

Design of NATM Initial Support on the Caldecott 4th Bore

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ABSTRACT: The proposed Caldecott 4th Bore, located along State Route 24 in Oakland, California is a 15 m wide, 9.7 m high, highway tunnel that will be constructed using the New Austrian Tunneling Method (NATM). Rock mass strength and deformability properties including local effects due to strain softening, intact rock softening and strength degradation due to erosion and softening of joint infillings are key design considerations for the project. Ground/structure interaction issues modeled in numerical analyses include stress relaxation ahead of the tunnel heading, arching of loads across weak zones, face stability, effects of early age creep in shotcrete and stress redistributions due to the formation of plastic hinges in the shotcrete lining. The design also evaluated forces, moments and rotations in the shotcrete lining to determine ground support requirements.

1. INTRODUCTION

1.1. Project Background

The existing Caldecott Tunnel complex includes 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 tunnel that will provide two additional traffic lanes. The proposed horseshoe-shaped fourth bore is 1,036 m (3,399 ft) long, with a span of 15 m (50 ft), and a height of 9.7 m (32 ft). The fourth bore will be constructed using the New Austrian Tunneling Method (NATM). The project will include short sections of cut-and cover tunnel at each portal, seven cross-passage tunnels between the fourth bore and the existing third bore, electrical substation buildings, and a new operations and control building. State Route 24, considered a lifeline route by Caltrans, is required to be open to emergency vehicles 72 hours after an earthquake with a return period of 1,500 years and a peak ground acceleration of 1.2 g. Construction of the fourth bore is anticipated to begin in the summer of 2009 and be completed in 2014.

1.2. Geology

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 [1]. 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 [1]. 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.

The 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 zones of weak ground will be encountered, such as smaller-scale faults, shears, and crushed zones.

Intrusive sandstone dikes and hydrothermally-altered igneous dikes occur most frequently in the Claremont Chert and Shale, but may be encountered less frequently in other geologic units.

The structure of the rock mass units along the alignment varies from blocky in the best ground to disintegrated or crushed in the poorest-quality rock. RQD ranges from 5 to 81. Rock Mass Ratings [2] and Q values [3] at the tunnel scale vary from 20 to 65, and 0.006 and 10.5, respectively. Intact rock strength varies from weak to moderate along the alignment. Average values of measured unconfined compressive strength in the various geologic units vary from 5.2 MPa (750 psi) to 21.6 MPa (3190 psi). Mudstone, siltstone, and shale in the Orinda and Claremont Formations are expected to exhibit swelling behavior. The fourth bore has been classified as a gassy tunnel by the California Occupational Safety and Health Administration.

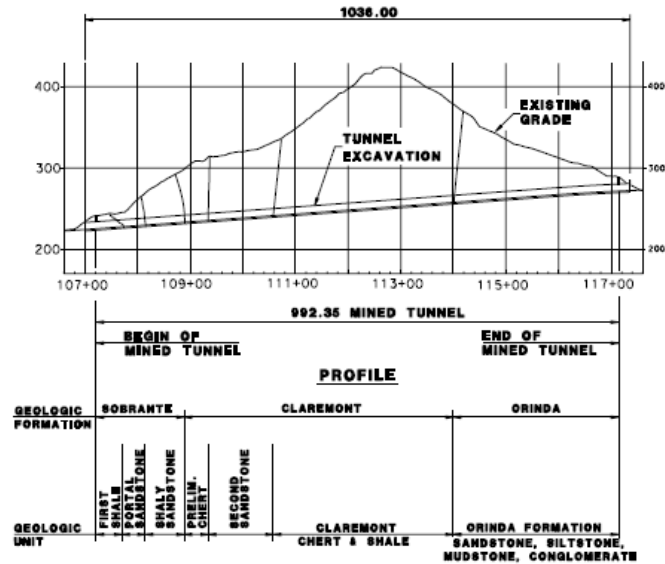


Figure 1: Geologic Formations and Geologic Units

1.3. 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 ($M < 6$) earthquakes, and more than a dozen moderate- to large-magnitude ($M > 6$) earthquakes. Major faults that comprise the 80-km-wide plate boundary in the San Francisco Bay Region include the San Gregorio, San Andreas, Hayward, and Calaveras Faults.

