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Examination of Shotcrete Liner at Devil's Slide Tunnel Utilizing ASTM 1550 Field Test Results and Back Analysis

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ABSTRACT: The Devil's Slide Tunnel project, located south of San Francisco along Highway 1, consists of twin bore tunnels approximately 1250 meters long. The tunnels are currently being excavated and supported utilizing the "New Austrian Tunneling Method" (NATM). In NATM design a flexible initial lining is used to allow some deformation to occur to mobilize the strength of the rock. The initial lining support utilizes fiber reinforced shotcrete (FRS). The ASTM 1550 Round Determinate Panel Test "Pizza Test" is being conducted on site to ensure the flexural properties or post-crack performance of the FRS. However, the ASTM 1550 does not analyze shear failure due to ground loads imposed on the liner. Therefore, it must be coupled with typical compressive strength testing. Furthermore, measured convergence during excavation presents the opportunity to back calculate and analyze the in situ loading of the FRS liner for a better understanding of its actual performance. In this paper, the to date results of the ASTM 1550 field test program along with a back analysis based on measured convergence to determine the loading of the FRS initial lining will be presented. A brief discussion of the ASTM 1550 testing and the back calculated in situ loading of the liner and how these demonstrate the overall performance of the FRS liner at the Devil's Slide Tunnel will be given.

1. INTRODUCTION

The ongoing excavation at the Devil's Slide Tunnel Project of two 1250 meter tunnels each with a profile of 80 m^2 is being conducted utilizing the "New Austrian Tunneling Method" (NATM). The Tunnels are located along the pacific coast just south of Pacifica, CA, a suburb of San Francisco. The Tunnels will serve as a bypass for a landslide prone section of California's famed Highway 1.

1.1. Geological Setting

The tunnel runs north-south through the San Pedro Mountain ridge which is part of the Santa Cruz Mountains. The tunnel lies within the San Andreas Fault system and is 7.2 km west of the surface trace of the San Andreas Fault and 2.8 km east of the surface trace of the San Gregorio Fault. The 10 km strip between these two tunnels is referred to as the La Honda structural and terrain block [1,2,3].

Locally, the tunnel is divided into three blocks representing different geological conditions: south block, central block, and north block (Fig. 1) [1,4].

The south block consists of Mesozoic aged granodiorite and quartz diorite. The South Block also contains many local shear zones and a low angle thrust fault (Fault A). The south block ends at Fault B, which is the southern border of the central block.

The central block consists of interlayered late Cretaceous and early tertiary aged conglomerate, sandstone, siltstone, and claystone. The conglomerate and sandstone range from massive to thinly bedded, while the claystone and siltstone tend to be interbedded with sandstone layers and blocks. The bedding in the central block dips 20 to 40 degrees towards the northeast. The central block contains a shear zone near fault B referred to as fault 02-5 zone. At the northern end of the central block another highly sheared zone associated with fault C marks the end of the central block.

The rock in the north block is similar to the central block except that the bedding tends to be thinner and dipping steeply to the south and north. The north block begins with the fault C shear zone and continues to the north portal through several smaller shear zones.

