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Tunnel Design and Construction in a Franciscan Melange

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ABSTRACT: This paper discusses the design and ongoing construction of an inclined inlet pipeline, a drop shaft, and a 620 meter outlet tunnel in ground comprised of a variety of rock blocks and crushed matrix of a Franciscan melange. The project, situated in the right abutment of the Lenihan Dam at Lexington Reservoir near Los Gatos, California and within 3 km of the San Andreas Fault, will provide a new water outlet pipeline for the reservoir. The tunnel, designed to provide long-term maintenance access, has a straight-legged horseshoe shape with an excavated width and height of 4.3 m and 3.7 m, respectively, and is being excavated by conventional roadheader and drill-and-blast methods. A detailed description of the geotechnical investigation, interpretation of geologic data, characterization of the Franciscan, and development of a design methodology for the initial tunnel support is given. A significant challenge during the design was to develop an initial ground support system that could be quickly adapted during construction to the chaotic nature of the Franciscan, which can exhibit a full spectrum of rock behavior – from stable to squeezing - within a few excavation rounds. To accomplish this, the design includes a robust but flexible initial support system that has the capability of being modified as excavation proceeds on the basis of geologic observation, pre-excavation probing, and convergence monitoring. The tunnel final lining incorporates elements of the initial support and is designed for long-term squeezing and a groundwater head up to 60 meters.

1. PROJECT DESCRIPTION AND BACKGROUND

From the late 1980s to the early 1990s, the 1.2 m (48-inch) steel lined outlet pipe beneath the earth-fill Lenihan Dam was repeatedly damaged by buckling. The 1989 Loma Prieta earthquake and periodically high discharge velocities have been cited as possible contributing factors [1]. Although repairs were made, the Division of Safety of Dams (DSOD) has since limited discharges to 2.0 cubic meters per second (70 cfs), a significant reduction from the historic allowable peak discharge of up to 11.6 cubic meters per second (410 cfs) and a constraint on the Santa Clara Valley Water District's use of the reservoir for downstream groundwater recharge. Moreover, this operating restriction put the current outlet in nonconformance with a primary DSOD emergency operating criterion stipulating that the outlet must be able to lower the reservoir's hydraulic head by 10 percent within seven days.

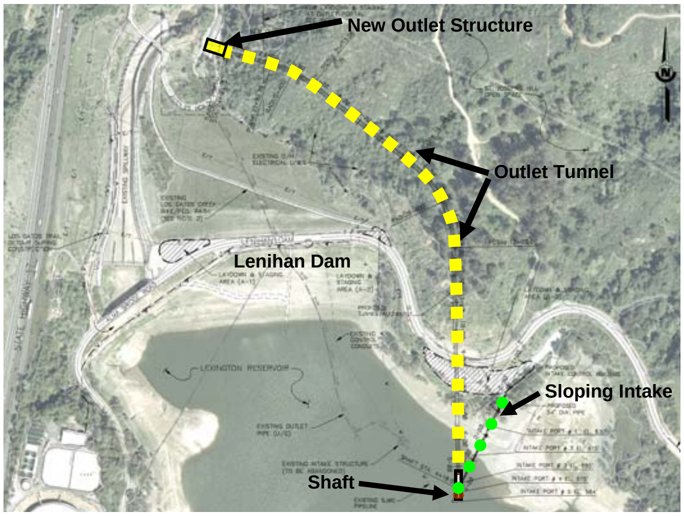


Figure 1. Site Plan (JA 2007)

The selected remedy has been to construct a new outlet works system with the outlet pipeline to be installed in an outlet tunnel through the right abutment of the dam. This configuration will satisfy the DSOD criteria and improve the Santa Clara Valley Water District's reservoir outlet controls in addition to offering a future option to convey raw water to a downstream pipeline.

The proposed outlet works are located at the north end of the Lexington Reservoir about 2.4 km south of the City of Los Gatos. As shown on the site plan in Figure 1, the facilities at the upstream end consist of an intake control building about 215 m east of the crest of Lenihan Dam and a 135-meter-long, 1.3-meter-diameter (54-inch-diameter) inclined pipeline with intake valves. The underground structures include a 12.2 meter deep inlet shaft at the base of the inclined intake and a 620 meter long outlet tunnel. An outlet structure with energy dissipation chambers will be constructed at the downstream tunnel portal. Figure 2 is a profile of the outlet works.

The outlet tunnel will have a straight-legged horseshoe shape (see Figure 3, below) and a final lining of cast-in-place concrete. The 1.3-meter outlet pipe will be installed on saddle supports, offset to one side of the tunnel to allow access for regular inspections and maintenance. The concrete tunnel lining is required to be sufficiently watertight to maintain dry conditions during personnel access and for long-term protection of the pipeline and equipment.