The active Hayward Fault, located 1.4 km (0.9 mi) west of the Caldecott Tunnel, is the closest major fault to the project site, capable of producing a magnitude 7.4 earthquake. The southern segment of the Hayward Fault produced the 1868 Hayward earthquake of estimated magnitude 6.8 that was accompanied by 30 to 35 km (19 to 22 mi) of surface faulting.

2. GROUND CLASSIFICATION APPROACH

The methodology used to characterize ground conditions along the alignment involved: (1) identifying and

characterizing Rock Mass Types (RMTs) based on an evaluation of geologic characteristics and relevant geomechanical parameters; and (2) grouping the RMTs into ground classes based on the anticipated rock mass behaviors. This approach is summarized in Figure 2.

Rock Mass Types

The identification of RMTs depends on the geological characteristics and relevant geomechanical parameters. The geologic units along the alignment each have characteristic lithologic properties, so these geologic units were the first criteria used to divide the alignment into potential RMTs. Many of these geologic units can be sub-divided based on fracture intensity and unconfined compressive strength (UCS). Other factors such as discontinuity surface characteristics, slake durability, permeability, and orientation of structures are less distinguishing, but are still important for determining the mechanical properties and behavior of the rock masses.

The geologic units were divided into 18 RMTs. Mechanical properties were estimated for each of the RMTs along the alignment and ground behaviors were evaluated considering the identified boundary conditions.

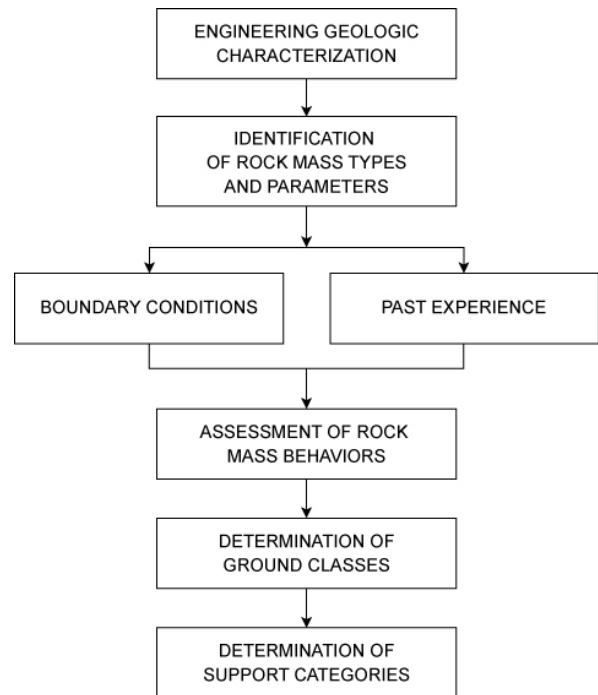


Figure 2: Flow Chart for Geotechnical Basis of Design

2.1. Ground Behaviors

The behavior of each RMT is dependent on its geomechanical properties and boundary conditions, including in situ stresses, tunnel dimension and orientation relative to the geologic structure, and groundwater conditions.

- The major ground behaviors that were evaluated include:
- Discontinuity-controlled failures,
- Stress-related shear failure within the rock mass,

- Raveling,
- Slaking/softening/swelling conditions

The potential occurrence of removable blocks around the arch and at the face of the full tunnel section was evaluated based on the identified discontinuity sets in each of the rock masses with a blocky character using Pantechnica Computational Workshop[4] and Unwedge [5].

To evaluate the potential for shear failure, a closed form elasto-plastic analysis was performed for an unsupported circular tunnel (7.5-m radius) in a hydrostatic in situ stress state. The solution provided by Caranza-Torres [7] as a function of the Hoek-Brown failure criterion and rock mass elastic modulus as implemented in the Rocscience software program Rocsupport was used.

The severity of stress-induced instability along the alignment was evaluated using the prediction of tunnel convergence (radial strain) for the highest cover in each RMT. Following the guidelines proposed by Hoek and Marinos [6], either deep seated shear failure or shallow shear failure were predicted, depending on whether the radial convergence was greater than about 2.5% or between 0.3% to 2.5% respectively.

RMTs prone to raveling, slaking/softening and swelling were identified based on evaluations of the rock mass characteristics. Raveling behavior was identified as a possibility in heavily broken RMTs as indicated by RQD and observations of core. Slaking was evaluated using results of slake potential tests. Swelling behavior was assessed based on past experience in the existing Caldecott Tunnel and from swelling test results. The evaluations for strength reduction related to seepage related degradation of the rock mass, intact rock softening and strain softening are discussed below.

