The Key for an Appropriate NATM Verification Process: The Observational Method during Construction of the Devil’s Slide Tunnel, California

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1 PROJECT OVERVIEW

The California Department of Transportation (Caltrans) is building twin highway tunnels through the Devil’s Slide area just south of the City of Pacifica in the County of San Mateo, California. The new 2-lane roadway leaves Route 1 and crosses over the Shamrock Ranch valley on twin one-lane bridges over a distance of about 365 meters to the North Portals of the twin one-lane tunnels. The tunnels extend beneath San Pedro Mountain for 1250 meters to the South Portals. From the South Portals the roadway rejoins Route 1 over a distance of about 500 meters, beside the Disposal Area and Operations and Maintenance Center (OMC) building.

![Figure 1: Project location and construction progress, status Oct. 5th 2009][1]

The horseshoe-shaped tunnels are generally 9 meters wide, 6.8 meters high and enlarged at the southern South Bound (SB) and northern North Bound (NB) portals. The SB and NB tunnels are approximately 18 meters apart. There are nine cross passages for pedestrians, an emergency vehicle cross passage, three equipment chambers with emergency accesses to the main tunnels. The tunnels are vented by jet fans and have lighting, fire protection, and operation and control systems.

The project site is located on the central California coastline west of the central San Francisco Bay in a mountainous terrain and landslide complex that forms the seaward termination of the northwest-aligned San Pedro Mountain ridge. The site is about 2.8 km east of the offshore trace of the San Gregorio fault and 7.2 km west of the surface trace of the San Andreas fault.
project will be constructed in a mountainous, highly deformed terrain of Mesozoic-age crystalline igneous (granitic-like) rock overlain by a series of thrust zones consisting of granitic rock and late-Cretaceous- and early-Tertiary-age clastic sedimentary rock. The stratigraphy of the site is shown on the longitudinal section along the tunnel alignment in Figure 2. It consists generally of Mesozoic granitic basement rock and a structurally overlying complex of early Tertiary clastic sedimentary rocks.

For purposes of reference and description, a system of three block units for the sedimentary rocks and the granitic basement unit is used: “South” (granitic rock), “Central,” and “North” Block units. The South Block is located between the South Portal and Fault B. It consists entirely of crystalline igneous rock. Three major lithology types are found within the Central Block unit: sandstone, conglomerate, and siltstone/claystone. Petrographic analyses indicate evidence of alteration for all rocks, likely by hydrothermal activity or weak cataclasis. The stratigraphy of the North Block consists of an overturned sequence of wellbedded claystone, siltstone and sandstone. Rocks at the North Portal consist primarily of fine-grained sandstone with claystone and siltstone with a lot of unfavorably intersecting fault zones.

2 NATM DESIGN FEATURES

The design and construction of the tunnels is based on the principles of the New Austrian Tunneling Method (NATM). Depending on ground conditions along the alignment, the initial support system may include shotcrete, rock dowels, lattice girders, spiles, and grouted steel pipes in various combinations. The tunnels at Devil’s Slide consist of a double shell lining system with an initial lining and a final lining separated by a waterproofing and drainage system. The final lining will be installed once the rock deformations have ceased. The initial lining is designed to support the rock loads that develop during stress redistribution upon excavation. It is assumed that the initial lining support will deteriorate with time and the final lining is designed to carry the entire static and seismic rock load.

Tunnel design is governed by the fact that “Rock masses are so variable in nature that the chance for ever finding a common set of parameters and a common set of constitutive equations valid for all rock masses is quite remote.” [3]. Therefore it has to be taken into account that, prior to tunneling, any design represents a prognosis which is either verified on site in case all assumptions are confirmed or which has to be adjusted to actual conditions. Design criteria are based on the principles of the New Austrian Tunneling Method (NATM). It is assumed that the initial support elements act as a ring-like support structure.