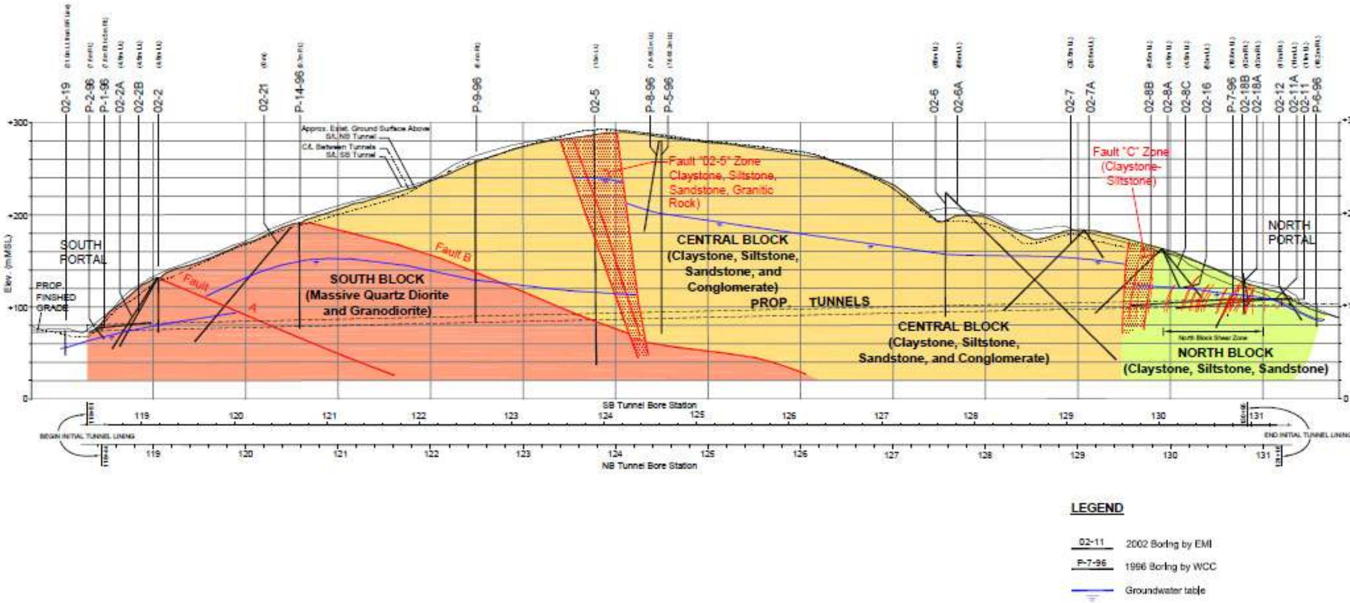

Fig. 1. Longitudinal profile along tunnels, showing South Block, Central Block, and North Block along with expected faults [4].

1.2. NATM Construction

The Devil's Slide Tunnels were designed to be excavated and supported utilizing the NATM methodology to help deal with the variable and difficult ground conditions expected. The basic concept of hard rock NATM is to utilize a thin flexible initial lining system that allows for some movement of the rock [1,5]. This movement mobilizes the strength of the rock and permits the rock to carry a portion of the load depending upon the quality of the rock.

NATM tunneling also allows the design to be optimized by defining categories of support that are based on the observed ground conditions and behavior [1,6,7]. Convergence readings, geological mapping, groundwater data, and other observations are utilized to make onsite decisions on the support category that should be applied. The Devil's Slide Tunnel has five support categories defined in the design and has utilized 4 of these categories to date during construction [1,8].

Several design conditions are essential in achieving desired results in NATM tunneling. First, the shape of the excavation must allow the rock to form an arch so that the full strength of the rock can be utilized. Therefore, a circular or oval shape is almost always used in NATM tunneling. Second, the liner system must be able to protect the rock from raveling or falling out and thus reducing its load bearing capabilities. Last, the liner system must be flexible enough to allow the needed movement and also be strong enough to prevent excessive deformations.

2. FRS LINER AT DEVIL'S SLIDE

2.1. Specifications and design

Fiber reinforced shotcrete was specified to be used at the Devil's Slide Tunnel to create a flexible liner along with the use of rock dowels and steel lattice girders. The use of steel fiber or synthetic fiber was left to the discretion of the contractor. The contractor in this case opted to use synthetic fibers.

The thickness of the shotcrete liner varies for each category and is applied in one or more applications as defined in Table 1. The liner is thicker in higher categories where more deformation and higher liner loading is expected due to poor rock conditions. Categories I and II are expected to see minimal deformations with a design tolerance of 30 mm [8]. Category III, IV and V which are defined as squeezing conditions have alarm levels at 30, 60, and 140mm respectively and design tolerances at 50, 80, 180mm respectively [1,8].

Table 1. Shotcrete thicknesses per Support Category [8].

CAT	Total, mm	Flash, mm	1st App., mm	2nd App., mm
I	100	0	0	100
II	200	25	150	25
III	250	50	150	50
IV	300	100	150	50
V	300	100	150	50

-Flash is applied right after excavation to keep the ground from raveling and make the excavation safe for workers.
-1st application occurs after the girder is placed and before the dowels are placed.
-2nd application occurs after dowels and pre-support measures such as spiles or canopy pipes are installed. This application occurs 2 to 3 excavation rounds from the face except in the case of CAT I where it occurs in the actual excavation round.

2.2. *Equipment and Application*

The contractor utilizes a Meyco Potenza Shotcrete Robot (see Fig. 2) to apply the FRS liner in the tunnel. The shotcrete robot has an on-board compressor, pump unit, and programmable accelerator dosing system. The robot also allows the nozzleman to be under supported ground at all times.

Fig. 2. Operator shown controlling the nozzle boom of the Meyco Potenza Shotcrete Robot utilized at Devil's Slide Tunnel.

2.3. *Testing program*

The shotcrete testing program at Devil's Slide Tunnel includes coring from a test panel and testing 1-day, 7-day, and 28-day compressive strengths [9]. The coring occurs onsite and the testing is performed both onsite and offsite in an independent lab. The results of the onsite compressive strength testing (ASTM C1140 [10]) are shown in Table 2.

Table 2. Shotcrete strength per curing time

Cure Time, days	Average compressive strength to date, MPa	Specified Strength Requirements, MPa [9]
1	14.1	9.8
7	33.9	22.1
28	47.4	28

In addition to compressive strength testing, ASTM 1550 Round Determinate Panel testing is performed per specification to test the flexural toughness of the shotcrete [9]. This testing is performed exclusively on site. The specified flexural toughness requirement is 320 joules at 7-days [9] and the results of the testing thus far will be discussed in the next section.

3. ASTM 1550 TESTING

The ASTM 1550 test (Round Determinate Panel-RDP Test) is used to determine the flexural toughness of fiber reinforced shotcrete [11]. The test was developed in 1998 by Bernhard [12,13] and is comparable to other flexural test methods such as the beam test [14]. In the last ten years since the RDP test was first devised it has been used in both in tunneling and mining industries [15,16].

