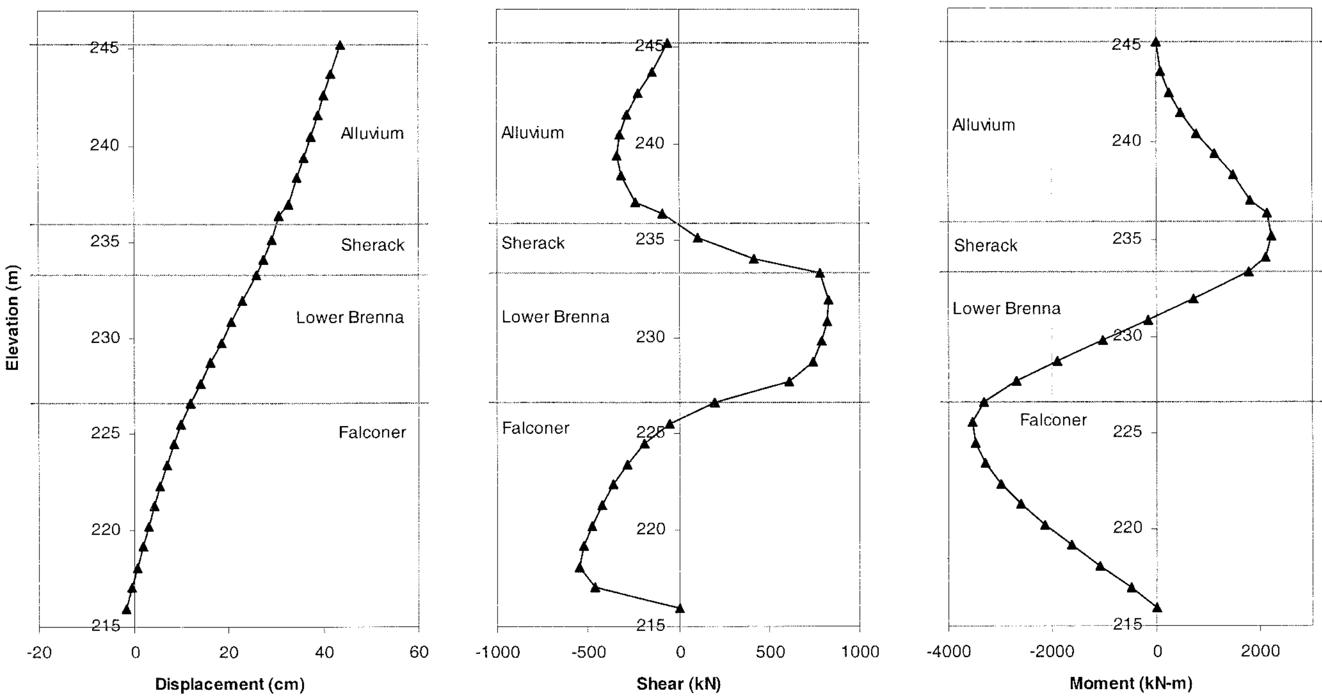

Figure 15. Pile Deflection, Shear, and Moment – Reeves Drive Site (S&W, 2002)

The final design included fifty-four 1.8-m- (6-foot-) diameter, 28.8-m- (94.5-foot-) long drilled shafts installed at two diameter center-to-center spacing to stabilize the slope subjected to loading from the flood control structure. The CPW was installed approximately 86 m (283 feet) down slope from the head scarp of the existing slide and the drilled shafts were designed to penetrate about 11 m (35 feet) into the stiffer Falconer Formation. The estimated deflection at the top of the CPW under working loads was calculated to be approximately 0.43 m (17 inches). We also recommended that a cap beam be installed to tie the shafts together and help distribute loads, which may reduce deflection and prevent failure in an individual shaft subjected to increased loading.

