

LESSONS LEARNED FROM NATM DESIGN AND CONSTRUCTION OF THE CALDECOTT FOURTH BORE

Bhaskar Thapa ■ Jacobs Associates

Axel Nitschke ■ Gall Zeidler Consultants

Ivan Ramirez ■ Contra Costa Transportation Authority

Mike McRae ■ Jacobs Associates

Vojtech Gall ■ Gall Zeidler Consultants

ABSTRACT

The Caldecott Fourth Bore is a 15.2 m wide (50 ft), 1,036 m long (3,399 ft) highway tunnel with seven cross passages located on State Route 24 in the San Francisco Bay Area, excavated through weak, highly fractured, and sheared sedimentary rock formations. This paper describes key observations and lessons learned from the design and construction of this NATM (New Austrian Tunneling Method) tunnel, including:

- Predicted versus observed ground behaviors and support performance based on direct observation and measured convergences
- Variations in tunnel production rate by support category
- Contractual considerations regarding support selection criteria
- Installation and performance of a 52 m long (170 ft) pipe canopy
- Effects of tunnel construction on slope stability

INTRODUCTION

Project Description

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) proposed construction of a Fourth Bore that provides two additional traffic lanes to address congestion on SR 24 near the existing three Caldecott Tunnels. The length of the proposed Fourth Bore is 1,036 m (3,399 ft). The project includes 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 provides two 3.7 m (12 ft) traffic lanes and two shoulder areas that are 3 m and 0.6 m (10 ft and 2 ft) wide. The horseshoe-shaped mined tunnel is 15 m (50 ft) wide and 9.8 m (32 ft) high. A typical cross section of the tunnel is shown in Figure 1. The tunnel includes a jet fan ventilation system; a wet standpipe fire protection system; and various operation and control systems, including closed-circuit television (CCTV) monitoring, heat and pollutant sensors, and traffic monitoring systems.

In accordance with general Caltrans practice for "important" facilities on lifeline routes such as SR 24, the seismic design for the Fourth Bore 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

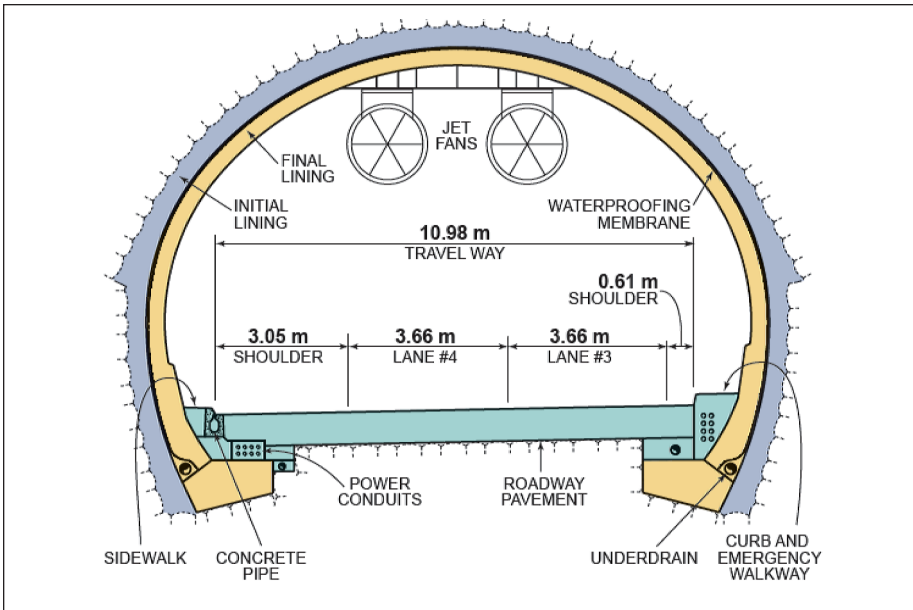


Figure 1. Typical cross section Caldecott fourth bore

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.

Ground Conditions

The geology of the alignment is characterized by northwest-striking, steeply dipping, and locally overturned marine and nonmarine 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 nonmarine claystone, siltstone, sandstone, and conglomerate of the Orinda Formation. Major formations and geologic units within these formations are shown in Figure 2. A summary of key properties of these formations is presented below, and further details on site investigations and ground characterization are described in Thapa et al. (2008a,b, 2009).

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 encountered 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 were encountered, such as smaller-scale faults, shears, and crushed zones. The active Hayward fault, located 1.4 km (0.9 mi) west of the Caldecott Tunnel, is the closest regional fault to the project site. Engineering characteristics of ground conditions along the alignment are described later in this paper.