2. GEOLOGY

2.1. Regional Setting

The Lenihan Dam is located in the Santa Cruz Mountains within the California Coast Ranges geomorphic province. The northwest-trending San Andreas fault bisects the Santa Cruz Mountains into southern and northern components and is located approximately 3 km southwest of the project site.

Northeast of the San Andreas fault, and underlying the project area, basement rocks of the Coast Ranges consist of the Franciscan Complex (the Franciscan) which includes heterogeneous melange bodies of marine sedimentary and volcanic rocks of Jurassic to Cretaceous age [2]. Franciscan lithologies consist primarily of greywacke sandstone and interbedded shale, with lesser amounts of limestone, radiolarian chert, pillow basalt, meta-basalt or greenstone, and associated tabular intrusive bodies of serpentinite [2]. Franciscan melange rocks are typically complexly deformed and intermixed due to faulting, fracturing, shearing, and crushing associated with ancient tectonic processes.

2.2. Project Geology

In the project area the Franciscan melange consists of blocks, mainly of greywacke and serpentinite with some greenstone, embedded in matrices of crushed and sheared shale and serpentinite that is frequently clayey. The geology is generally chaotic. However, as was shown by the geologic mapping and current tunneling, there are grossly mappable units. For example, the tunnel has encountered a zone of predominately serpentinite melange (blocks of serpentinite in a matrix of serpentinite) that transitions to a zone of predominately greywacke melange (blocks of greywacke in a matrix of shale).

The bedding, foliation and structures observed in blocks of sandstone and shale in the Melange are not generally representative of the overall structure of the melange rock mass because the individual rock blocks and the surrounding matrix have been subject to random and chaotic movements during the evolution of the Franciscan. However, some weak foliation trends in the matrix – for example, foliation strikes and dips measured within the sheared matrix east of the Lenihan Dam – generally indicate consistent northwest to northeast

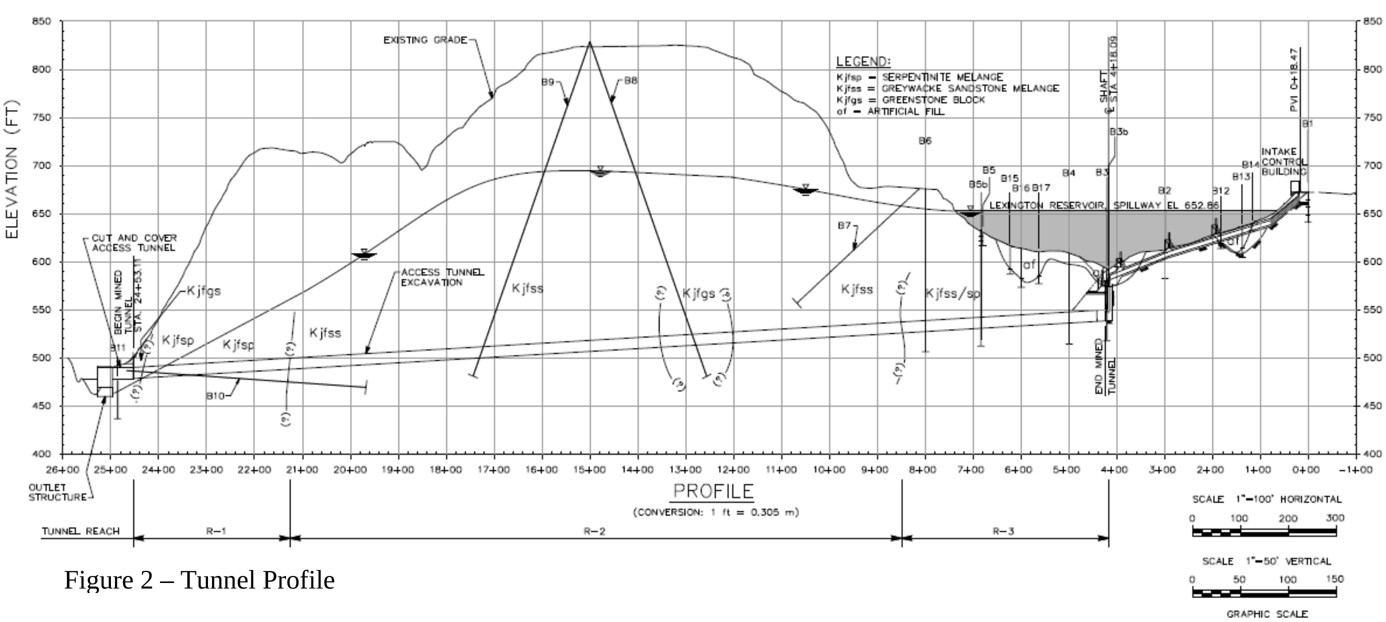


Figure 2 – Tunnel Profile

strikes and undulating dips that vary between 15 to 80 degrees to the east [2].