All of the above ground behaviors influence the stand-up time. Stand-up time in an unsupported tunnel was evaluated empirically, using correlations to RMR, as published by Bieniawski [9].

Based on these evaluations and a review of past construction experiences, the ground behaviors anticipated in the rock masses of the fourth bore alignment are summarized in Table 1.

2.2. Ground Classes

A typical NATM design includes different excavation and initial support categories. Individually, the support categories address sets of similar ground behaviors, and as a whole they address all anticipated ground behaviors along the alignment.

The design groups the RMTs into four major ground classes based on similarity in anticipated ground behavior. Three of these ground classes are further divided into two sub-types. Each ground class correlates to a specific support category. 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.

Any unique behaviors of a particular ground class that require specific excavation or initial support measures will be addressed through additional support measures for that support category. NATM sequence and support designs have been developed for all support categories. Ground modification

measures such as predrainage or presupport ahead of the tunnel heading are included in initial support requirements.

Behavior	Description of failure modes
Block failures	Discontinuity-controlled, gravity-induced failure of rock blocks
Raveling	Progressive, discontinuity controlled failure of small rock blocks within the general rock mass at or near the excavation surface.
Shallow shear failure	Shallow shear failures result from overstressing of the ground within 0.25D to 0.5D of the tunnel perimeter (D=tunnel width), and may be enhanced by the potential for discontinuity and gravity-controlled failure modes.
Deep shear failures	Deep-seated shear failures result from overstressing of the ground beyond 0.25D to 0.5D from the tunnel perimeter (D=tunnel width).
Slaking/softening	Slaking is the deterioration and breakdown of intact rock upon exposure by excavation. Softening, which is dependent on wetting and exposure by excavation, is the reduction of intact rock strength at the invert or elsewhere
Swelling	Swelling occurs due to absorption of water by clay minerals in rock upon excavation induced unloading.
Crown instability due to low cover	Excessive crown overbreak and chimney-type failure under low cover reaches at portals.
Instability of tunnel face	Instability of the tunnel face due to discontinuity-controlled, gravity-induced sliding of blocks; shear failure into the face; and raveling at face due to tight or closely-spaced cross joints.

Table 1: Anticipated Ground Behaviors

3. ROCK MASS CHARACTERIZATION

Generally, the ground along the alignment consists of a jointed rock mass. Consistent with expected failure modes, strength and deformability characterization included an assessment of the overall rock mass properties for an isotropically jointed rock mass as well as the strength of discontinuities that define keyblocks and control failure involving adverse bedding/joint plane orientations. Additionally, some of the RMT within the Caldecott fourth bore alignment are prone to localized weakening upon excavation as discussed above and this propensity was considered in the development of the rock mass characteristics. The discussion below is limited to evaluations of RMT rock mass strength and deformability, including localized weakening effects.

3.1. Strength

Rock mass properties are controlled by the properties of the intact rock pieces and the freedom of these pieces to slide and rotate under different stress conditions [9] and are scale dependent. The volume of rock within one diameter of the tunnel excavation will largely control the behavior of the

tunnel opening [9]. Laboratory tests only provide data of the intact rock while, in-situ field tests enable rock mass characterization on a larger scale, but even field tests may not address the influence of all discontinuities and heterogeneities of the rock mass at the scale of concern. Hence, rock mass properties were determined by using a combination of geologic characterization, laboratory testing, and empirical methods. Geologic characterization is based on observation of core, core logs, core photographs, and outcrops. Laboratory test results used in the assessment of rock mass strength parameters include uniaxial and triaxial compression tests, direct shear tests, field point load tests, and field index tests. The empirical method used to assess rock mass properties is the Geological Strength Index (GSI) system [10].

The GSI system empirically combines qualitative engineering geology assessments and laboratory test results to estimate rock mass strengths. The system involves determining the GSI and intact unconfined compressive strength (UCS) classification parameters, Hoek-Brown parameter (m_i), and disturbance factor (D). From these classification parameters, the effective stress Hoek-Brown strength parameters m_b , s , and a can be calculated as detailed in Hoek, Carranza-Torres, and Corkum [10]. Also, best-fit effective stress Mohr-Coulomb strength parameters c' and ϕ' can be determined from the non-linear Hoek-Brown strength envelope.