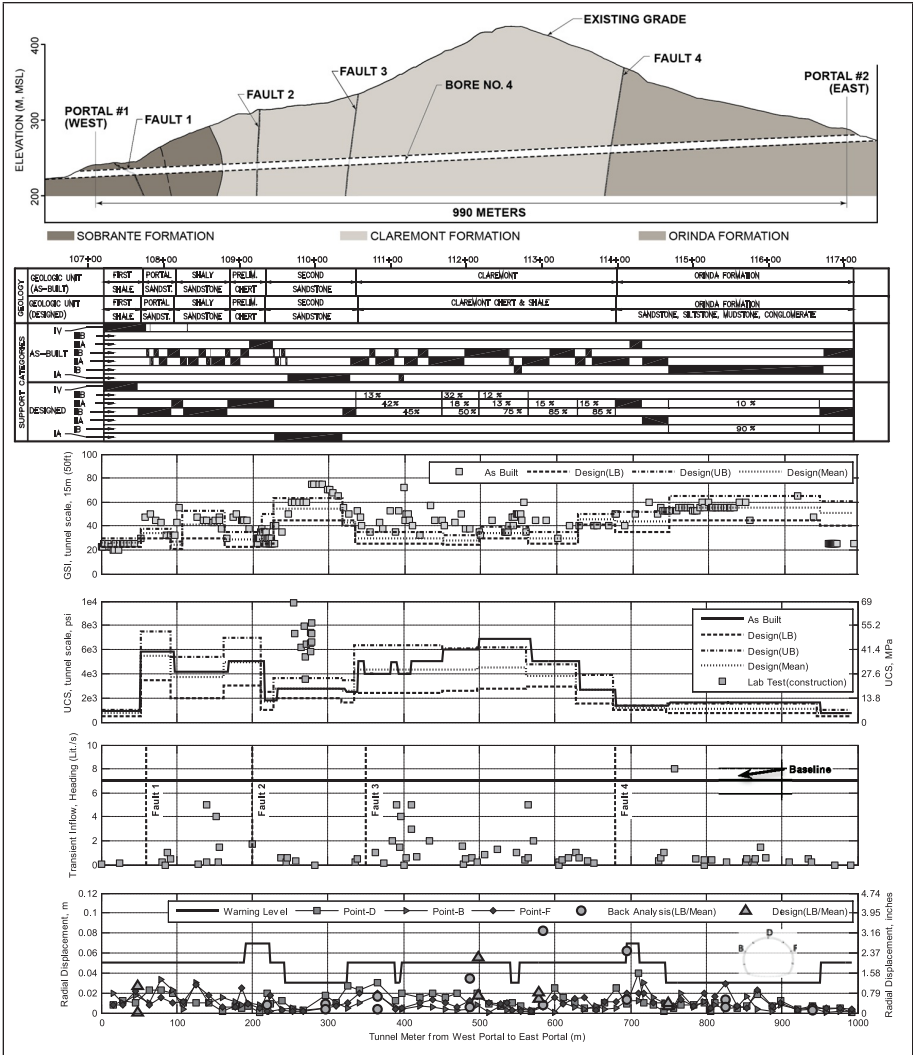


Figure 2. Longitudinal profile of design prognosis versus as-built ground conditions, support categories, and tunnel convergence (monitoring point location key shown looking at the heading for east and west portal drives)

Based on the occurrence of gas in the first three tunnels, the Fourth Bore was classified as a gassy tunnel by the California Occupational Safety and Health Administration, but was later reclassified to potentially gassy after breakthrough of the top heading.

OVERVIEW OF DESIGN AND CONSTRUCTION

A prescriptive approach to the specification of the excavation and initial support requirements was adopted to implement NATM construction of the Fourth Bore. Excavation and support requirements for each support category addressed the overall excavation and construction sequence—including restrictions on advance lengths, drift dimensions, arrangement and dimensions of support elements, as well as acceptable,

alternative schemes where applicable. The global construction sequence consisted of a top heading and bench, or top heading, bench, and invert excavation sequence. The initial support system comprised fiber-reinforced shotcrete; drill and grout, as well as self-drill and grout rock dowels; lattice girders; invert arch; and drill and grout, as well as self-drill and grout spiles, pipe canopy, fiberglass face dowels, face sealing shotcrete, and a sloped core as face support as an alternative to face dowels. The appropriate combination of these support elements was prescribed based on ground conditions and predicted ground behavior.

Design and payment of the NATM tunnel excavation and support was organized into: (1) standard support consisting of four major support categories and three subtypes, each having a separate pay item; and (2) 20 additional support elements (tool box items) on a unit-price basis (including time-dependent costs such as impacts on advance rates) to address local ground conditions/behaviors, as required. Table 1 summarizes the key support elements for the four major support categories, and Figure 3 shows the arrangement of support elements and support installation requirements for one of the support categories.

Additional support measures were supplementary to the standard support measures. These additional measures were required to address observed or measured local ground conditions or behaviors and were installed when, for example, the measured convergence exceeded warning levels or when specific ground conditions or support system behaviors were observed, as defined in the contract documents. Estimated quantities of additional support measures included in the contract were based on an assessment of variations in expected ground conditions based on the results of the site investigation program. Additional support elements included spiling, rock dowels, additional shotcrete lining or face sealing shotcrete, face dowels, lattice girders, and an invert arch. Other additional pay items included the drilling of additional probe and drain holes.