Blocks and also the rock matrix are very closely fractured, and joints/fractures are generally randomly oriented, although systematic joints do occur locally. On a larger scale, numerous faults and shears occur within the melange, and some prominent mapped faults, which strike approximately northwest and have moderate to steep dips to the northeast, form boundaries between blocks and matrix. Overall, the matrices often exhibit foliations that appear to wrap around the melange blocks with sheared contacts.

Faults and foliation are typical of the melange and most, if not all, are likely associated with tectonic regimes active during early Tertiary Franciscan Complex emplacement. [2]

3. GEOTECHNICAL INVESTIGATION

3.1. Field Investigation

The field geologic and geotechnical investigation was performed in two stages. The initial phase included an on-land investigation, with drilling of 7 borings for the outlet works structure, tunnel alignment, and intake control building site. Two of these borings were angle holes greater than 122 meters long and another was a 152 meter long horizontal hole from the tunnel portal. The second phase included an over-water investigation which involved drilling 11 borings that reached as far as 37 meters below the reservoir bottom. This phase explored the upstream portion of the tunnel and the sloping intake pipeline alignment. Field geologic mapping, borehole sampling and testing, and laboratory testing were carried out. The field mapping was of particular importance in acquiring an understanding of the geometry of the melange blocks and matrix. For example, outcrops demonstrated that blocks encountered in the tunnel could be far larger than the tunnel face and result in hard-rock tunneling conditions.

Borehole logs and observation of the core aided in establishing a basis for the proportions of the various rock types and also in determining the applicability of rock mass classification techniques for ground support selection. However, interpretations for potential tunneling behavior were often difficult because the core, whether of a block or a matrix sample, was typically highly fractured and tended to break apart in the core box. In general, this can lead to overly conservative predictions of ground behavior unless the influence of in-situ confining stress can be accounted for.

Groundwater conditions were of concern given the proximity to the reservoir and the fractured nature of the melange. In-situ tests included packer tests and piezometer installations. Although packers were often difficult to seat and seal in the melange materials, permeability data - as well as information on fluid and core loss - aided in assessing the hydrogeologic regime.

3.2. Laboratory Testing

Laboratory tests were performed on selected soil and rock core samples from borings to evaluate their physical characteristics, engineering properties, and chemical characteristics. The testing program included: moisture content, Atterberg limits (liquid and plastic limits), and consolidated undrained triaxial tests with pore pressure measurements for soils. The test program for rocks included: unconfined compressive strength, slake durability, and Cerchar Abrasivity.

The Franciscan serpentinite rocks also required certain tests for environmental considerations, notably for chromium and nickel metal content to satisfy U.S. Environmental Protection Agency regulations, and asbestos content to satisfy the California Air Resources Board requirements.

Table 1: Ground Conditions and Rock Loads

Ground Condition	Definition	Terzaghi Rock Load Classification[1]	
		Vertical Rock Load[2]	Side Pressure
Blocky and Seamy	Rock mass mainly consisting of imperfectly interlocked, angular to subangular blocks or fragments of rock. Rock mass contains occasional seams of silty, sandy clay and clayey sand. Individual blocks or fragments of rock are chemically intact or nearly intact are typically smaller than one foot. Where individual blocks are larger than one foot, the ground is termed moderately blocky and seamy.	(0.35 to 1.1)C	Little or no side pressure
Crushed	Rock mass consisting of poorly interlocked, heavily broken rock fragments that are chemically intact or nearly intact and gravel sized or smaller. Rock mass is frequently interspersed with zones and seams of silty, sandy clay and clayey sand.	1.1 C	Considerable side pressure
Squeezing	Soil-like ground that is predominantly fine-grained, consisting mainly of clayey, silty, and sandy gouge. Non-blocky due to soil-like consistency and closely-spaced shears and foliation. Contains gravel-sized or smaller rock fragments.	(1.1 to 2.1)C	Heavy side pressure

Notes: 1 From Terzaghi (Proctor and White, Table 3, 1968 [5])
2 C=B+H, where B=tunnel width, and H=tunnel height. Formulas are valid for any consistent set of units. Multiply by unit weight to get load per unit area (pressure).