Evaluation of the classification parameters was performed on core lengths of about 15 m (50 ft.) although in a few instances lengths as short as 6.1 m (20 ft.) were used in order to accommodate heterogeneity along the core. Each of these core lengths is referred to as “samples” in the following discussion. The parameters for each sample were projected to the tunnel elevation along the dip direction of bedding

Based on qualitative identification of the appropriate rock mass structure and discontinuity strength, GSI evaluations were made for each sample using Marininos et. al. [10]. In addition, the RMR and Q systems were used as a secondary evaluation of GSI with the necessary adjustments.

Evaluations of m_i and D were based on recommended values by Hoek et al. [8], and Hoek-Diederichs [11], respectively. A disturbance factor of $D < 0.25$ was assigned globally for the evaluation of rock mass strength and deformability since the excavation-induced disturbance is likely to be minimal if a roadheader is used as anticipated. Table 2 summarizes the strength properties for each RMT.

3.2. Deformability

The most common measure of deformability in a rock mass used in underground design is the deformation modulus as described by Hoek et al. [11]. The deformation modulus corresponds to a strain level beyond the initial part of the stress-strain curve associated with closing of gaps and the mechanical components of the loading system. The deformation modulus accounts for elastic strains in the rock mass as well as inelastic or irrecoverable strains that occur near the tunnel excavation.

In contrast to the elastic response that can be expected from a fresh, hard rock sample in a laboratory specimen, a rock mass generally will not exhibit perfect elasticity. This is because of the combined effects of fissures, fractures, bedding planes, contacts, zones of altered rock, and clays with plastic

properties [12]. Excavation-induced disturbance can further reduce the deformation modulus. Consequently, the deformation modulus is always smaller than the elastic modulus. The deformation modulus is used for tunnel design since it represents both elastic and inelastic responses of rock masses and also allows for construction-induced rock mass disturbance. Hoek and Brown [13] summarize the use of the deformation modulus for analysis of several tunnels.

The rock mass deformation modulus is one of the most sensitive parameters for modelling the stress and strain state around a tunnel opening. Estimates of modulus in a fractured rock mass can have a significant error margin [11], but the probable range of the deformation modulus of a rock mass can be estimated [14]. There are several methods of determining the deformation modulus. Apart from direct in-situ measurements (i.e. pressuremeter tests, dilatometer, etc.) other approaches are available to derive the rock mass deformation modulus. These methods include an analytical approach using the intact rock properties and discontinuity properties or empirical methods using classification schemes.

For the fourth bore, several approaches were used in combination to estimate the range of deformability of the rock mass. The methods used included:

- Empirical equations by Hoek, Carranza-Torres, and Corkum [11] and Hoek-Diederichs [12].
- A mechanical model of fractured rock deformability by Rafael and Goodman [15].
- Available downhole seismic tests using a ratio between static modulus and dynamic modulus of 13 for highly fractured rock and 7 for moderately fractured rock [15].
- Field testing results from pressure meter and Goodman jack testing in crushed or massive materials.
- Measurements of the intact rock modulus (provides an upper bound value).

All of these approaches to the estimation of rock mass modulus have limitations. Various studies clearly indicate the limits of rock mass estimation methods, i.e. Edelbro et al. [16] and M. Romana [17]. A combination of approaches to the estimation of the deformation modulus was used due to these limitations.

Table 2 summarizes the modulus of deformation and Poisson's ratio for each RMT.

3.3. Local Weakening Effects *Erosion and Softening of Joint Infillings*

Water can have a deleterious effect on the strength of rock masses. Joint fillings can be eroded or softened upon excavation and erosion of joint infillings, or piping, loosens the rock mass, making it less interlocked and more prone to instability. This effect is most significant for RMTs with disintegrated or very seamy structures. Softened joint infillings also reduce the shear strength of joints, which is

most significant for discontinuities with persistent, fine-grained infillings. Also, the higher the plasticity of the infilling material, the larger the shear strength reduction will be upon softening.