The test comprises the making of a 75mm by 800mm round shotcrete panel (Fig. 3), which is tested in a special apparatus that applies a strain controlled load in the center of the panel while the panel is supported by three pivot points at equal distances around the perimeter of the sample (Fig. 4).

Fig. 3. Round Panel sample after testing.

Fig. 4. ASTM 1550 test machine at Devil's Slide Tunnel with sample ready to be tested.

The round determinate panel test does not analyze the behavior of the shotcrete liner in the classical shear failure mode that would occur due to ground loads imposed on the shotcrete liner. Instead, the test looks at the flexural strength or toughness. The toughness is important when evaluating failure modes that would create a flexural or bending type crack as seen in Fig. 5 during the testing of a round shotcrete panel. Examples of such failure mechanisms are a loosened block acting as a point load or a zone of weak rock producing a bagging of the ground support [16] (Fig.6). Resisting these types of failure mechanisms allows the shotcrete liner to perform its function of keeping the rock from raveling and blocks coming loose.

Fig. 5. Round panel being tested showing the flexural crack and the exposed synthetic fibers continuing to carry load.

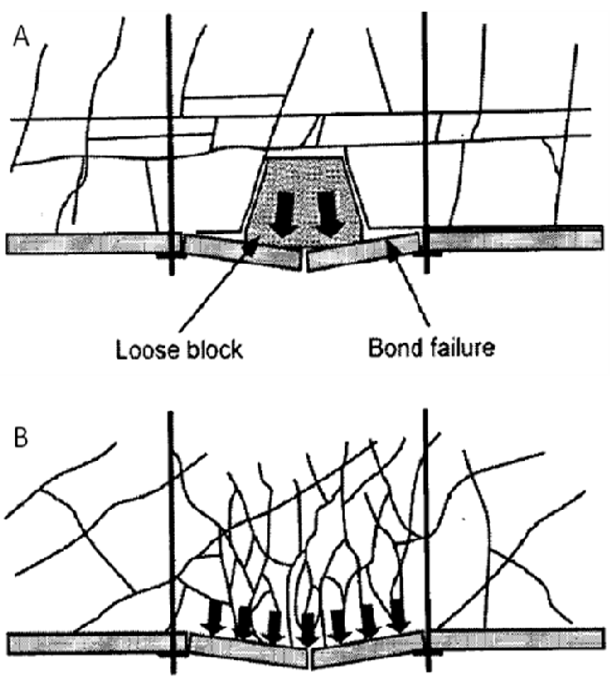

Fig. 6. Flexural failure modes due to A) loose block and B) weak rock zone from Martin et al. [15]

The flexural strength or toughness of the panel is determined by calculating the area beneath the load vs. deflection curve which is measured in joules and is referred to as the absorbed energy. Fig. 7 shows a typical load vs. deflection curve for shotcrete with synthetic fibers like at Devil's Slide. As shown in the figure, the deflection is carried out to 40 mm at a constant strain rate. The flexural crack appears at a very small deflection and the load bearing capacity of the panel reduces to approximately half as the load is now primarily carried by the fiber-shotcrete interaction. After the first crack and initial load reduction, the load bearing capacity slowly decays as the crack widens and more fibers tear or are pulled out. As seen in the Fig. 3 three flexural cracks will occur starting in the middle and extending radially between the three reaction points around the perimeter of the round panel. Any test panel that only develops two cracks is considered an invalid test. A total of three panels are tested with at least two of the tests needing to be valid. The average of at least two tests is the energy absorption reported.

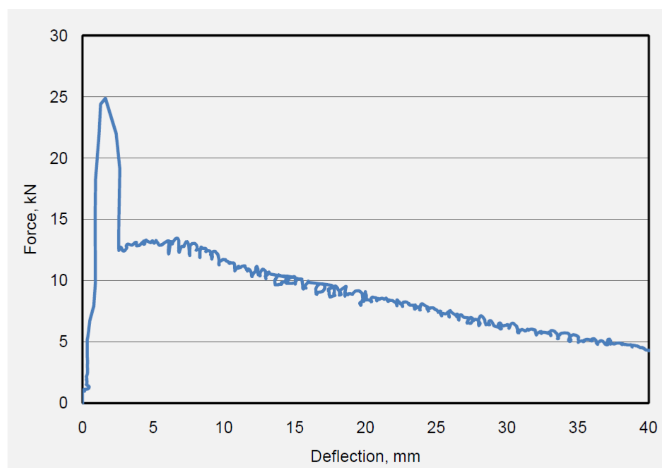

Fig. 7. Typical load vs. deflection curve for the synthetic fiber reinforced shotcrete at Devil's Slide.