4. GROUND CONDITIONS FOR TUNNELING

4.1. Ground Classification

In tunnel design, one method used to evaluate rock mass characteristics and select support requirements is to use data from core logs and outcrops and apply empirical methods, notably Bieniawski's Rock Mass Rating (RMR) system [3] and Barton's Tunneling Quality Index (Q) [4]. However, RMR and Q both use the Rock Quality Designation (RQD) [5] as a primary index in addition to other ratings based on joint characteristics and intact strength. Because discontinuities are frequent and ubiquitous in the melange, RQD is typically zero and therefore does not allow sufficient differentiation of the rock mass properties. However, RQD did allow estimates to be made of the percentage of ground that could be classified by the Terzaghi classification, for example: "very blocky and seamy", "crushed", and "squeezing" ground. (Table 1 provides a modified version of the Terzaghi classification in Proctor and White [6]) Further definition was made on the basis of predominate grain size - that is, whether gravel, sand, or clay.

A summary of the Terzaghi classification categories of the materials encountered in the borings by rock type is presented in Table 2. Although the distribution of melange materials is chaotic, it has been assumed that the amounts of materials (blocks and matrices of the various rock types) encountered during construction will be approximately proportional to the percentage of materials logged from the boreholes.

In order to determine the distribution of the melange materials and classify the rock types, it was necessary to distinguish the blocks from the matrix within the rock core. For this project the smallest significant block size was taken as 15 cm (6 in) in diameter or about 5 percent of the excavated tunnel diameter, based on the approach used by Medley [7]. A block was defined as a rock mass that is significantly stronger than the material surrounding it. The distinction between block and matrix was not always apparent, however, because differences in rock strength and condition were often gradational rather than abrupt. For example, a strong and intact zone of serpentinite may grade into a strong but brecciated zone of serpentinite, which may further grade into a highly sheared and clayey zone, with no clear distinction between the zones. Or a shale/argillite may progress from relatively strong and intact to weak and friable over the course of a meter within the core.

As stated above, the distinction between block and matrix was based on relative strength and block size, and not necessarily on lithology. The matrix encountered on this project typically consisted of sheared and altered black shale, but was also composed of sheared serpentinite, shale/argillite, and fragments of greywacke. The matrix identified in Table 2 is the typical black shale matrix only.

Based on the assessment of the ground conditions, the tunnel was anticipated to encounter zones comprised primarily of friable to very strong rock (blocks), zones comprised primarily of crushed rock and soil (matrix), and mixed face conditions where various properties of blocks and matrix are encountered in any given location. The unconfined compressive strength of the intact rock can range up to 240 MPa (35,000 psi) for greywacke and 275 MPa (40,000 psi) for greenstone although strengths of the fractured rock mass are generally much lower. Block shapes range from generally ellipsoidal to irregular and appear to follow a 2-to-1 ratio of major to minor axes. Sizes range from sand, gravel and cobbles up to boulders and large blocks hundreds of feet across.

The irregularity and wide variation in block size has resulted in a material ranging from dense and hard soil, to rock that is friable to very strong. The main consequence of this geologic mixture, especially in regard to the prediction of ground conditions for tunneling, is that the melange typically lacks spatial continuity and therefore exhibits frequent and abrupt variations in geomechanical characteristics, which makes choosing an effective yet economical excavation approach challenging.

Table 2 – Ground Condition Percentages for Melange Rock Types

Rock Type	Squeezing (%)	Crushed (%)	Blocky and Seamy (%)	Moderately Jointed (%)	Total (%)
Matrix (shale)	18	5	--	--	23
Greywacke	--	8	24		32
Greenstone	--	1	5	3	9
Serpentinite	1	16	4	--	21
Shale/Argillite	3	8	3	--	14
Chert	--	1		--	1
Total	22	39	36	3	100

The following factors summarize the tunneling issues in the melange:

- Weak and friable rock units that will have low stand up time below the groundwater level.
- Squeezing ground conditions that result in tunnel sidewall and invert movement and that may occur immediately or weeks following excavation.
- Blocks of strong, hard, highly abrasive rock with moderately spaced joints.
- Mixed ground conditions consisting of boulders of strong, hard rock embedded in a weak, clayey and gravely matrix.