The extent of infilling erosion or softening will be most significant in the rock mass immediately adjacent to the tunnel because the effects of excavation-induced stress relief and higher seepage velocity will be more pronounced close to the tunnel excavation. To properly account for the strength loss in susceptible rock masses, reduced strength parameters were applied the volume of rock within 1.5 meters of the tunnel perimeter in the RMTs susceptible to softening, which correspond to rock masses with disintegrated or very seamy structures.

The GSI was reduced to represent strength reductions. The reduction in the GSI value is specific to RMTs as is based on evaluations of groundwater conditions, rock type and joint infillings. The reductions ranged from 3 to 6 GSI points.

Intact Rock Softening

Intact rock softening is caused by absorption of free water upon excavation stress relief. Within the fourth bore alignment, only the mudstone and siltstone beds in the Orinda Formation are known to experience significant softening of the intact rock. This phenomenon is attributed to the clay mineralogy of the rock, as well as the degree of induration and cementation. Permeable conglomerate and sandstone beds provide pathways for groundwater to access otherwise impermeable mudstone and siltstone beds. The softening will occur only in the rock mass close to the tunnel excavation. For design purposes, it was assumed that the average intact UCS of the rock is reduced by 40 percent to a limited depth from the tunnel periphery, which is the upper bound of strength degradation for mudstones, according to studies by Morgenstern and Eigenbrod [23].

Strain Softening

Strain softening is only expected for the RMTs that show a potential for significant stress-related shear failure and plasticity. Based on an elasto-plastic analysis of an unsupported tunnel, convergence is only expected to exceed 1 percent (which is the threshold for minor squeezing) in two RMTs corresponding to crushed chert and shale (Tc-5) of the Claremont Formation, and the Orinda Formation mudstone, siltstone and sandstone under highest cover (Tor-2). This indicates that significant plasticity and shear failure will be limited to these two RMTs.

The plastic zone that forms around an unsupported tunnel in these two RMTs is slightly greater than half the tunnel diameter, or about 8 m (26 ft.). Laboratory triaxial test results performed on samples of clayey crushed rock of the Claremont Formation show no strain softening within the confining stress range expected around the walls and crown of the fourth bore in this material. This is consistent with the recommendation of Hoek and Brown [13] that an elastic-perfectly plastic model is appropriate for very poor quality rock masses. In contrast, the majority of the triaxial tests

performed on samples of the Orinda Formation (Tor-2 RMT) intact rock do exhibit strain softening behavior, with some samples also exhibiting elastic-plastic and strain hardening behavior. Hoek and Brown [13] recommend reducing the GSI to represent strain softening in average quality rock masses comparable to the Orinda Formation rock. In our assessment the GSI values were lowered such that the unconfined compressive rock mass strength was reduced by about 40%.

Rock Mass Type	GSI Classification Parameters					Deformation Modulus	
	Intact UCS		GSI	m _i	D	(MPa)	(ksi)
	(MPa)	(psi)					
Tsf	5.2	750	26	7	0	410	60
Tsp	39.3	5700	34	17	0	2070	300
Tss-1	29.7	4300	38	16	0	1380	200
Tss-2	29.7	4300	24	16	0	830	120
Tcp	34.5	5000	28	8	0	1380	200
Tcs-1	19.3	2800	56	17	0	2070	300
Tcs-2	17.6	2550	42	17	0	1030	150
Tcs-3	12.1	1750	35	15	0	690	100
Tc-1	47.6	6900	45	8	0	4480	650
Tc-2	34.5	5000	36	8	0	2410	350
Tc-3	18.6	2700	45	7	0	2070	300
Tc-4a	27.6	4000	33	12	0	1380	200
Tc-4b	41.4	6000	33	8	0	2070	300
Tc-5	34.5	5000	22	8	0	690	100
Tc-5a	18.6	2700	24	7	0	690	100
Tor-1	10.9	1580	58	7	0	1380	200
Tor-2	8.3	1200	45	7	0	1030	150
Tor-3	5.2	750	47	7	0	690	100

Table 2: RMT Properties

4. GROUND STRUCTURE INTERACTION ISSUES

This section describes some of the key considerations for the analysis performed to evaluate the tunnel initial support systems. These issues include: stress relaxation ahead of the face, face stability, and arching of loads across weak zones and have been described in detail in Thapa et. al. [19]. Other issues include creep of early age shotcrete and the formation of plastic hinges in the shotcrete lining. These issues are described in relation to ground structure interaction analysis of support requirements. Analyses performed to determine block support requirements are not described in this paper.