Standard support categories and additional support measures were designed to address seven anticipated ground behaviors (Table 2). These predicted behaviors at defined locations along the alignment were established using numerical analyses to evaluate forces, moments, and rotations in the shotcrete lining; forces in the rock reinforcement support elements; the stresses, strains, and associated displacements in the ground around the tunnel; as well as evaluations of possible block failure modes. In addition, the recorded behavior in the existing three Caldecott Tunnel bores (Thapa et al., 2007, 2008a,b, 2009) confirmed possible ground behavior modes—including running ground (23 cubic meters [30 cubic yards]) with groundwater inflow of 5 L/sec (80 gpm), block failure, caving in of ground above the tunnel crown (extending to the ground surface in one instance), and slaking. Each standard support category was designed to support a defined ground condition that, along with the in situ conditions, resulted in a combination of these anticipated behaviors; it is this combination of ground conditions/in situ conditions and ground behaviors that defines a ground class (GC). GC and SC typically had a one-to-one correspondence. Support application criteria were based on encountered ground behaviors, conditions, measured lining deformations, and observed support performance. Daily meetings between the contractor's and engineer's tunneling experts were used to discuss monitoring data and support performance and to decide upon the required excavation and support measures for the next 24 hours. The support category as well as additional support measures were typically proposed by the contractor and approved by the engineer.

The contract was advertised to bidders by the California Department of Transportation in May 2009 and was awarded to the low bidder, Tutor Saliba Corporation (TSC), on November 20, 2009. Tunnel construction was preceded by portal excavation and support, which began concurrently on the east and west sides of the alignment in May 2010. Canopy pipes were installed at both portals prior to start of tunneling. Break-in occurred in August 2010 at the east portal, and in March 2011 at the west

Table 1. Summary of support categories (SC)

Support Category	Max. Advance Length, m (ft)	Systematic Presupport Measure	Face Support	Min. Shotcrete Thickness, mm (in.)	Avg. Radial Dowel Spacing, m (ft)	Temporary Shotcrete Invert Arch
I	1.8 (6.0)	None	SC IA: face dowels/ sealing fiber reinforced shotcrete (FRS) as required SC IB: systematic face dowels/sealing FRS	203 (8)	1.8 m (6)	None
II	1.4 (4.5)	SC IIA: none SC IIB: spiles	Face dowels/sealing FRS or sloping core/ sealing FRS	254 (10)	1.5 m (5)	None
III	1.0 (3.3)	Spiles	Sloping core/sealing FRS	305 (12)	1.2 m (4)	SC IIIA: none SC IIIB: top heading and bench
IV	1.0 (3.3)	Pipe canopy	Sloping core/sealing FRS	305 (12)	None	Top heading and bench

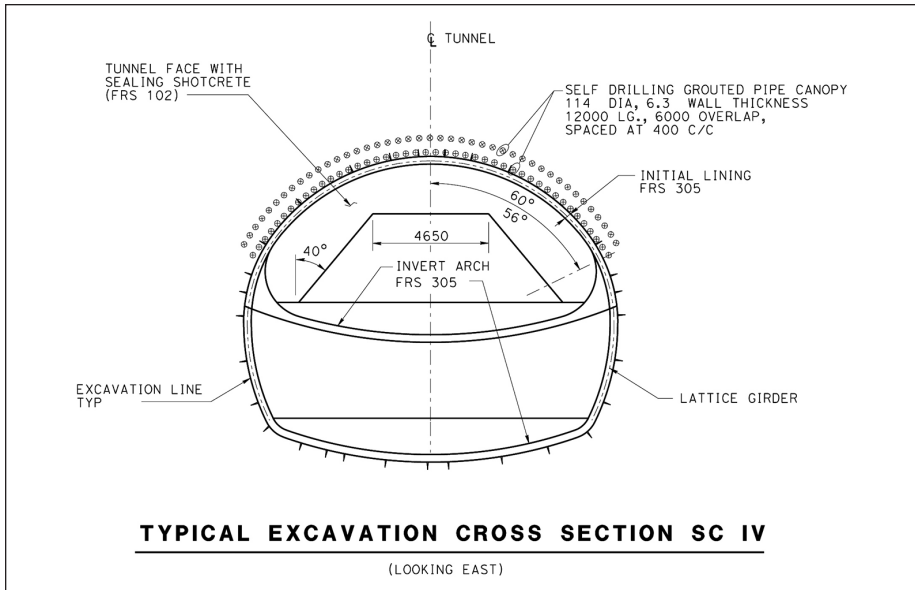


Figure 3. Example of support category requirements, typical excavation cross section for Support Category IV

portal. The contractor elected to drive the top heading from both ends of the alignment concurrently to expedite the schedule. Benching followed completion of the full top heading. Approximately 80% of the mined tunnel alignment (790 m [2,590 ft]) was excavated from the east portal, and the remaining 200 m (660 ft) was excavated from the west portal.

The east portal heading was advanced by TSC using a Wirth T3.20 roadheader, a Putzmeister shotcrete robot, an RDH two boom drill jumbo, RDH haul trucks and an RDH scissor lift, and a Sandvik load haul dump. Fiber reinforced shotcrete was pumped into the tunnel from the east portal as much as 700 m (2,100 ft) and was retarded for a maximum period of five hours. Beyond 600 m the shotcrete was trucked into the tunnel using a permissible haul truck with attached concrete mixing drum. Required excavation and support sequencing of the tunnel cross section within each advance length was performed in halves starting with excavation on the left side. While continuing with the excavation of the right side, the contractor concurrently started support installation on the left. The cycle finished with support application on the right side. DIBIT laser scanning of the excavated surface was performed immediately after excavation and again after shotcrete application to verify installed thickness and sufficient clearance from the design excavation lines. Production shotcrete testing was performed on a daily basis at the beginning of the project to verify achievement of early strength development and ultimate strength. Testing frequency was relaxed as construction proceeded, because the testing results consistently met or exceeded the specified requirements.