4.2 Tunnel Reaches

In order to describe the ground conditions anticipated during construction, the Access Tunnel has been divided into three reaches, numbered from the Portal to the Shaft, in an upstream direction and in the direction of the mining operations. See Figure 2 for the extents of the reaches:

- Reach R-1 is predominately composed of serpentinite melange, extends 100 meters from the mined portal face to a transitional contact with the greywacke sandstone of Reach 2. The cover over the tunnel invert in this reach ranges from 6 to 70 meters.
- Reach R-2 is predominately greywacke sandstone melange and extends 386 meters from the contact with the serpentinite melange of Reach R-1. The cover over the tunnel invert in this reach ranges from 47 to 98 meters.
- Reach R-3 consists predominately of serpentinite and sandstone melange and extends 132 meters from the transitional contact with the sandstone melange of Reach R-2 to the base of the shaft. The cover over the tunnel invert in this reach ranges from 17 to 47 meters.

Reach boundaries are not precise contacts but transitions between areas where certain melange lithologies are expected to be encountered. As mentioned above, each reach is defined by a predominant rock type; for example, sandstone blocks and shale matrix, or serpentinite blocks and shale or serpentinite matrix. The serpentinite melange of Reach 1 included approximately 90 meters of serpentinite blocks in serpentinite matrix.

Descriptions of the anticipated ground conditions for the three reaches are summarized below.

5. TUNNEL EXCAVATION METHODS

Previous methods used in tunneling in melange, (e.g., the Claremont Bypass tunnel [8], the BART Berkeley Hills tunnels [9], and the Pacheco tunnel) include roadheader and/or drill-and-blast techniques. TBMs have also been used locally for excavation in the melange (Richmond Transport tunnel); however, the small radii of the planned horizontal curves and the inefficiency of using a TBM for a relatively short tunnel preclude the use of a TBM for this project. The preferred excavation method for the Lenihan Tunnel is the use of a roadheader, alternating with drill-and–blast to deal with resistant blocks, either in mixed face/ground conditions or in full-face tunneling.

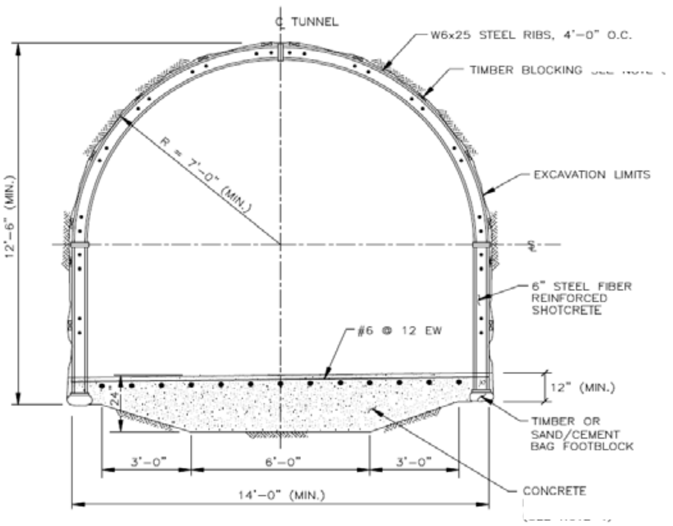
6. TUNNEL INITIAL SUPPORT

6.1. Ground and Support Categories

The melange materials were classified on the basis of the Terzaghi ground conditions as shown in Table 1. Three ground types were developed: Type 1, Type 2, and Type 3, roughly corresponding to blocky-and-seamy, crushed, and squeezing ground, respectively. However, the ground types typically overlapped so that Type 1 ground, for example, might include blocky-and-seamy ground and a certain percentage of crushed ground. Factors such as overburden pressure, anticipated rock mass permeability, lithology, and average strength along the tunnel alignment affected the distribution of ground types. As an example, squeezing ground classified as Type 3 was anticipated only when the overburden pressures exceeded about 1.5 times the average strength of the rock mass (see Table 3 below).

Because the ground conditions at the Lenihan Tunnel were expected to vary widely, the initial support system was designed to be flexible and adaptable. This was achieved by developing a basic initial support design that could be augmented without making substantial changes to the geometrical cross section of the excavation or creating a need to remove or change any structural components if loads/deformations are found to be higher than anticipated at the beginning of excavation.

Ground support types correspond directly with the ground types, thus three types of initial rock support were designed for the variable rock conditions. Type 1 consists of blocked steel ribs spaced at 1.2 meter (4 ft) on-center, with up to 8 cm (3 in.) of steel fiber reinforced shotcrete applied as needed to stabilize the excavation surface and control localized instabilities. Type 2 support consists of steel ribs spaced at 1.2 m on-center with 15 cm (6 in.) of steel fiber reinforced shotcrete lagging. Type 3 support consists of steel ribs spaced at 1.2 m on-center, with 23 cm (9 in.) of steel fiber reinforced shotcrete lagging. Type 3 may be split-spaced with jump sets at 0.6 m (2 ft) if the ground loading is expected to be particularly heavy. Typical initial support configuration is shown in Figure 3. For Types 2 and 3 support configurations, the excavated opening was anticipated to have adequate stand-up time to allow for installation of initial support.