4.1. Relaxation ahead of the face

FLAC3D models of the full NATM excavation and support operation were developed for each support category to estimate the amount of relaxation in the ground ahead of tunnel face. The FLAC3D models explicitly represent the face support and spiling presupport. Figure 3 shows a partial view of a FLAC3D model at the tunnel heading. The shotcrete lining is modeled using Mohr-Coulomb elastic-plastic continuum elements in FLAC3D. The hardening of the shotcrete lining is modeled as the tunnel top heading and bench excavations advance at prescribed rates and lags to

represent the early age creep effects of shotcrete described in Thapa et al. [21]. Ground relaxation factors are estimated based on a tunnel longitudinal displacement profile (LDP) from the FLAC3D analysis and a ground reaction curve (GRC) developed from a FLAC2D analysis. The tunnel LDP demonstrates the development of tunnel radial displacement as a function of distance along the length of the excavation. The GRC shows the tunnel radial displacements as a function of support pressure, and can be generated from a two-dimensional FLAC analysis using an approach consistent in principle with the current practice in tunnel design [5, 21]. Using the methodology described in Thapa et. al. [21], the amount of stress relaxation ahead of the tunnel excavation can be determined and input into the two-dimensional analysis.

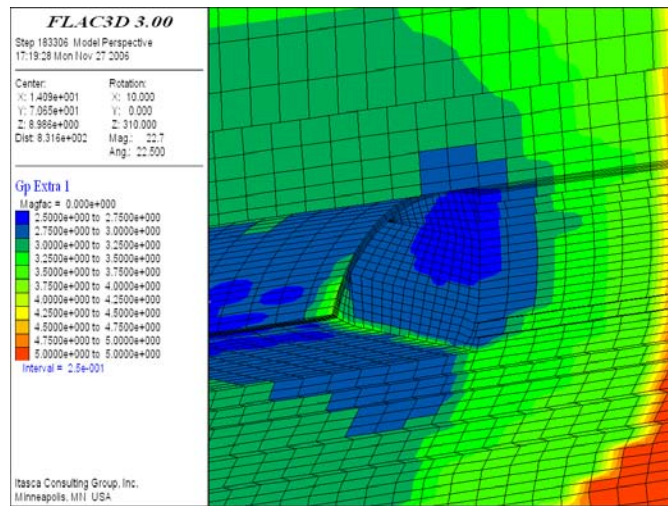


Figure 3: Longitudinal view of FLAC3D model of tunnel heading (section through the tunnel centerline)

4.2. Arching of loads across weak zones

Reaches of the tunnel up to about 6 m long will encounter poor to very poor quality ground in between adjacent reaches of better quality ground in the Claremont Chert and Shale geologic unit. Shotcrete lining thickness requirements for these reaches were evaluated using FLAC3D to account for the effect of longitudinal arching on lining loads. Comparison of FLAC3D results to a FLAC2D analysis that does not account for longitudinal arching shows that lining loads computed using FLAC3D are about 30% lower than the loads calculated using a plane-strain FLAC2D analysis. It is noted that the FLAC3D results are in general agreement with the FLAC2D results in other reaches with uniform ground conditions.

4.3. Face Stability

FLAC3D was also used to evaluate face stability by determining the factor of safety (F) against global shear failure for the top heading drift. The face stability analyses were performed after the simulation of tunnel excavation and support application so that these evaluations incorporate stress conditions affecting face stability. Additionally, this approach has the advantage of including all three dimensional stresses and strains ahead of, at and behind the tunnel face. Our approach to the FLAC3D face stability analysis, used a factor of safety (F) that the rock mass shear strength must be divided

by to bring the drift face to the verge of failure. The factor of safety (F) is expressed as:

$$F = \frac{\tau_o}{\tau_r}$$

where, τ_o is the actual shear strength and τ_r is the reduced shear strength at incipient failure. The factor of safety of the face was evaluated using FLAC3D iterations at different values of F and contouring all zones that fail in each iteration. A velocity criterion was used in FLAC3D to define failure of any zone during an iteration.

Predicted factors of safety against general shear failure for the top heading drift ranged from 3.2 in Support Category I without any face support to 1.3 in Support Category III with a sloping core for face support.