The west portal heading was advanced by Foxfire Constructors (FFC), a subcontractor to TSC. FFC used a Caterpillar 330C excavator with Alpine roadheader attachment or heavy hydraulic hammer attachments for excavation. Fiber reinforced shotcrete was pumped into the tunnel from the west portal for the full length of this heading. Other means and methods were similar to the east heading.

Table 2. Ground behaviors*

Behavior	Description of failure modes and manifestations in an unsupported tunnel
Block failure	Block failure is the discontinuity-controlled, gravity-induced failure of rock blocks that manifests as falling and sliding of blocks.
Raveling	Raveling is the progressive, discontinuity-controlled failure of small rock blocks within the general rock mass at or near the excavation surface. Raveling is manifested as successive fallout of small rock blocks and can ultimately result in a significant overbreak.
Shallow shear failure	Shallow shear failure results from overstressing of the ground within 0.25D to 0.5D of the tunnel perimeter (D = tunnel diameter) and may be enhanced by the potential for discontinuity and gravity-controlled failure modes. Shallow shear failure is manifested by moderate inward movement of the tunnel perimeter, including invert heave, and possibly by movement of rock into the tunnel opening along discontinuities.
Deep shear failure	Deep-seated shear failure results from overstressing of the ground beyond 0.25D to 0.5D from the tunnel perimeter. Deep-seated shear failure manifests as large radial convergence of the tunnel perimeter, including invert heave.
Slaking/softening	Slaking is the deterioration and breakdown of intact rock upon exposure by excavation and manifests as slabbing of material from the crown and side-walls. The severity of this behavior is assessed on the basis of slake durability tests performed according to ASTM Test Method D4644. Softening, which is dependent on wetting and exposure by excavation, is the reduction of intact rock strength at the invert or elsewhere and manifests as the development of a muddy or unstable invert or sloughing along segments of the tunnel perimeter elsewhere.
Swelling	Swelling occurs because of absorption of water by clay minerals in rock upon excavation-induced unloading. Swelling manifests as movement of the ground into the tunnel opening or additional tunnel support loading.
Crown instability due to low cover	Excessive crown geological overbreak and chimney-type failure will occur because of lack of confinement under low-cover reaches at portals. It manifests as block fallout and raveling above the crown.

* Modified from Austrian Society for Geomechanics, 2004.

Breakthrough of the top heading occurred at the end of November 2011 from the east portal heading after tunneling from the west side was completed to the breakthrough location roughly two weeks earlier. The breakthrough location at the west side was supported with a face supporting core, face sealing shotcrete, and spiles. When the tunnel heading approached from the east side, the same support measures as on the west side were installed to stabilize the decreasing rock pillar between the two headings. Breakthrough was completed successfully without any unexpected ground behaviors.

TSC's bench excavation sequence consisted of a center cut excavation followed by excavation of remaining side berms and installation of the tunnel sidewall support. TSC elected to perform the center cut bench excavation working eastward from the breakthrough point for the majority of this reach. Foxfire excavated the full face of the bench from the west portal towards the breakthrough point. Invert excavation and support followed benching, where required. Bench and invert excavation were completed in September 2012.

Final lining construction used a 15 m (50 ft) long form that was advanced uphill from the west to the east from April to October 2012. Typically, it took 8 to 10 hours to move, set, and place the concrete and another 8–10 hours for the concrete to set sufficiently to allow form removal, resulting in 4 to 5 form advances per week over a 6-day workweek.

KEY LESSONS

Predicted Versus Observed Ground Behaviors and Support Requirements

Encountered ground conditions and behaviors were mapped by the contractor's as well as the engineer's geologists on a daily basis during all phases of construction for each face. Probe holes were instrumented using an automatic data logger that recorded feed pressure, torque and advance rate and this information was interpreted to predict the ground conditions ahead of the tunnel face. Convergence monitoring was carried out across the tunnel arch and bench walls at instrumentation stations spaced approximately 15 m (50 ft) that were typically monitored within 100 m (300 ft) of the tunnel heading. All of this information was reviewed by the contractor's and the engineer's tunneling experts for ground classification and support selection at daily meetings.

Results of mapping are summarized (Figure 2) in terms of geological unit limits, Geological Strength Index (GSI; Marinos et al., 2005), and intact rock strength. Figure 2 shows that encountered ground conditions along the alignment were generally consistent with the design prognosis, with the exception of two reaches of the tunnel totaling 87 m (286 ft), or 9% of the alignment. These two reaches of differing site conditions occurred within the Second Sandstone between TM 241 and 322 (79 m [260 ft]) and within the Claremont Chert and Shale between TM 386 and 394 (8 m [26 ft]). In the Second Sandstone, the rock structure of the Second Sandstone encountered in the tunnel between TM 241 and 322 was blocky to massive, in contrast to the predicted blocky structure, and the intact rock strength was approximately 25% higher on average than indicated from strength tests performed during the design stage. The sandstone dikes in the Claremont Chert and Shale encountered in the tunnel between TM 386 and 394 exhibited a blocky to massive structure, in contrast to the predicted very blocky rock structure in the best rock mass in this formation.