Construction, Monitoring, And Performance

Construction at the Reeves Drive site began in October 2002. The contractor set an oversize surface casing to a depth of approximately 6-7 m (20-23 feet), and then continued to drill the hole an additional 22.9 m (75 feet) to a final depth of 28.8 m (94.5 feet), as shown in Figure 16. The holes remained essentially dry throughout drilling operations. Each drilled shaft took approximately 3 hours to drill. Upon completion, the steel reinforcing cages were placed (Figure 17), and the holes were filled with water pumped from the Red River. Concrete then was placed by pumping from the bottom of the hole upwards with a constant 1.5-m (5-foot) head above the end of the discharge line, displacing the water back into the river (Riddick and Behling, 2004). As the pumped concrete filled the shaft hole, the displaced water flowed back to the river. Inclinometers were installed in two drilled shafts for long-term monitoring purposes. Construction of the Reeves Drive CPW and the flood barrier is now complete and is being monitored for performance.

Conclusions

Large landslide masses are difficult to stabilize with traditional methods of drainage, buttresses, and slope regrading. This paper discusses the use of a cylinder pile wall system to stabilize landslides along the Red River in Grand Forks, North Dakota. The paper presents the methodology and design for the wall using both limit equilibrium and numerical modeling methods. The results indicate that both limit equilibrium and numerical modeling provide similar factors of safety and failure surface locations for large landslide masses, where plane strain conditions control. However, numerical modeling techniques provide information on deflections and structural properties that cannot be calculated using limit equilibrium techniques.

Figure 16. Drilling Operations – Reeves Drive Site.

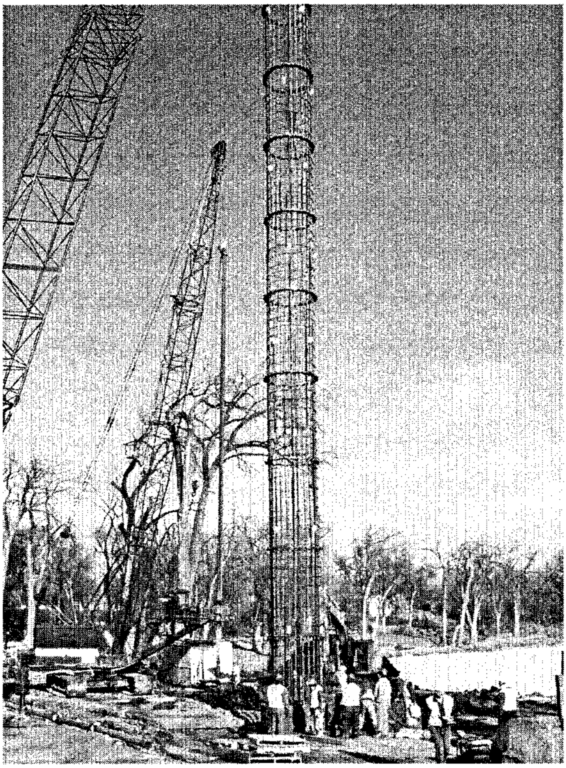

Figure 17. Placement of Steel Reinforcing Cages – Reeves Drive Site.

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Rehabilitation of the St. John's Tunnel

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Abstract

The St. John's Tunnel, constructed in 1903, is located on the south slope of Pikes Peak, Colorado at over 3,350 m (11,000 feet) elevation. The majority of the tunnel is unlined and, because of past instability, houses a 0.76-m (30-inch) -diameter steel pipe for conveyance. The tunnel is a critical element of the water supply system for Colorado Springs Utilities (CSU). Because of past rockfall and its future potential, CSU was concerned about the long-term reliability and integrity of water conveyance through the tunnel.

In the summer of 2003, CSU let a design-build contract to evaluate and rehabilitate the tunnel support problems. Because of difficult underground access, operational constraints, and the necessity to complete the work prior to inclement weather, the successful completion of the project required the adoption of a number of innovative construction methods. This paper describes tunnel geology, ground support recommendations (short mechanical bolts and steel sets), prioritization of improvements, and the unique scaling and rock bolting stabilization program undertaken.