Conversion: 1 ft = 0.305m

Figure 3: Tunnel Cross Section and Initial Support

Both Type 2 and Type 3 supports require the use of a reinforced concrete invert slab to act as a strut to carry side loads and prevent heave of the invert. In order to maintain a consistent excavation cross section throughout the support types, a 30 cm slab thickened to 61 cm at the tunnel center line was used for all support types. Although the invert slab is thicker than is necessary for Type 1 and Type 2 initial support, the added thickness is not wasted. High hydrostatic groundwater loading on the final lining necessitates the use of a thick final concrete invert. The reinforced concrete invert used for initial support will be incorporated into the final lining to provide permanent ground support.

6.2. Empirical Analysis

The initial analysis for the horseshoe shaped steel rib supports utilized a conventional structural analysis of a circular beam loaded at specific, uniformly-spaced blocking points (Proctor and White [6]). This method uses empirical estimates of rock loads based on Terzaghi's rock mass classifications (Refer to Table 1).

Each rock condition and associated rock load was analyzed to define the required size and weight of the steel beam to be used for the steel sets. It was determined that W6x25, Grade 50 steel sets spaced at 0.6 to 1.2 m centers would provide adequate support for the range of ground conditions.

6.3. Numerical Analysis

To verify the initial support selection developed using the empirical design methods, numerical analyses were carried out for all three support configurations. The analyses were performed to determine the static tunnel loads using the finite-difference program FLAC (Version 5, Itasca, 2005) and were focused on evaluating ground behavior, rock loads, and the associated axial forces and bending moments induced in the initial support.

In the analysis, behavior of the outlet tunnel and its initial support under static loading were evaluated using the ground-structure interaction approach. The analysis assumed that the tunnel would be drained during the construction period, with a zero pore pressure on the tunnel boundary.

Laboratory data and the methodology proposed by Hoek [10] were used to develop the geomechanical properties for the rock mass used in the numerical analyses. Table 3 presents the properties used in the analyses.

The tunnel section analyzed corresponds to the maximum overburden depth of about 96 m. The regional groundwater level was assumed to be 40 m below the ground surface. The horizontal-to-vertical stress ratio (K_o) was assumed to vary between 0.5 and 1.

Table 3. Mechanical Properties of Rock Masses

Material Constant	Type 1 Support (Blocky and Seamy)	Type 2 Support (Crushed)	Type 3 Support (Squeezing)
Wet Unit Weight g/cm3 (PCF)	2.56 (160)	2.56 (160)	2.56 (160)
Elastic Modulus MPa (KSI)	3100 (450)	1620 (235)	275 (40)
Poisson's Ratio	0.3	0.3	0.3
C' MPa (psf)	0.41 (8600)	0.11 (2400)	0.077 (1600)
Φ' (degrees)	48	23	26.5
Max. Depth m (ft)	96 (315)	96 (315)	96 (315)
Max. Static Head m (ft)	56 (185)	56 (185)	56 (185)

Ground reaction curves for the three support types / ground conditions are illustrated in Figure 4. These curves show the tunnel radial displacements as a function of the ratio of support pressure to in-situ stress, and were used to assess the ground behavior during and following excavation. The results indicate that the Type 1 ground conditions are expected to stabilize without support. However, the FLAC analysis did not account for loosening blocks, and therefore some support is required. The analyses show that Ground Type 2 (corresponding to Type 2 support) will experience large displacements, approaching 13 cm (5 inches) without support, and that Type 3 Ground is not stable without support.