3.1. Set up at Devil's Slide

The ASTM 1550 testing is performed utilizing an onsite testing facility design and built exclusively for testing the large round panels (Fig. 8). The facility consists of a curing room equipped with a conveyor system designed to store and allow the panels to be easily moved (Fig. 9). The panels (Fig. 10) are shot in the tunnel during actual shotcrete application. Three panels are typically shot at a time along with a square panel for core testing. The specimens are left in the tunnel for at least 24 hrs, before being removed to the onsite curing facility to be stored prior to testing.

The specimens are tested utilizing a custom made testing device that utilizes a PLC and data logger. The data is taken from the logger and is processed utilizing EXCEL.

Fig. 8. Devil's Slide Tunnel Project onsite shotcrete and concrete testing facility.

Fig. 9. Curing room with conveyor system for storage and handling of panels.

Fig. 10. Panel forms used at Devil's Slide for ASTM 1550 test.

3.2. *Results to date*

Fig. 11 shows the results of all the ASTM 1550 tests performed during the tunnel excavation. The running overall average is also displayed. The results show that the majority of the tests are above 320 joules with the overall average showing a lot of improvement at the beginning of the excavation and then leveling off at about 370 joules.

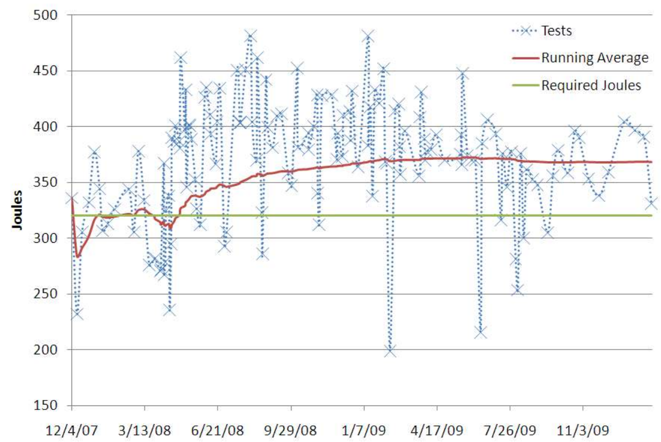

Fig. 11. Results of onsite ASTM 1550 testing at Devil's Slide Tunnel showing individual tests and the running total average.

The onsite testing program has been a challenging process due to the large size of the specimen and the many variables that can come into play when creating and storing the panels [17]. The size of the panels makes it difficult to store and move the panels. Special care must be taken to ensure the panels shot in the tunnel are in a location where they will not impede production and will not be disturbed. It has been noted that if the panels are handled too early during the curing process that the results can be drastically reduced. The transporting of the panels can be difficult and dangerous due to the size and weight of the specimen. Attention must given to the means of transporting the specimen to limit disturbance and to limit the possibility of a panel falling or striking any personnel. The panels also need to be stripped, the stripping process can lead to sample disturbance as well.

The shooting of the panels in the field has many variables that can have a big affect on the results. Some of these variables are the accelerator dosage, closeness of nozzle to panel, and finishing of panel. When the accelerator is high the panels are difficult to finish and brittle, when the accelerator is too low the shotcrete slumps in the panel which is laid at 45º during shooting. Since the panels are thin and the shotcrete is shot onto a flexible plywood backing, if the nozzle is too close to the panel it will likely cause many fibers to separate out giving a low fiber count and poor results. The finish of the panel can affect the strength of the sample and also make the sample out of specification.

In an earlier study [17] it was observed that a failing round panel test did not necessarily correspond to a failing compressive strength test. Therefore, a failing round panel test was rarely an indication of poor quality shotcrete, but was usually due to one of the above factors. It has also been shown as the shooting, transportation, and curing of specimen is closely observed and monitored the desired results are more easily obtained and repeated.

The running average in Fig. 11 indicates that there is a learning curve to producing consistent results when testing round determinant panels onsite. Therefore, results of the test being performed onsite should take into consideration the factors discussed above.

The overall ASTM 1550 testing program indicates that the shotcrete is performing up to the specified flexural toughness. This can also be verified in the field since the shotcrete has performed very well in resisting failure mechanisms that create flexural or bending cracks.

4. BACK ANALYSIS

The compressive strength testing program ensures that the shotcrete maintains the design strength throughout the project. As shown earlier in Table 2 this testing has shown that the shotcrete strength exceeds the specified strengths. However, the NATM tunneling method facilitates an extensive deformation monitoring program referred to as convergence reading [1,7]. This convergence reading coupled with visual observation lends the opportunity to perform back analyses to further evaluate the strength performance of the shotcrete.

A back analysis was performed for this paper based on convergence measured during the NB top heading excavation in a CAT IV excavation, which includes a temporary invert during the top heading excavation. The back analysis was then used to estimate the axial and moment loads carried by the shotcrete liner after the stabilization of deformation. This location was chosen only as an example and is not necessarily representative of deformation in the tunnel as a whole.

Fig. 12 shows the convergence readings that were used in the back analysis. The readings show 15 to 20 mm of vertical and lateral deformation before the deformations stabilized within a week of excavation.