4.4. Shotcrete creep and development of plastic hinges

The fiber reinforced shotcrete lining was modeled in FLAC using beam elements. The elastic modulus of shotcrete was varied from 5,000 to 15,000 MPa depending on the location in the excavation [22], to account for the effect of creep in early-age shotcrete. The lining is initially assigned an elastic modulus of 5,000 MPa immediately after installation. The elastic modulus is gradually increased to 15,000 MPa for hardened shotcrete, as the excavation progresses, to simulate the stiffness of the shotcrete as it increases with time. This approach was used to simulate the effect of varying shotcrete stiffness on ground-structure interactions of stress redistributions in the ground, tunnel convergence and lining loading that occur as consequence of excavation and support installation.

The analysis also incorporates an elastic-plastic material model to simulate the inelastic behavior of fiber reinforced shotcrete. Each beam element was assigned a tensile and compressive strength, which along with the section geometry defines an interaction diagram as shown in Figure 4. At each computational 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. The adjustment is based on the ratio of overstressing, which is defined by:

$$R = \frac{\text{Stress}}{\text{Strength}} = \frac{\frac{P}{A} - \frac{M}{S}}{f_t}$$

Where:

P = Axial Force in Lining

M = Moment in Lining

A = Cross-sectional Area

S = Section Modulus

f_t = Tensile Strength of fiber reinforced shotcrete

The axial forces and moments in the beam elements are then reduced by the overstressing ratio in equal proportions until they fall within the 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. This feature of the analysis allows the development of a shear failure mechanism should the combination of load in the support system and ground shear strength lead to such a ground structure behavior.

The analysis assumes a peak tensile strength for fiber reinforced shotcrete of 2.8 MPa (410 psi) and a residual tensile strength of 0.8 MPa (120 psi). The analysis neglects the presence of lattice girders. The analysis assumes peak 28-day compression strength for fiber reinforced shotcrete of 27.6 MPa (4000 psi) for all Support Categories. The compressive strength is continuously updated in the analysis from the 1-day strength to the 28-day ultimate strength based on the advance rate of the excavation.

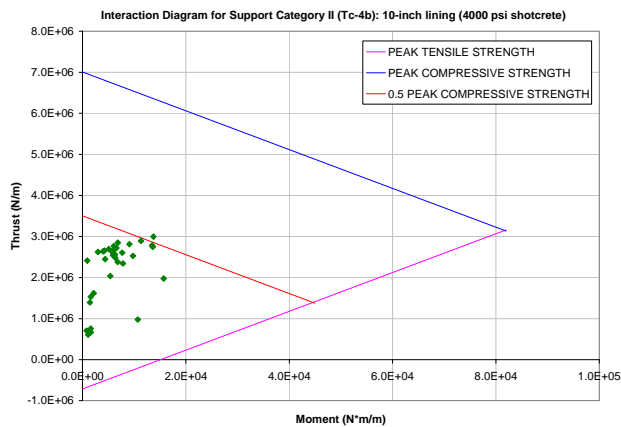


Figure 4: Moment – Thrust Interaction Diagram

An analysis was performed to check that the rotational capacity of the shotcrete lining throughout the individual construction sequences is not exceeded. Rotations that develop in the lining were plotted on thrust-curvature diagrams [24] to verify that they are within allowable limits. The rotations that develop at the nodes of the lining were used to compute the curvature induced in the lining. The curvature was monitored at all stages of the excavation and was compared with a predefined allowable curvature. The allowable curvature is defined as the curvature that will result in a compressive strain 0.003, and is uniquely defined by the section properties, material properties of the shotcrete and the axial thrust in the shotcrete lining.

5. SUPPORT REQUIREMENTS

The final step of the geotechnical design process shown in Figure 2 is determination of support requirements for the four major ground classes, and sub-types, described above. Support categories define the excavation and support requirements for each ground class.

Support requirements increase proceeding from Support Category I to IV, corresponding to changes in ground class

predominant behaviors. The shotcrete lining requirement varies from 203-mm (8-in-thick) in Support Category I to 304 mm (12 in) in Support Category IV. Radial spacing of rock reinforcement is 1.8 m in Support Category I and decreases to 1.5 m in Support Category III. Systematic pre-support is required in all support categories except Support Category I. An invert arch is required in Support Categories IIIB and IV in both the top heading and bench. Advance lengths vary from 1.8 m in Support Category I to 1 m in Support Category IV. Figure 5 shows a typical cross-section of the tunnel indicating support requirements.