The first step is to establish geological data in sections along the tunnel profile with consistent characteristics and summarizing geological series with similar mechanical properties. Further, the boundary conditions such as virgin stresses, size, shape and orientation of the opening have to be taken into account in order to establish a possible failure mechanism, thereby establishing the behavior of the opening. Different failure mechanisms require different support measures as well.
as models of analysis to design the support measures. In order to simplify procedures at the site, support categories are established which are applicable for the various types of behavior of the opening. Finally, criteria for the application of support categories have been defined.

3 DESIGN PROCEDURE

Analysis and design of the initial support is controlled by the rock mass behavior during excavation. The rock mass behavior is governed by the type and installation sequence of the various initial support members. The type of analysis depends on the potential failure mode during excavation. The following procedure is used to determine the required initial support:

- The tunnel tubes are divided into sections of similar rock mass characteristics based on the consistent geologic and geotechnical data such as lithology, properties of intact rock, and rock discontinuities.
- For each rock mass type (ten rock mass types were defined for Devil's Slide), the rock mass parameters, such as compressive strength and deformability are obtained.
- The boundary conditions of the excavated tunnel have been evaluated to determine the rock mass behavior, including the potential failure mechanisms. The factors to be considered for analyzing the rock mass behavior are the virgin (primary) stress field, the ground water conditions, the orientation of the tunnel related to the rock mass structure and the dimension / form of opening.
- The failure mechanism is differentiated by the following failure modes: failure of rock blocks, fracturing induced by stresses and/or discontinuities, progressive failure induced by stresses, failure induced ahead of tunnel face, failure of tunnel face.
- Depending on the potential failure modes, one or more of the following methods of analysis are selected: Key Block Theory, Finite Element Model and Slope Stability Models for face stability.
- Based on the analyses, individual support categories have been defined for relevant tunnel sections. Support elements of five support categories for heading, bench and invert excavations have been summarized as indicated in following table for the top heading:

<table>
<thead>
<tr>
<th>Support Category</th>
<th>Typical Heading Advance</th>
<th>Shotcrete Thickness</th>
<th>Length of Rock Dowels</th>
<th>Spacing of Lattice Girders</th>
<th>Spiles</th>
<th>Face Dowels Length</th>
<th>Face Shotcrete Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>2.2 m</td>
<td>100 mm</td>
<td>3.6 m</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>II</td>
<td>1.6 m</td>
<td>200 mm</td>
<td>4 m</td>
<td>1.6 m</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>III</td>
<td>1.2 m</td>
<td>250 mm</td>
<td>6 m &amp; 4 m</td>
<td>1.2 m</td>
<td>4 m</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>IV</td>
<td>1.0 m</td>
<td>300 mm</td>
<td>6 m</td>
<td>1.0 m</td>
<td>4 m</td>
<td>n/a</td>
<td>50 mm</td>
</tr>
<tr>
<td>V</td>
<td>0.9 m</td>
<td>300 mm</td>
<td>6 m</td>
<td>1.0 m</td>
<td>Umbrella</td>
<td>9 m</td>
<td>100 mm</td>
</tr>
</tbody>
</table>

Table 1: Support elements for 5 categories – heading.

- Expected rock mass behaviors serve as the basis for design of the initial support elements of each of the five support categories. Selected rock mass configurations, for which failure along discontinuities or loosening of blocks are decisive, are used to assess the loading of rock dowels. In case of potential failure of the rock mass initial lining deformations are calculated for the relevant support categories. For the application of the relevant support
measures during construction criteria have been developed in accordance to the rock mass behavior, see Figure 3. Warning and alarm levels are applied according to the stability analyses provided.

- During construction, geotechnical monitoring is carried out to verify the designed support categories (i.e. observational method). The following monitoring and observations will be carried out: behavior of the opening by surveying the predefined deformation points, rock mass behavior by extensometer measurements, rock dowel performance by measuring devices attached to special dowels, stress measurements in the shotcrete lining and/or at the interface between rock mass and shotcrete lining using pressure cells.