As summarized in Figure 2, the groundwater inflows measured during construction were within the values of flush flow and sustained flow baselined in the Geotechnical Baseline Report—7 L/sec (110 gpm) and, 6 L/sec (95 gpm), respectively—at all but one location in the Orinda Formation. In this location the measured flow exceeded the predicted flush flow by approximately 1 L/sec (16 gpm) for a few hours.

Limited block failure, raveling, and shallow shear failure behaviors were observed to manifest during excavation and before the application of sealing shotcrete at the face and perimeter of the advanced heading in Support Categories II and III. Block failures typically involved sliding or toppling of blocks associated with steep bedding planes inclined into or away from the face. Raveling occurred over parts of the face and perimeter at locations of relatively higher fracturing and/or loosening. Raveling progressed above the crown spiles, in some instances requiring backfill grouting and longer 6 m (20 ft) spiles. Shallow shear failures primarily manifested as tunnel convergence, described below, and also as failure and movement of weak fractured zones at the face and heading perimeter. Large movements associated with deep shear failure were not observed. Slaking and softening behaviors were evident in the face and heading perimeter, as well as the invert prior to application of sealing shotcrete and mud mat, respectively. This was particularly the case within the Orinda Formation. Limited convergence of about 5 mm (0.2 in.), measured at a few locations in the Orinda Formation well behind the heading reach undergoing stress/strain redistributions, indicates possible manifestation of swelling behavior. However, we note that these movements behind the heading may also be due to other causes, such as the build-up of local hydrostatic pressure. The potential for swelling behavior was confirmed by swell testing of rock samples from the invert and was addressed in the design of the final lining invert. Observed behavior regarding crown instability due to low cover is discussed later in this paper. All of the anticipated behaviors were encountered to varying degrees, and

were successfully addressed by the combination of excavation and support sequences specified in the contract.

Figure 2 shows that the measured tunnel convergences are generally consistent with values predicted during the design phase, as well as with back analyses made during construction based on encountered ground conditions. We note that there is variability of measured convergences around the tunnel perimeter at any location along the alignment, as well as in-between adjacent monitoring locations along the alignment, reflective of factors such as local variations in ground or groundwater conditions, differences in excavation and support installation details (such as overexcavation, shotcrete thickness, and cycle times), and the amount of time after excavation when the monitoring bolt was installed.

Figure 4 shows that the measured tunnel convergences are below the warning level except at a few locations, where they approached but never exceeded the warning level. Additional support measures were installed at some of these locations to address the higher deformations. Warning levels were defined by considering the anticipated movements, shotcrete lining capacity, and practical considerations such as measurement accuracy.

Figure 2 shows that the design prognosis for the four major support categories matches the installed support quantities along the majority of the alignment. The major difference between the design prognosis and installed support quantities is related primarily to the lesser quantity of SC III that was installed compared to the design prognosis (this difference is not associated with the differing site condition reaches discussed above). The reason for this difference is that, while ground conditions anticipated to require SC III based on GSI, UCS data, and ground cover (Figure 2) were encountered, SC III could nonetheless be avoided in these reaches because of the contractor's high strength fiber reinforced shotcrete. This was shown by back analysis results, which indicate that while an SC II shotcrete lining with contractually required minimum strengths would have been overstressed (Figure 4, left), normal forces and moments are within the allowable criteria using the much higher as-built shotcrete strength (Figure 4, right). The better than specified shotcrete strength allowed for support selection of a thinner shotcrete lining, while still maintaining the required lining performance. In addition to savings in material costs, savings were realized because there was less time required for shotcrete installation and there was a longer allowable advance length. Both effects produced higher production rates with SC II instead of SC III. The predicted total quantity of SC III was 257 m (843 ft), as compared to the installed quantity of 60 m (197 ft).

Figure 2 also shows a significant deviation of the installed quantity and locations of SC II subtypes SC IIA and SC IIB from the design prognosis. The difference between these SC II subtypes is the extent and pay mechanism for spiling. Spiling was an additional support measure in SC IIA, while SC IIB included systematic spiling (total of 54 spiles) over the entire arch. The design intent was that SC IIB would be utilized where spiling was necessary around the majority of the arch and that SC IIA would be utilized where spiling was required over a limited portion of the arch. The contractor's interpretation of the contract was to apply the pay item for additional spiles applicable to SC IIA unless the full number of 54 spiles, as prescribed for SC IIB, were required. Negotiations between the contractor and the engineer established a payment criterion that compensated the contractor for SC IIB when more than 37 spiles were required at a particular location, and compensated the contractor for SC IIA plus the unit price for the number of spiles when less than 37 spiles were required. This deviation from the design intent resulted in the following differences between the predicted support and as-installed support:

- The predicted total quantity of SC II was 412 m (1,351 ft), compared to the installed quantity of 568 m (1,863 ft).