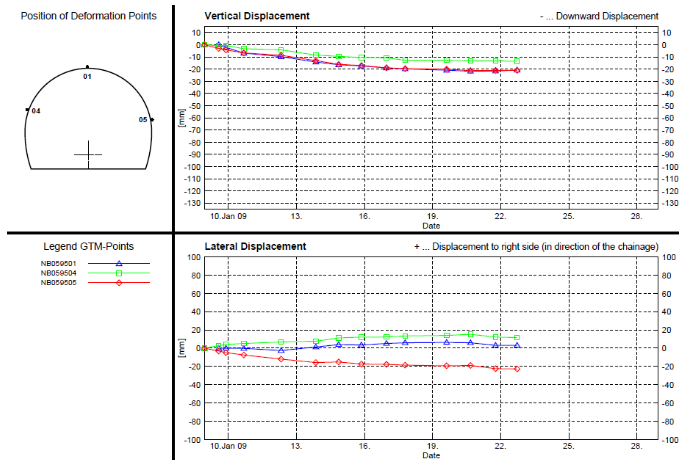

Fig. 12. Convergence measured during top heading excavation of the NB tunnel showing 15 to 20 mm of vertical and lateral displacement.

4.1. *Back Analysis Methodology*

The back analysis was performed utilizing the 2D finite element modeling software Phase2 [18]. The model was created using a plain strain analysis with the Mohr Coulomb failure criterion. The friction angle and cohesion utilized in the model take into consideration not only the properties of the rock but the rock mass behavior, which is influenced by the properties of the discontinuities present in the rock mass. Although the model is 2D, tunnel convergence is heavily influenced by the distance to the tunnel face. Therefore, to model this 3D affect with a 2D finite element code, stages were used along with a material softening approach. Stages were also used to model the construction of the temporary invert which was placed four meters behind the top heading face.

A total of five stages were utilized:

1. The first stage is used to initialize the in situ stress state prior to any excavation

2. The second stage is used to soften the top heading material to account for 3D affects allowing some deformation to occur before any lining is placed. In the field, 20 to 50% of the deformation can occur prior to the liner being placed and any convergence measurements being taken. The material softening is accomplished by reducing the rock elastic modulus. This reduction is typically between 40 and 60% [19]. For this model the modulus was reduced by 50%. It is assumed that the convergence up to this stage is not measured; therefore, only the deformation after stage 2 is compared with the actual convergence readings.

3. The third stage consists of the top heading being fully excavated and the shotcrete liner being placed along with the rock dowels. The rock dowels consist of grouted 4 and 6m dowels with

280 kN pullout capacity. The shotcrete liner is assigned early strength parameters along with a reduced elastic modulus which is typically 1/3 of the 28-day modulus [19]. During the third stage the invert material is softened as the top heading material was in stage 2.

4. The fourth stage consists of the invert being fully excavated and the shotcrete lining being placed in the invert. The shotcrete properties for the top heading are not changed.

5. The fifth stage is used to harden the shotcrete liners for both the top heading and invert. This step is not necessary to determine the loading in the liner due to the top heading excavation, but is an essential step to prepare the model for bench excavation if the bench is not be excavated close to the top heading which is typically the case in hard rock NATM tunnels (Fig. 13).

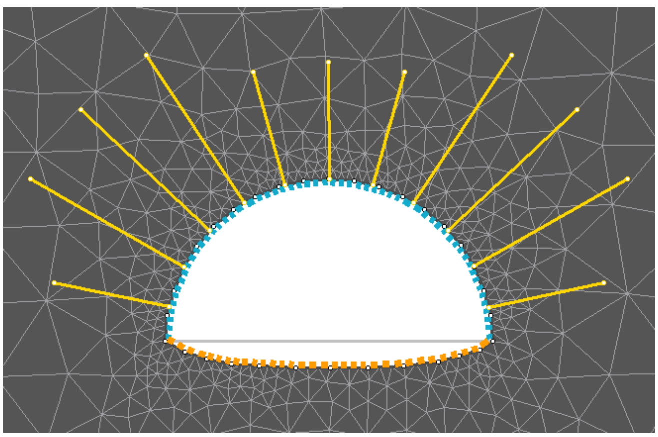

Fig. 13. Excavation and support after running through five stages of finite element model.

The rock type in the area of the convergence was mainly highly fractured and sheared gray to black siltstone/claystone with inter-layered sandstone as seen in face photo shown in Fig. 14. The ground type mostly fit the description of ground type SH2 as defined in the baseline report [4]. Therefore, the initial model utilized the design parameters given in the Geotechnical Design Report [20] for SH2 as given in Table 3.

The overburden in the area of the measured convergence was 185 meters. Therefore, the model was extended up to 185 meters above the tunnel so that the unit weight of the rock could be used to determine the in situ stress. The convergence occurred in the central block which consisted of shallow dipping lithology, however, no other rock types were implemented above the tunnel besides SH2. There are several reasons for this simplification. First the other rock types had similar unit weights. Next, the section of tunnel where the convergence occurred was a fairly thick section of similar material; therefore, the SH2 material would likely be 25 to 40 meters above the tunnel crown which is five to eight times the height of the top heading. Last, the rock was found to be highly folded and sheared making it difficult to determine lithology boundaries by projecting from known boundaries mapped in the tunnel.