<table>
<thead>
<tr>
<th>SUPPORT CATEGORY</th>
<th>ROCK MASS BEHAVIOR</th>
<th>APPLICATION CRITERIA</th>
<th>ALARM LEVEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>ROCK MASS BEHAVIOR TYPE 1: FAILURE OF ROCK BLOCKS, (BLOCKY GROUND)</td>
<td>AVERAGE SPACING OF DISCONTINUITIES VARIES IN THE RANGE OF 10 TO 50 CM WITH AN AVERAGE CONSISTENCY OF LESS THAN 6 M. THE EXPECTED MAX. BLOCK HEIGHT IS 2.4 M.</td>
<td>CRACKS APPEAR IN SHOTCRETE</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUPPORT CATEGORY</th>
<th>ROCK MASS BEHAVIOR</th>
<th>APPLICATION CRITERIA</th>
<th>ALARM LEVEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>IV</td>
<td>ROCK MASS BEHAVIOR TYPE 3, 4: PROGRESSIVE FAILURE INDUCED BY STRESSES AT ALL EXCAVATED FACES (FAST RAVELING OR RUNNING GROUND)</td>
<td>EXPECTED ROOF SETTLEMENT 40 MM</td>
<td>ROOF SETTLEMENT 60 MM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EXPECTED DIFFERENTIAL SETTLEMENTS OF ROOF TO INVERT 12 MM</td>
<td>DIFFERENTIAL SETTLEMENTS OF ROOF TO INVERT 20 MM</td>
</tr>
</tbody>
</table>

Figure 3: Examples for Application of Support Categories (Support Category I and IV)

4 DESIGN VERIFICATION PROCESS

4.1 General

NATM support measures generally include standard support categories which correspond to anticipated rock mass behavior. Contingency measures are used supplementary as required locally for specific ground conditions or if lining deformations exceed predefined warning levels. The support measures shall provide sufficient flexibility during construction to allow adaptation to actual conditions. Appropriate selection of support categories during construction requires comparison and matching of encountered rock mass behavior to anticipated behavior. In the course of the design verification process the support measures have to be continuously reviewed and updated by comparing the original prognosis and real behavior (actually encountered deformations and support as built). Such an interpretation of data according to the Observational Method is the main key to achieve the goal of an appropriate design verification process. Two special cases are reported hereafter.

4.2 South Block: Support Category II adaptation

Based on field data (face mapping) the requirement had to be analyzed to change the spacing of bolts from 1.6 m to 2.0 m spacing. Hence, block stability calculations have been performed for the south block excavation chainage 0 – 48 (NB) in order to adopt Support Category II. The results of the face maps analyses yield to 6 dominated discontinuity orientations (see Figure 4). These
different orientations are used to carry out block stability calculations for different discontinuity combinations.

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip Direction</th>
<th>Dip Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint Set 1</td>
<td>195</td>
<td>73</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>275</td>
<td>65</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>230</td>
<td>70</td>
</tr>
<tr>
<td>Joint Set 4</td>
<td>125</td>
<td>68</td>
</tr>
<tr>
<td>Joint Set 5</td>
<td>320</td>
<td>55</td>
</tr>
<tr>
<td>Joint Set 6</td>
<td>040</td>
<td>75</td>
</tr>
</tbody>
</table>

Legend: Red lines: relevant discontinuity sets, green dashed line: tunnel axis, blue dashed line: orientation of tunnel face

Figure 4: Results of face mapping with 6 dominated discontinuity orientations TM 0 – 48 (NB)

To determine the required dowel system a dowel pattern with 4.0 m long dowels in a distance of 1.5 m between the dowels in radial direction and 2.0 m between the dowels in longitudinal direction has been used. The calculations were carried out using standard SWELLEX dowels with a steel capacity of 100 kN. For bond strength 130 kN/m were chosen. Shotcrete is not considered in this analysis because of the short-term loading when the shotcrete achieves only low load bearing capacity.

Figure 5: Example of an Unwedge “rock wedge geometry”, calculation TM 0 – 48 (NB)

Using the key block theory and 6 relevant joint sets 20 potential wedges are formed around the tunnel opening including 4 wedges at the tunnel face. Since the analyses is carried out for the standard cross section the dowel requirements is investigated for all relevant key blocks. The assumption for the block size is defined by two rules: the apex height is limited to 6.0 m and the block z-length is set to 2.50 m. An example of block stability analysis is given in Figure 5.