Introduction

The St. John's Tunnel is located near Seven Lakes at high elevation in a remote area on the south slope of Pikes Peak in Teller County, Colorado. The design-build rehabilitation of this tunnel was performed by Mining & Environmental Services LLC (MES) as prime contractor to CSU. Shannon & Wilson, Inc (S&W) was retained as a subconsultant to MES to perform engineering services on the project.

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Project and Site Description

The 100-year-old tunnel is approximately 670 m (2,200 feet) long, approximately 2.0 m (6.5 feet) tall by 1.8 m (6 feet) wide, and houses a 0.76 m (30-inch) diameter steel pipe. Timber blocking is present beneath the pipe, raising it approximately 0.45 m (1.5 feet) above the tunnel invert, resulting in limited clearance over the pipe. The tunnel alignment has numerous minor deviations in size and straightness, resulting in alternating areas of near zero clearance between the pipe and tunnel ribs on the right and left sides.

The pipe conveys water from CSU's Reservoir No. 4 on the west side of a ridge to an unnamed drainage feature that feeds Lake Moraine on the east side of the ridge. Because the pipe inlet is submerged in the reservoir, access to the upstream portion (western portal) tunnel is gained only by descending a 2.4 m (8-foot) diameter vertical shaft. The downstream (eastern) portal of the tunnel daylights directly above the pipe outfall to the unnamed drainage feature. Access to the eastern portal is at ground level, via a steel arch supported adit portal.

The small cross section of the tunnel, combined with the limited clearances around the pipe and difficult access resulted in a challenging construction project that required innovative measures. The majority of the tunnel is unlined in the Pikes Peak Granite Formation. While this formation is generally competent, it has variable weathering, shear zones, and joint patterns that can develop kinematic instabilities. As such, CSU was concerned that a rockfall event within the tunnel could damage the pipe and disrupt water flow. Rockfall potential also posed a safety hazard to workers performing routine inspections and maintenance. In addition, the steel pipe is old and shows signs of corrosion and the timber blocking under the pipe was deteriorating. As such, CSU desired improvements to the tunnel and pipeline and initiated preliminary studies in 2001. The results of these preliminary studies (ME&T, 2001) indicated that significant cost would be required to correct all of the deficiencies and CSU did not have the required funds. As such, CSU commenced a limited scope design-build project to decrease the rockfall hazard within sections of the tunnel. Throughout the process the design-build team worked closely with CSU to complete the project within the limited budget and fast-track schedule.

Previous Studies

The tunnel and pipeline were previously evaluated by Mountain Engineering & Testing (ME&T) in a January 30, 2001 report, entitled "St John Tunnel Inspection and Pipeline Evaluation." This report included a pipeline survey, geologic mapping of the tunnel, evaluation of the pipeline condition, tunnel stability recommendations, water conveyance recommendations, and construction cost estimates (ME&T, 2001).

Site Reconnaissance

In August of 2003, two geotechnical engineers and a mining engineer performed a site reconnaissance within the St. John's Tunnel. This reconnaissance included a review of information in the ME&T (2001) report, identification of rock conditions and shear zones, assessment of ground support types, and prioritization of rockfall risk. Our identification of rock conditions was based on the categories and descriptions outlined by the International Society for Rock Mechanics (Brown, 1981).