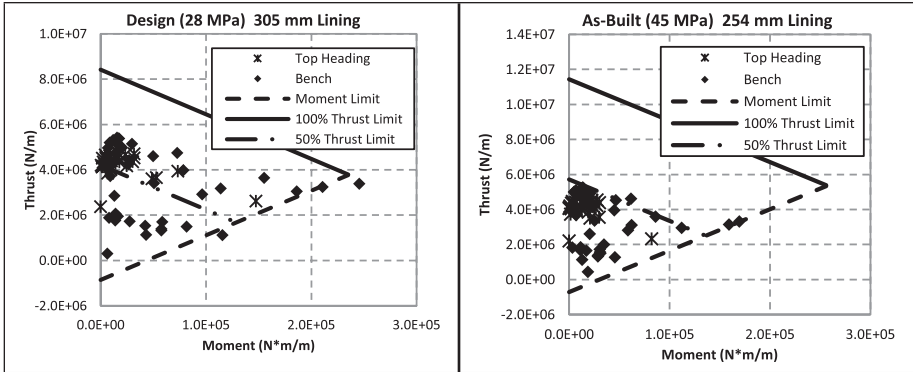


Figure 4. Shotcrete utilization at TM 690

- 35 m (115 ft) of SC IIA was predicted to be required, compared to the 380 m (1,246 ft) installed.
- 377 (1,236 ft) m of SC II B was predicted to be required, compared to the 188 m (616 ft) installed.

Contractual lessons related to the variation in quantities of SC IIA and SC IIB resulting from the application criteria for SC II are discussed further below.

Variations in Production Rates with Support Requirements

The majority of the tunnel (approximately 790 m [2,590 ft]) was excavated from the east portal, and the excavation of the top heading from the east portal took 15 months. The excavation of the west portal top heading was also completed within this timeframe. The length of the workweek varied between 5 and 6 days, with 20 to 24 working hours per day. The average production rate for the entire tunnel was 15.5 m (50.9 ft) per workweek, or 2.9 m (9.5 ft) per workday, based on the total length of tunnel and the time required for the east portal advance.

The best production of the top heading was achieved in Support Category IB with 11 advances of 1.8 m (5.9 ft) advance length, which is equivalent to 19.8 m (65.0 ft) per workweek. The highest production rates for other support categories (top heading) are:

- Support Category IA (in the Second Sandstone): 7 advances, with an increased advance length of 2.0 m (6.6 ft), or 14.0 m (45.9 ft) per workweek
- Support Category IIA: 14 advances, with a 1.4 m (4.6 ft) advance length, or 19.6 m (64.3 ft) per workweek
- Support Category IIB: 10 advances, with a 1.4 m advance length, or 14.0 m per workweek
- Support Category IIIA: 6 advances, with a 1.0 m (3.3 ft) advance length, or 6.0 m (19.7 ft) per workweek.

The weekly advance rates of the bench were subject to significant variation because of the different excavation sequences, the impact of footing excavations, longer haul routes, and impacts of off-cycle activities. Typical off-cycle activities were smoothing layer installation, excavation of the center cut ramp, and excavation and support of niches. Towards the end of the benching operation, the contractor focused on a concurrent bench/footing excavation, and weekly advance rates between 25 and 35 m (82 and 115 ft) were achieved. The peak weekly advance rate for the bench was approximately 45 m (148 ft). While typically the support and pre-support installation is

critical in the top heading, the loading and hauling of excavated material were the critical activities in the benching operations.

The production rates achieved by the contractor confirm that performance (i.e., increased production) is improved by repetition of a systematic sequence of activities and that changes in the support measures generally slows down production.

It is also interesting to note that the “best” support categories (applicable to the best ground) with the longest allowable advance lengths did not result in the highest production rates. The highest production rates were achieved where all key activities of the cycle tied into each other with minimal friction and waiting time so that these activities progressed in parallel as much as possible. Longer advance lengths, for example, may lead to lower production rates as support installation activities “idle” during completion of the mucking operation, which can be significant with a long advance length for a large span tunnel.

Contractual Considerations and Unit Price Structure for Support Selection

The detailed and prescriptive design of the excavation and support sequence was developed to minimize the number of support categories and pay items with the goal to simplify the construction operations and avoid an overly complex and cumbersome contractual payment process. Standard support categories on the Fourth Bore were measured and paid on a per meter basis, with the pay item covering all associated excavation and support requirements. This approach was judged to be more conducive to promoting competitive and responsive bids. By comparison, the application of NATM in Europe typically allows selection of excavation sequence and initial support elements in combinations appropriate to variations in encountered ground behaviors so as to achieve the most efficient tunnel support system possible. The European approach often results in a highly variable excavation and support process that requires different pay items for each support element such that they can be combined as needed. However, using this approach can result in significant variations between estimated quantities that have to be addressed in the contract.

The experience with construction of the Fourth Bore indicates that the simplified contract structure minimizes the potential for misinterpretation of the contract as related to a multitude of support variations. As experience with NATM grows in the United States, it will be possible to develop designs with more flexibility that will require more sophisticated contractual payment structures.