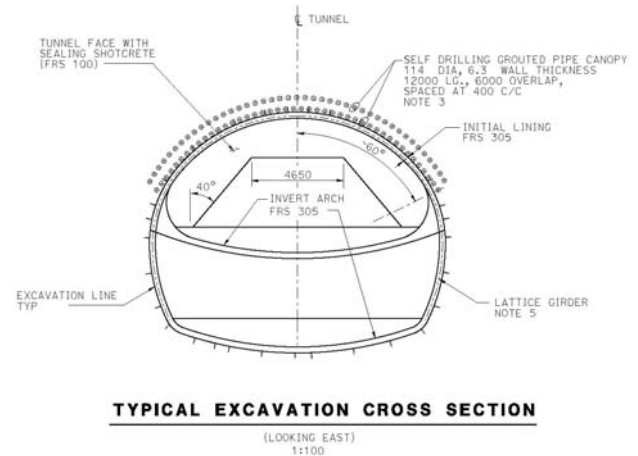


Figure 5: Support requirements in Support Category IV

5.1. Construction Sequencing Requirements

The dimensions of the top heading and support installation requirements, and the bench sequence, are based on anticipated ground behavior as well as typical construction equipment limitations and construction logistics. Drift advance length is primarily controlled by anticipated ground stand-up time and the size of the drift.

The overall excavation and support sequence consists of a top heading and bench. The top heading excavation will be accomplished using a single drift. The design allows the top heading to be driven as far ahead of the bench as the contractor chooses, but the bench excavation is subject to a minimum lag. The lag distance between the face of the top heading drift and the first benching operation is controlled by longitudinal arching and support installation sequencing. The extent of longitudinal arching is determined by three-dimensional simulation of the construction sequence using FLAC3D. The required lag between the top heading drift and the start of benching is based on the distance needed for the top heading drift to pass into a biaxial state of stress. Thus, the lag separation between the top heading drift and benching ensures equilibrium under biaxial loading before additional loading is introduced as a result of the excavation of bench drifts.

The bench excavation can be performed in two stages in support categories that do not require an invert arch. The two-staged benching allows the bench to be carried along with the top heading using a ramp. However, if the contractor chooses to drive the top heading far ahead of the bench, bench excavation can be performed full face instead of in two stages as continuous ramp access to the top heading is not expected

to be required. Bench excavation will be performed in one stage in support categories where an invert arch is required.

In addition to the minimum required separation (lag) between the top heading drift and the benching operation, the design also requires that support installation within the bench be completed within a specified distance of the start of bench excavation.

6. CONCLUSIONS

Rock mass quality is expected to vary from very poor to good along the alignment of this large span tunnel. The excavation and support design classified this range of anticipated ground conditions into four major ground classes based on rock mass characteristics and behavior in an unsupported opening. Three of these ground classes were further sub-divided into two sub-types each to accommodate behaviors expected under specific in-situ conditions or ground characteristics. Support requirements corresponding to each ground class were developed. Support requirements will be adjusted using additional support measures during construction based on observations of ground conditions and behaviors.

The analytical approach to the design of the fourth bore required careful consideration of rock mass strength and deformability parameter selections, including local weakening effects due to erosion and softening of joint infillings, intact rock softening and strain softening. Strength and deformability parameters were estimated for 18 Rock Mass Types along the alignment, accounting for issues such as scale dependency of rock mass properties and limitations in the various approaches to estimating deformation modulus.

The geotechnical design process involved a process starting with engineering geologic characterization of the rock mass along the alignment and proceeded through a sequence of evaluations to identify rock mass types, behaviors of these rock mass types, grouping of the rock mass types into ground classes and finally determination of the support requirements for the ground classes.

Ground-structure interaction analysis was used for determination of support requirements and excavation sequence restrictions for critical sections in each ground class. Determination of support requirements required analysis of the effect of stress relaxations ahead of the tunnel face, stability of the tunnel heading, longitudinal arching across weak zones and early age shotcrete behavior.

Required shotcrete thickness for the support categories varies from 203 mm (8 in.) to 304 mm (12 in.) in relation to variations in predominant ground behaviors. Longitudinal arching controls excavation sequencing and distances within which support must be installed in each support category.

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