In general, the data gathered during our reconnaissance concurred with the earlier ME&T (2001) report. A summary of our findings is as follows:

- Weathering: The degree of rock weathering was fresh to slighty weathered throughout the majority of the tunnel. However, several shear zones with highly to completely weathered rock were mapped, and there were also isolated areas of moderate weathering in the tunnel. The largest shear zone (approximately 12 m [40 feet] long) was supported by timber sets and timber lagging.
- Strength: Based on Schmidt hammer results and rock hammer resistance, the majority of rock within the tunnel was classified as medium strong to very strong (ISRM Classification R3 to R5). Similar to the degree of weathering as stated above, the material in the immediate vicinity of the shear zones was classified as very weak rock (ISRM Classification R1).
- Joints: Three major joint sets intersected the tunnel perimeter. Two of the joint sets had strikes roughly parallel and perpendicular to the tunnel with dips nearly vertical. The third joint set was nearly horizontal. Joint spacing varied significantly throughout the alignment. Because of the blocky and slabby nature of the rock, the tunnel shape is nearly rectangular in areas where the joint pattern is regular and the rock strength is high. Exceptions to this occur in areas where the joint sets are not as well defined, where shear zones are coincident with the tunnel sidewalls, and where the rock mass is highly to completely weathered. The condition of the joints varied throughout the tunnel. In general, in areas where the rock was fresh and strong, the joints were relatively tight, rough, and contained no infilling. In contrast, the tunnel also contained several shear zones in which the joints were smooth with clay infilling.
- Groundwater: Throughout the majority of the tunnel, the groundwater inflow was relatively small, typically consisting of minor drips through open joints.

Rehabilitation Alternatives

Three alternative strategies were developed for reduction of tunnel rockfall hazard and stabilization of the pipe. These alternatives were:

1) Encapsulation of the pipe with cellular concrete grout, by filling all or part of the annular space between the pipe and tunnel floor, ribs and roof;
2) Removal of the old steel pipe to allow improved access and working clearances for tunnel rehabilitation, with water conveyed on the tunnel invert and collected into a new intake structure at the east portal; and

3) Rehabilitation of the tunnel and pipe blocking, working within the limited clearances between the pipe and tunnel.

The first (cellular grouting) alternative had the advantages of easy and safe installation and effective support of both the pipe and the tunnel. The disadvantages were that CSU's budget could not be met with full encapsulation, pipe size could not easily be increased later, and a deteriorated old pipe with documented internal corrosion and turbulence damage would be permanently encased.

The second (pipe removal) alternative had the advantage of providing superior access, which would allow full rehabilitation of the tunnel at much lower unit costs. Engineering concerns included the need to evaluate scour along the tunnel invert, freezing concerns related to the east portal intake structure, and geotechnical stability of the rock slope around the east portal if the open channel flow introduced additional groundwater, and thus increased porewater pressure, into the rock joints. Additional concerns included the indeterminate level of effort and time required to pull the old asbestos and asphalt encapsulated heavy steel pipe out of the tunnel, especially with the imminent onset of winter conditions.

The third rehabilitation alternative, including working around the pipe, had issues related to productivity and safety because of the limited working clearances. Transportation of materials would be difficult, and drilling and scaling operations would be taking place in a very confined space. Special measures would be needed to protect the pipe from rockfall, as replacement of a section within the tunnel would be nearly impossible.

After consideration of the alternatives, and in light of available budget and schedule constraints, CSU elected to proceed with conventional tunnel rehabilitation working around the existing pipe. There were two major types of ground support recommended for the tunnel, as discussed in the following section, including: 1) scaling and installation of rock bolts (both pattern and spot bolts) for the majority of the tunnel, and 2) removal of existing timber sets in the major shear zone and installation of steel sets.

Geotechnical Analysis of Typical Tunnel Support Requirements

As discussed earlier, three major joint sets intersected the tunnel perimeter. The orientation of these joints relative to the tunnel trend and plunge forms generally small rock wedges in the tunnel sidewall and crown. The near horizontal joint set also forms 15- to 30-cm (6- to 12-inch) -thick rock slabs in the crown at various locations in the tunnel.