The payment approach for each support category was successful except in the case of the spiling that is part of SC II, as described above. Based on the divergence of the contractor’s interpretation from the design intent and the variability in the number of spiles required per advance, it may have been more advantageous to remove a prescriptive design for the spiling from standard support measures and pay for the spiles as additional support (including time-dependent costs such as impacts on advance rates).

The daily meetings between a small group of the contractor’s and the engineer’s tunneling experts provided an efficient forum for jointly evaluating technical data—monitoring data, geological and hydrological conditions, support performance, and the expected conditions ahead of the face. Based on this joint review the group decided on the support class and support measures required for the day’s advances. A key lesson was to keep the meetings focused on solving the technical issues and to leave contractual disputes for another forum.

Experience with Installation of 52 m Long Canopy Pipes

The design drawings showed nine fans of 12 m (40 ft) long, 114 mm (4.5 in.) diameter canopy pipes to provide presupport for the first 52 m (170 ft) length of tunnel from

the west portal, where the combination of low ground cover and disintegrated to very blocky disturbed seamy First Shale predisposed the tunnel to crown instability. The contractor proposed an alternative approach that entailed the use a single fan of 52 m (170 ft) long, 203 mm (8 in.) diameter canopy pipes installed using the Atlas Copco Symmetrix system and a Casagrande C8 drill, citing previous positive experience with this system. The canopy pipes were provided with one-way grout ports for grouting the pipe annulus.

The contractor's proposal was accepted after installation of two test pipes demonstrated that the maximum specified deviation of less than 396 mm (15 in.) along the entire pipe alignment and a maximum clear distance between any two adjacent pipes of less than 500 mm (20 in.) measured along the canopy arc could be achieved. The contractor agreed to install additional remedial pipes or spiling if the installed locations of the canopy pipes deviated from the design tolerances and cut out any pipes intruding into the tunnel excavation line.

The total of 51 canopy pipes including 8 remedial pipes were installed over a 3-month period, with construction difficulties resulting in significant delays especially during the installation of the initial pipes. Pipes that encroached into the initial lining were notched at the lattice girder locations or completely cut out, if the encroachment was more than 152 mm (6 in.) into the initial lining. At locations where pipes could not be installed for the full length or where deviated pipes created a gap between adjacent pipes of more than 500 mm (20 in.), remediation pipes were installed, if sufficient room for a remediation pipe installation was available. Where sufficient room was not available, local spiling was installed during the excavation and support of the section, if the encountered ground conditions dictated additional presupport measures. Once installed, the pipe canopy provided effective presupport for the ground.

Effects of Tunnel Construction Through Toe of a Weak Slope

The west portal (Portal No. 1) was cut into the side of a 43 m (140 ft) high slope with an average inclination of 33 degrees and consisted of colluvium underlain by the First Shale Formation. The First Shale is composed of a weak, black, silty shale that is highly fractured and crushed. The drained strength properties of the rock mass were determined using the Hoek-Brown procedure (Marinos et al., 2005), while the undrained rock mass strength was determined from laboratory testing of 60 to 145 mm (2.4–5.7 in.) diameter core samples. The rock mass properties are summarized below:

- Geological Strength Index (GSI): 26
- Unconfined compressive strength of intact rock (UCS): 5.3 MPa (800 psi)
- Hoek-Brown constant m_i : 7
- Mohr Coulomb friction angle: 30 degrees
- Cohesion: 65 kPa (1,350 psf).
- Undrained shear strength: S_u (kPa) = $7.35(z) + 38.5$ (kPa) or S_u (psf) = $153(z) + 804$ (psf)

A colluvium deposit, ranging in thickness from 0 to 10 m (0–32.8 ft), overlies the First Shale on the slope above the tunnel alignment. The colluvium deposit consists of stiff to very stiff clayey sand with gravel-sized rock fragments, sandstone blocks up to 0.3 m (1 ft) in dimension. No evidence of slip surfaces that would indicate past mass movement were identified during the investigation program that included eight borings into this deposit. Mohr Coulomb properties of the colluvium are a friction angle of 34 degrees and a cohesion of 10 kPa (200 psf). The groundwater table in the slope followed the slope contours at a depth of about 7 to 9 m (23–30 ft) below the ground surface.

Landslides frequently occurred on the slopes above the west portal cuts in the First Shale during construction of the First and Second Bores, at times measuring several thousand cubic yards. No slope instability occurred during construction of the Third Bore; however, the 6 m (20 ft) deep excavations adjacent to the slope to the north of the tunnel utilized soldier beam support.

The design of the portal structures (not discussed in this paper) and tunnel support system through the toe of the First Shale slope were developed to prevent any destabilization of the steep and potentially unstable slope. Measures employed included the use of pipe canopy presupport as discussed above; closing of the tunnel ring within 6 m (20 ft) and 10 m (33 ft) of the top heading and bench cut faces, respectively; limiting advance lengths to 1 m (3.3 ft) and 2 m (6.5 ft) for the top heading and bench, respectively; and the implementation of face support measures comprising a buttress as well as 102 mm (4 in.) of sealing shotcrete.