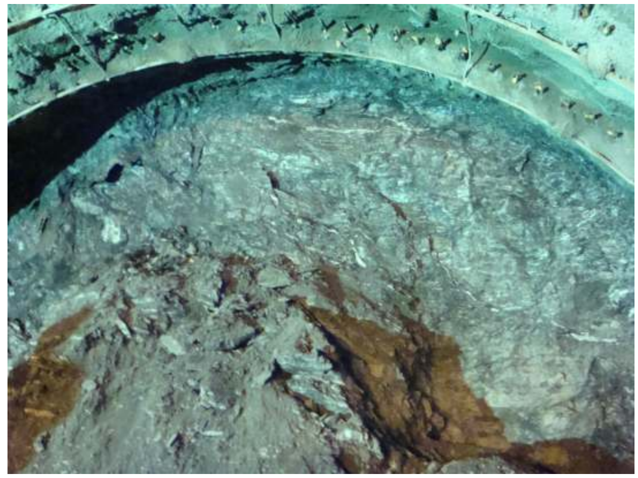

Fig. 14. Photo of an excavation face in the area of the measured convergence.

Table 3 gives the initial and final rock parameters used in the model. The rock parameters were all increased to achieve the actual measured deformation. The elastic modulus (E) was increased from 0.5 to 2.0 GPa along with increasing the friction angle (ϕ) from 22.5º to 25º and the cohesion (c) from 0.2 to 0.6 MPa. Besides preliminary modulus (E) tests in siltstone/claystone material during the geotechnical investigation where the intact rock modulus (E) ranged from 10 to 30 GPa [21], no testing has been performed to verify the increase in these parameters. However, the purpose of the model was to investigate the liner loading and thus matching the deformation was the main goal.

Table 3. Initial and Final Rock parameters used in the back analysis

Rock Parameters	Initial Model	Final Model
E, GPa	0.5	2
ν	0.25	0.25
γ, MN/m^3	0.026	0.026
ϕ, º	22.5	25
c, MPa	0.2	0.65
$\phi_{dilation}$, º	22.5	25
$\phi_{residual}$, º	22.5	22.5
$C_{residual}$, MPa	0.07	0.22

Table 4 gives the Shotcrete strength parameters for early strength and 28 day strength. The shotcrete early uniaxial compressive strength was set at 24 MPa which is the interpolated 3-day compressive calculated from the compressive strength data (see Table 2). The 3-day strength was chosen based on the timing of the support installation and the fact that the measured convergence occurred over the first week after excavation. The early strength elastic modulus used was 7 GPa which was 1/3 of the 28-day modulus of 21 GPa.

Table 4. Early Strength and 28-day Shotcrete strength parameters used in back analysis.

Shotcrete Parameters	Early Strength (Top Heading Stage 3 & 4, Invert Stage 4)	28-day Strength (Top Heading and Invert Stage 5)
E, GPa	7	21
ν	0.25	0.25
UCS, MPa	24	47

4.2. Analysis Results

The final vertical deformation at the crown was 26 mm, while the deformation after Stage 2 was 7 mm (27% of total) leading to 19 mm of measurable vertical deformation. The measurable lateral deformation was calculated at 17 to 18 mm in the location of the convergence points located in the side wall. The above values are within measuring accuracy of the actual measured deformations. Fig. 15 shows the excavation at Stage 5 with the computed σ_1 stress contours, stress trajectories, and the exaggerated deformation of the liner.