Using the average orientations for the three joint sets, we performed kinematic analyses to estimate the size and weight of the largest permissible rock wedge that could develop in the tunnel crown and sidewall. We also evaluated other possible rock loading conditions by considering a continuous triangular rock wedge in the crown of the tunnel with a wedge height equal to one-half the tunnel diameter. These analyses indicated that the maximum rock wedge requiring rock bolt support would weigh approximately 33.5 kN/m (2,300 pounds per foot) longitudinally along the tunnel.

To evaluate spot bolting requirements, we considered a finite wedge that would be supported by a single bolt. For this step, we analyzed both a pyramid-shaped wedge 1.2 m (4 feet) by 0.9 m (3 feet) by 1.07 m (3-½ feet) high and a cone-shaped wedge 1.07 m (3-½ feet) high with a 1.2 m (4-foot) diameter. Both rock wedge shapes resulted in a load of approximately 10.7 kN (2,400 pounds) that would be supported by a single bolt.

Ground Support Recommendations for Shear Zone

As previously discussed, a timber supported shear zone was observed in the tunnel (Figure 1). This shear zone defined the south sidewall beginning near one end of the timber supports, crossed the crown in the vicinity of the start of the timber sets and extended into the sidewall at the other end of the timber sets. The shear zone was estimated to be about 15-cm (6-inches) thick, 5 cm (2 inches) of which is filled with clay. The shear zone separated the very low strength and completely weathered granite on the south side (footwall) of the shear from the moderate to high strength, fresh to slightly weathered granite, which occupied the north or hanging wall side of the shear.

Figure 1. Timber Support in Shear Zone.

To determine the ground support requirements in this area, we used the Q System (Barton et. al., 1974), which is an empirically based rock mass rating system that is commonly used to estimate final rock support requirements for new tunnels. The numerical value of Q varies on a logarithmic scale from 0.001 to 1,000 and is defined as:

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF}$$, where:

RQD is the Rock Quality Designation
Jn is the joint set number
Jr is the joint roughness number
Ja is the joint alteration number
Jw is the joint water reduction factor
SRF is the stress reduction factor

To relate the Q value to ground support requirements, an Equivalent Dimension (expressed in meters) is defined as the width of the underground opening, divided by the excavation support ratio (ESR). The value of ESR depends on the ultimate use of the underground opening and the time of exposure. The value of ESR ranges from 0.8 for underground railroad stations or sports arenas to 3 to 5 for temporary mine openings. An ESR of 1.3 was selected for this project, which is recommended for minor road and railway tunnels and access tunnels.

The values of the input parameters were as follows:

RQD = 10 (RQD values were not available, so the minimum value was used – consistent with completely weathered rock);
Jn = 9 (three joint sets);
Jr = 1 (the zone containing clay minerals is thick enough to prevent rock wall contact);

Ja = 8 (zones or bands of disintegrated or crushed rock and clay)
Jw = 1 (Dry excavations or minor inflow less than 1.3 gallons/minute)
SRF = 1 (single weakness zones containing clay or disintegrated rock)
Span = 2 m
ESR = 1.3
Equivalent Dimension = 1.5

Using the above input values, the calculated Q is 0.14, which corresponds to very poor rock. Using this value and the Equivalent Dimension, the recommended ground support (Barton et. al., 1974) consisted of tensioned bolts installed on a 1 m (3.2-foot) spacing with 5 mm (2 inches) of mesh reinforced shotcrete. Grimstad and Barton (1993), recommend 7.6 mm (3 inches) of fiber reinforced shotcrete with 1.2 m (4-foot) long bolts spaced 1.2 m (4 feet) apart for similar parameters.

As indicated above, bolting alone without shotcrete would be insufficient in this reach. Because of the small tunnel size and distance from the east portal and west shaft, it would be difficult to install shotcrete in this area.

As an alternative, we recommended that steel sets blocked with timber and fully lagged with steel channel be considered as an alternative to rock bolt and shotcrete ground support for this area. To prevent further deterioration and fallout of the completely weathered granite, it would be appropriate to backfill with cellular or