Stability of the slope was monitored during tunnel excavation by measurement of ground surface settlement and inclinometer readings. The tunnel section was also monitored for convergence, and the shotcrete lining was inspected daily to identify cracking. Monitoring during construction showed maximum ground surface settlements were about 35 mm (1.4 in.), and as shown in Figure 2, measured tunnel convergence was less than 25 mm (1 in.).

CONCLUSIONS

Encountered ground conditions and behaviors were generally consistent with design prognosis except for two locations, which amounted to 9% of the full tunnel length. Major support categories installed were also generally consistent with the design prognosis, with the exception of the less than anticipated use of Support Category III, which can be attributed to the installation of shotcrete that achieved almost double the specified strength. A significant deviation from bid quantities between subtypes of Support Category II (SC IIA versus IIB) occurred because of inclusion of spiling in the lump sum pricing structure used for Support Category IIB, and the contractor's interpretation of the conditions where each support subtype should be used. This variation from the bid quantity for the overall support category could have been avoided by paying for the spiles on a unit price basis.

One of the challenges on this project was construction of this 15 m (50 ft) span tunnel through the toe of a slope consisting of weak and broken shale. The use of ring closures, canopy pipes, and small advance lengths allowed for successful advance of the tunnel through this weak slope without adverse impacts to the slope. Canopy pipes at the west portal were installed in a single 52 m (170 ft) length per the contractor's preference, instead of nine fans of 12 m (40 ft) long pipes. Several difficulties were encountered during installation of the 52 m long canopy pipes, requiring removal of some pipes and installation of additional pipes. However, the system, once installed, provided the required presupport to the ground and allowed an uninterrupted excavation and support cycle through this difficult reach of tunnel.

Construction of the Fourth Bore demonstrated that production is enhanced by the consistent repetition of the same or similar excavation and support cycles. In general, too many changes between excavation and support activities and the implementation of off-cycle activities interrupts the overall cycle and results in negative impacts on production. Typical weekly advance rates for the Caldecott Fourth Bore were between 15 and 20 m (49 and 66 ft) for the top heading and 25 and 35 m (82 and 115 ft) for the bench.

The as-designed tunnel excavation and support sequence effectively controlled adverse ground behaviors that had been encountered during construction of the previous Caldecott Bores. Drainage ahead of the face, face dowels, a face supporting

buttress, spiling, control of advance lengths, and sealing shotcrete were all critical to control block failure, raveling, and shallow shear behaviors at the face in various instances. Similarly, installation of a high-performance shotcrete lining and fully cement-grouted dowels immediately behind the face was effective in addressing anticipated behaviors behind the face in the fractured and weak ground conditions along the alignment. Daily meetings between the contractor and the engineer were key to application of the appropriate combination of excavation and support measures, which enabled advance of the heading in the most efficient manner possible and completion of tunnel excavation and support on schedule.

ACKNOWLEDGMENTS

The authors would like to acknowledge Peter Kottke, Rene Starck, and Roozbeh Geraili Mikola of Jacobs Associates for their work on numerical analysis and interpretation of data. The contents of this paper were reviewed by the State of California's Business, Transportation and Housing Agency; the California Department of Transportation; and the Contra Costa Transportation Authority. The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein, and do not necessarily reflect the official views or policies of the State of California or the Contra Costa Transportation Authority. This paper does not constitute a standard, specification, or regulation.

REFERENCES

- ASTM D4644. Standard Test Method for Slake Durability of Shales and Similar Weak Rocks. ASTM International.
- Austrian Society for Geomechanics. 2004. Guideline for the geomechanical design of underground structures with conventional excavation. Draft English translation.
- Marinos, V., Marinos, P., and E. Hoek. 2005. The geological strength index: applications and limitations. *Bulletin of Engineering Geology and the Environment*, 64, 55–65.
- Page, B.M. 1950. Geology of the Broadway Tunnel, Berkeley Hills, California, *Economic Geology*, 45(2).
- Thapa, B.B., McRae, M.T., and Van Greunen, J. 2007. Preliminary design of the Caldecott Fourth Bore. In *2007 RETC Proc., Toronto, Ontario, Canada, June 10–13*. Littleton, CO: SME.
- Thapa, B.B., Van Greunen, J., Sun, Y., McRae, M.T., and Law, H. 2008a. Design analyses for a large-span tunnel in weak rock subject to strong seismic shaking. In *2008 NAT Proc., San Francisco, CA, June 8–11*. Littleton, CO: SME.
- Thapa, B.B., Marcher, T., McRae, M.T., Sander, H.J. 2008b. Design of NATM initial support on the Caldecott Fourth Bore. In *Proceedings of the American Rock Mechanics Symposium, San Francisco, 2008*.
- Thapa, B.B., Marcher, T., McRae, M.T., John, M., Skovajsova, Z., and Momenzadeh, M. 2009. NATM strategies in the U.S.—Lessons learned from the initial support design for the Caldecott 4th Bore. In *2007 RETC Proc., Toronto, Ontario, Canada, June 10–13*. Littleton, CO: SME.
- U.S. Department of Transportation Federal Highway Administration (FHWA). March 2009. *Technical Manual for Design and Construction of Road Tunnels—Civil Elements*. NHI Course 130090. Publication No. FHWA-NHI-09-010.