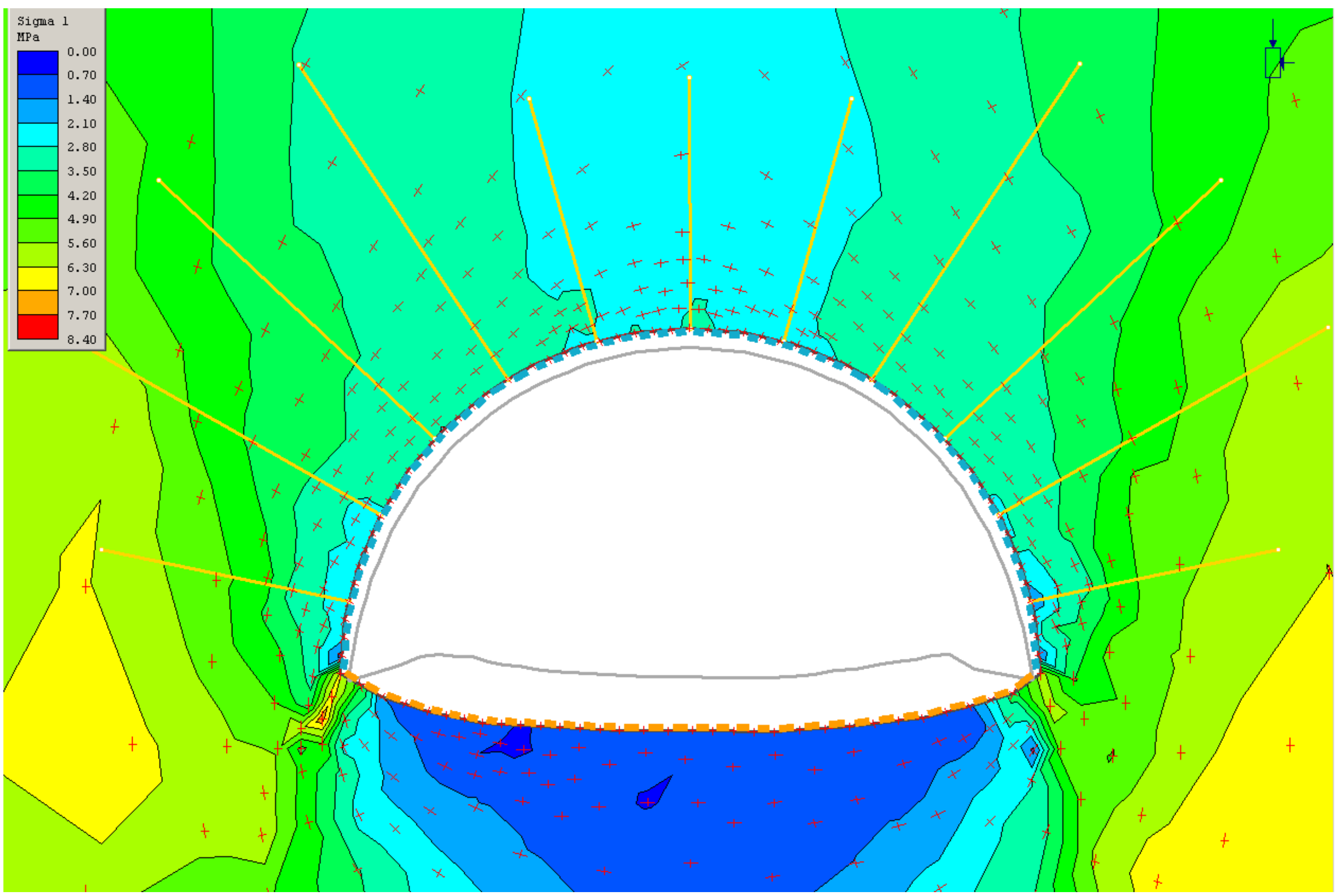

Fig. 15. Back analysis after Stage One showing σ_1 contours, stress trajectories, and exaggerated FRS liner deformation.

The FRS liner axial and moment loads computed in the back analysis are plotted on an M-N chart given in Fig. 16. The maximum axial load computed was 5.7 MN per unit width and the maximum moment was 0.096 MN-m per unit width. Using the interpolated 3-day strength, the liner was able to withstand this loading, which is verified by the fact that there were no visual signs of yielding in the actual liner during the deformation. The computed loads are the maximum likely loads that the FRS liner experienced given the elastic modulus that was used in the model. It is likely that the actual loading was less due to the following reason, a good portion of the deformation occurred while the FRS liner was one to two days old and the modulus during this time may have been less than the early strength modulus used in the model.

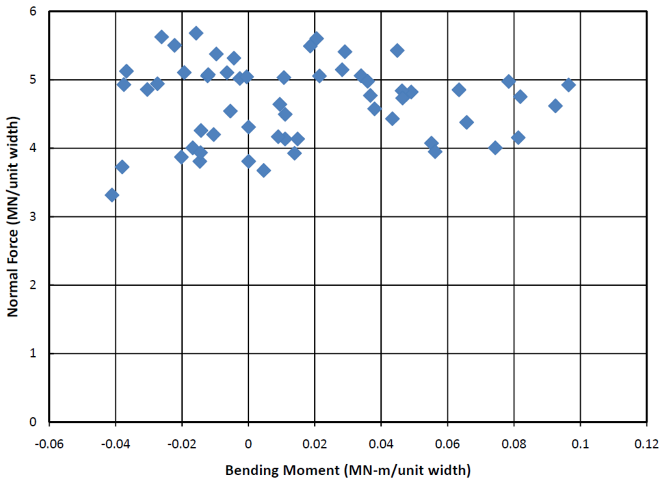

Fig. 16. M-N chart for the FRS top heading lining showing the axial and moment loading.

5. DISCUSSION OF FRS PERFORMANCE AT DEVIL'S SLIDE

- The compressive strength testing has shown that the FRS liner has more than adequate strength for early strength and long term strength requirements.
- The ASTM 1550 onsite testing, despite many challenges, has proven successful due to tight control on the process from shooting the panels to curing the panels to testing the panels.
- The ASTM 1550 testing has shown that the FRS liner at Devil's Slide Tunnel displays the specified flexural toughness.
- Visual inspection and experience with the FRS liner at Devil's Slide Tunnel presents no reason for doubt concerning the flexural toughness of the liner.
- Measured convergence throughout the tunnel has shown that the FRS liner has sufficient flexibility to allow the rock to deform within contract limits without excessive cracking and yielding.
- The back analysis further demonstrates the flexibility of the liner and the ability to carry loads due to deformation of the rock.