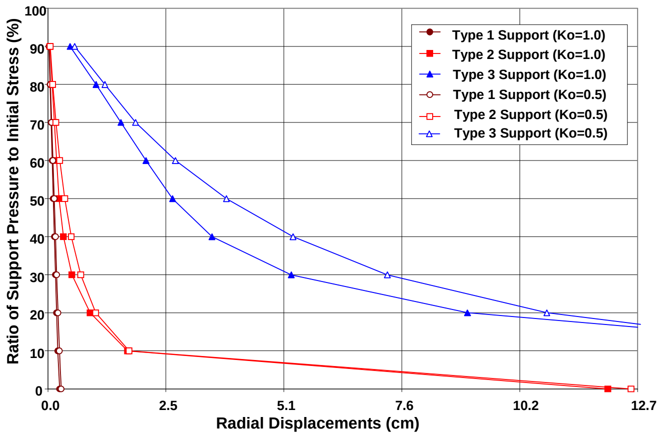
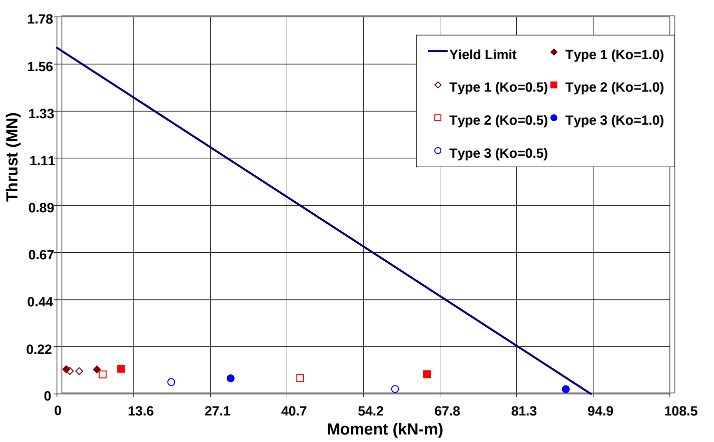


Figure 4: Ground Reaction Curves at Tunnel Crown

Ground relaxation ahead of the tunnel face was assumed to be about 80% and 70% for the ground conditions associated with Type 2 and 3 supports, respectively. The rock loads back-calculated from the thrusts and moments predicted by the numerical model (See Figure 5) are less than 12, 19, and 29 m of ground for ground Types 1, 2 and 3, respectively.



Note: Points represent thrust-moment loads that are normalized to a single steel set and also reflect the effect of thrust capacity of shotcrete lagging. The yield envelope represents only the thrust-moment capacity of a single steel set.

Figure 5: Interaction Diagram for W6x25 Steel Sets

7. GROUNDWATER CONTROL

In the melange, groundwater flow is expected to occur within zones of rock containing discrete fractures, generally in fresh rock. In other zones, typically where rock is moderately to severely weathered, the measured hydraulic conductivity is one or two orders of magnitude lower, and therefore, the maximum water inflows are anticipated to be confined to discrete and limited zones of rock.

Based on estimates of the effective hydraulic conductivities, potential groundwater inflows were determined by the methods of Heuer [11] for the three tunnel reaches. Instantaneous heading inflows are groundwater inflows that occur at the tunnel face and at the sidewalls of the tunnel within 30 meters of the face. Maximum inflows are the peak groundwater inflows that will occur in a reach as the tunnel is excavated. For Reach 1, groundwater levels are expected to range from about 1.5 to 21 meters above the tunnel invert. Packer tests within this rock mass measured in-situ hydraulic conductivities ranging from about 3 x 10^{-6} to 3 x 10^{-5} cm/sec. Instantaneous heading inflows in this reach are not expected to exceed 3 liters per second (lps), and maximum inflows for the entire reach will not exceed 1.3 lps.

Groundwater levels for Reach 2 are expected to be higher - about 21 to 58 meters above the tunnel invert. However, packer tests within this rock mass measured similar in-situ hydraulic conductivities ranging from about 1 x 10^{-6} to 3 x 10^{-5} cm/sec. Instantaneous heading inflows in this reach are not expected to exceed 4.4 lps, and maximum inflows for the entire reach would be less than 1.9 lps.

In Reach 3, groundwater levels are expected to range from about 35 to 41 meters above the tunnel invert, coincident with a full reservoir level at elevation 199 meters. Because packer tests within this rock mass measured in-situ hydraulic conductivities ranging from about 3 x 10^{-6} to 1 x 10^{-2} cm/sec, estimated instantaneous heading inflows are significantly higher - up to 9.5 lps - and maximum inflows for the entire reach could reach 32 lps.

The tunnel excavation operations include continuous probing to detect water bearing features ahead of the face. Two probe holes are drilled from the face, typically 15 m (50 ft) in length with a minimum 6 m (20 ft) overlap from the previous probe. When ground water inflows from either probe exceed prescribed limits, the probe holes are grouted and additional holes are drilled to determine if the water bearing feature has been successfully grouted off. This procedure is continued until inflows are at acceptable levels.

