final lining, despite the close proximity of the project to a major active fault and seismic design criteria corresponding to an earthquake with a 1,500-year return period and a peak ground acceleration of 1.2g.

REFERENCES

Page, B.M. (1950), Geology of the Broadway Tunnel, Berkeley Hills, California, Economic Geology, Vo. 45, No.2

John, Max and Mattle, Bruno. “Shotcrete Lining Design: Factors of Influence”. 2003 RETC Proceedings.

Itasca Consulting Group Inc., 2005, Fast Lagrangian Analysis of Continua (FLAC) Version 5.0, Minneapolis.

Hashash, YMA et al., Seismic Design and Analysis of Underground Structures, Tunneling and Underground Space Technology 16 (2001) pp. 247-293, Elsevier

Penzien, J, Wu, C, Stresses in Linings of Bored Tunnels, Int. J Earthquake Eng. Struct. Dyn., 27, pp283-300, 1998

Penzien, J, Seismically Induced Racking of Tunnel Linings, Int. J Earthquake Eng. Struct. Dyn 29, pp. 683-691,2 000

SAP 2000, Computer and Structures Inc, Berkeley, California, 2005

ADINA, ADINA R&D, Watertown, Massachesetts

Earth Mechanics Institute, Seismic Hazard Report and SEE Ground Motions, Caldecott Improvement Project, 2005

Bieniawski, Z.T. (1989), Engineering Rock Mass Classifications, Wiley, New York

ACKOWLEDGEMENTS

The authors would like to acknowledge Geomatrix Consultants for their work on the site geology, ILF Consultants for independent reviews of the initial support designs, Earth Mechanics Inc. for their work on seismic hazard and wave scattering analysis and SC Solutions for their work on seismic demand analysis.