REFERENCES

1. Decker, J.B., and P.H. Madsen. 2009. Utilizing Convergence Reading to Determine Stability and Support Category in NATM Tunneling at the Devil's Slide Tunnel, Proceedings of the 43rd US Rock Mechanics Symposium and 4th U.S.-Canada Rock Mechanics Symposium, Asheville, NC, June 28-July 1, 2009.

2. Amini, M., M. John, H. Sander, and Y.N. Wang. 2005. Geotechnical Design of Devil's Slide Tunnel. In Proceedings of RETC 2005, Seattle, 27-29 June 2005, eds. J. D. Hutton and W.D., 859–870. Society for Mining, Metallurgy, and Exploration.

3. Earth Mechanics, Inc. 2003. Interpretative Geology and Seismicity Devil's Slide Tunnels Project. Prepared for Caltrans and HNTB Corporation.

4. HNTB Corporation, ILF Consultants, Inc., and Earth Mechanics, Inc.. 2005. Geotechnical Baseline Report: Devil's Slide Tunnels Project. Prepared for Caltrans.

5. Karakus, M., and R.J. Fowell. 2004. An Insight into the New Austrian Tunnelling Method. In Proceedings of ROCKMEC 2004-VIIth Regional Rock Mechanics Symposium, Sivas, Turkey, 2004.

6. Ashley, D.B., E.C. Tse, H.H. Einstein. 1979. Advantages and Limitations of Adaptable Tunnel Design and Construction Methods. In Proceedings of RETC 1979, New York, 1979, 989-1011, Society for Mining, Metallurgy, and Exploration.

7. Ugarte, E., V. Gall, and G. Sauer. 1996. Instrumentation and its Implications—DART Section NC-1B, City Place Station, Dallas, TX. North American Tunneling 1996, 503-512, Balkema, Rotterdam.

8. State of California Department of Transportation, 2006. Project Plans for Construction on State Highway in San Mateo County near Pacifica from 4.7 km to 1.0 km South of Linda mar Boulevard, Contract No. 04-1123U4.

9. State of California Department of Transportation, 2006. Notice to Contractors and Special Provision for Construction on State Highway in San Mateo County near Pacifica from 4.7 km to 1.0 km South of Linda mar Boulevard, Contract No. 04-1123U4.

10. American Society of Testing and Materials. 2003. Standard C-1140, "Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels", ASTM, West Conshohocken, PA.

11. American Society of Testing and Materials. 2005. Standard C-1550, "Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panels)", ASTM, West Conshohocken, PA.

12. Bernard, E.S. 1999. Correlations in the Performance of Fibre Reinforced Shotcrete Beams and Panels. Civil Engineering Report CE9, School of Civic Engineering and Environment, University of Western Sydney, July.

13. Bernard, E.S. 2000. Correlations in the Performance of Fibre Reinforced Shotcrete Beams and Panels: Part 2. Civil Engineering Report CE15, School of Civic Engineering and Environment, University of Western Sydney, June.

14. American Society of Testing and Materials. 1997. Standard C-1018, Standard Test Method for Flexural Toughness and First-Crack Strength of Fiber Reinforced Concrete (Using Beam with Third Point Loading), ASTM, West Conshohocken, PA.

15. Hanke, S.A., A. Collis, and E.S. Bernard. 2001. The M5 Motorway Tunnel: an Education in Quality Assurance for Fibre Reinforced Shotcrete, In Proceedings of the International Conference on Engineering Developments in Shotcrete, ed. E.S. Bernard, Hobart, Tasmania, Australia, April 2-4, 2001.

16. Martin, L., J. Seymour, C. Clark, R. Pakalnis, M Stepan, M. Roworth, and C. Caceres. 2010, An Analysis of Fiber-Reinforced Round Panel Strengths as Correlated to Wire Mesh Bag Strength, SME 2010, Phoenix, Arizona, February 28-March 3, 2010.

17. Madsen, P.H., J.B. Decker, K. Zeidler, V. Gall, and T.M. O'Brien. 2009. Experience with Synthetic Fiber Reinforced Initial Shotcrete Lining at the Devil's Slide Tunnel Project, Spritzbeton-Tagung 2009, Alpbach, Tirol, January 15 & 16, 2009.

18. Rocscience Inc. 2008, Phase^2 Version 6.028 - Finite Element Analysis for Excavations and Slopes. www.rocscience.com, Toronto, Ontario, Canada.

19. U.S. Department of Transportation Federal Highway Administration. 2009, Technical Manual for Design and Construction of Road Tunnels—Civil Elements, Publication No. FHWA-NHI-09-010, March 2009.

20. ILF Consultants, INC. 2005. Devil's Slide Tunnel: Geotechnical Design Report, Prepared for Caltrans.

21. Earth Mechanics, INC. 2003. Geological and Geotechnical Data Report: Devil's Slide Tunnels Project, Prepared for Caltrans and HNTB Corporation.