8. CONSTRUCTION TO DATE

The tunnel excavation at the time of this writing (April 2008) has advanced approximately 348 meters. To date, more Type 1 support has been installed than anticipated, and the ground has been sufficiently stable to frequently allow the contractor to excavate and install several steel ribs before shotcreting. Even when crushed or brecciated ground is encountered the material is typically very dense, and subsequent loosening and raveling are often minor. A notable exception is where locations of seeps correspond with locations of crushed ground or slickensided clay filled joints. In crushed zones the ground mass may slowly ravel, while locations with clay filled slickensided joints yield localized block failures in the presence of small amounts of water. These problems have been infrequent, however, and have been controlled by timber forepoling and steel rebar spiling. In limited areas, where overbreak due to raveling has occurred, timber cribbing has been used.

Deformation of the tunnel side walls has been monitored at convergence monitoring points installed in the tunnel. See Figure 6 for a typical configuration.

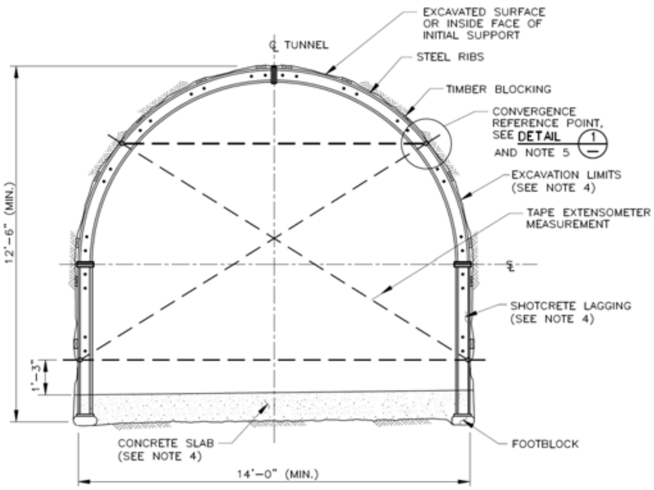


Figure 6. Typical Convergence Measurement Station

Monitoring points are typically installed on the shotcrete between the ribs within about 1 to 2 meters from the excavated face, and initial readings are taken within approximately 3 to 5 meters from the excavated face. This is typically 10 to 16 hours after the excavated face has progressed past the monitoring location. Convergence measurements typically stabilize after 2 to 3 weeks at 10 mm to 20 mm. The measured convergence thus far has not exceeded 35 mm in any of the support types.

The excavation has primarily been advanced using an Alpine (AM 50) roadheader. This relatively small roadheader fits well within the excavated dimensions of the tunnel and has been able to excavate a majority of the stronger blocks. The greywacke sandstone blocks are often high in strength, but can be excavated without too much trouble because they are often highly fractured and tend to be plucked by the cutting teeth. Large blocks of serpentinite and greenstone have been more problematic for the roadheader due to wider spacing of discontinuities, and have been blasted using controlled drill-and-blast techniques. Blasting has been required for individual blocks a meter in diameter in the face, to full face blasting through blocks as large as 37 meters in diameter.

The total sustained groundwater inflow over the 348 meters excavated to date has been less than about 0.5 lps, even under the anticipated maximum head of 56 m. The maximum inflow from any discrete feature thus far has not been sufficient to produce a steady stream of water, with one exception. The excavated rock mass is typically dry to damp, leading to often very dusty conditions during roadheader excavation.

Approximately 5.6 lps was measured from the primary grout holes when the tunnel heading was approximately 300 m from the portal. At that location the tunnel face was in a zone of shale matrix that was highly weathered to very stiff clay, and the water bearing feature was encountered during probing in harder rock just beyond this zone.

Type 3 ground support has been required infrequently. To date, Type 3 support has been installed in a potentially squeezing zone - an 8.5-meter-long section of the above-mentioned shale matrix zone which was under high ground cover. At another location, rock movement pushed a steel rib about 5-7 cm from the sidewall, indicating squeezing conditions.

9. CONCLUSIONS

As anticipated, the mélange has proven to be a highly variable mixture of blocks and matrix. However, ground behavior upon excavation has been more stable than anticipated with no significant movement and limited raveling. Except for very local occurrences, squeezing ground has not occurred, even in clayey ground with nearly 100 m of cover.

The ground support system has been adaptable and has allowed the contractor to utilize his preferred means and methods.

Groundwater inflows have typically been minor, less than about 0.3 lps.

10. ACKNOWLEDGEMENTS

Santa Clara Valley Water District owns and operates the Lenihan Dam. As part of the Jacobs Associates design team, geotechnical data gathering and evaluations are being carried out by Geomatrix, and on-going design support for the outlet and inlet works are the responsibility of Montgomery Watson Harza (MWH). The outlet tunnel and inlet shaft are being constructed by Drill Tech Drilling and Shoring (DTDS) as subcontractor to the project constructor, Flatiron Construction Corp (FCI).

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