It can be concluded that for geological conditions as observed in the area of chainage 0 – 48 Support Category II can be adapted to Support Category II-A:
4.3 Central Block: Support Category IV adaptation

The top heading on NB excavation has been driven employing ground Support Categories III and IV. Roof settlements between Tunnel Meter 524 to Tunnel Meter 571 have exceeded the warning and alarm levels for category IV due to the presence of a shear zone across the tunnel section. Support categories have been adjusted based on field observations, therefore the calculations from the design stage are not applicable. The existing safety margin has been defined by conducting back calculations. Deformations during NB-top heading excavation as measured in various sections can be summarized as follows:

- roof settlement of approx. 50 mm,
- top heading footing settlements of approx. 80 mm,
- top heading footing horizontal deformation of 80 mm.

Additional deformations during NB-bench excavation can be summarized as follows:

- roof settlement +10 mm,
- top heading footing settlements +20 / +40 mm (anisotropic),
- top heading footing horizontal deformation +30 mm.
The subsequent excavation of bench of the SB tunnel influenced the deformations in the NB tunnel to a minor degree. The additional deformations were less than 5 mm.

Back analyses were aimed to define stresses in the shotcrete lining of top heading, bench and invert to allow determination of the safety margin and to evaluate rock loads to be applied to the inner (final) cast in place lining. The geotechnical model is based on the available geological information between TM 500 – 600.

The Finite Element Method is used for the back analysis. Construction stages are considered according to sequencing illustrated in Figure 6. A two dimensional plane strain FE - model is applied. The dimensions of the Finite Element mesh are selected in a way that far field effects of the excavation on the boundaries of the system are negligible. The mesh above the tunnels extends 50 m above the crown and 27.5 m below the invert. The full overburden of 145 m is simulated with application of additional stresses at the top of the FE-model of 95 m * 24 kN/m³ = 2280 kN/m². The horizontal primary stress field is derived by variation of the K0-value to fit the horizontal displacement measurements of the shotcrete lining (K0-value of 0.9 has been analyzed). The failure mode used is Mohr Coulomb with linear elasticity and perfect plasticity. The thickness of the shear zone has been assumed to be 2 m with an angle of inclination of the shear zone equal to 45° to horizontal level (see Figure 7). A variation of the location of the shear zone has been investigated during an iterative numerical process to fit anisotropic deformation measurements.

Geotechnical Properties are indicated in Figure 6 distinguishing between:

- The silt-claystone (SH-2) with slightly higher joint spacing compared to the GBR (>10cm); GSI as per design (20-30) and no conglomerate interbeddings.
- Conglomerate (C-1) according to GBR with slightly higher joint spacing (>50cm); GSI slightly better (50-60).
- The sandstone (SS-2) as well as the granitic rock (G2) not affecting the tunnel excavation itself.

The FE - back analysis confirms that excavations of NB and SB are sufficiently stabilized by the support measures installed. Deformations measured could be reproduced in calculations within
the variations according to local ranges of geological conditions. By means of profile measurements of the intrados of the shotcrete lining it is to be checked if the required thickness of the final lining is achieved all around. Rock loads for the final lining have been determined on the basis of assumptions that the shotcrete lining fully deteriorates.

5 CONCLUSIONS

During construction of the Devil’s Slide Tunnel using the NATM excavation method a continuous verification process takes place by the designer’s representative, guaranteeing that support measures are reviewed and updated as required. This methodology enables both, reaction of the design team as soon as possible in case of unexpected geotechnical conditions and allowing optimization of construction to achieve cost and schedule efficiencies. The paper presents two case studies at which the interpretation of data according to the observational method is the main key for construction.

REFERENCES

[1] project webpage: www.dot.ca.gov/dist4/dslide

[2] GEOTECHNICAL BASELINE REPORT. Devil’s Slide Tunnel Project for the California Department of Transportation, Prepared by: HNTB Corporation, ILF Consultant Engineers and Earth Mechanics, Inc., October 14, 2005


