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SYNOPSIS: At the Rocky Mountain Pumped Storage Project a 12 meter diameter power tunnel was excavated through sedimentary rock for 760 meters. Approximately 10% of this tunnel was through Pennington shale that is described as a dark gray massive organic shale. This paper will describe the methods of testing and rock characterization, the results of instrumentation and monitoring, and the post-construction testing program for the excavation, and conclude with a discussion of the observed rock response in relation to the measured strength and deformation properties. This particular zone of the tunnel required the addition of longer rockbolts, and a discussion of that supplemental rock reinforcement will be included.

INTRODUCTION

The Rocky Mountain Project is a pumped storage facility rated at approximately 800 megawatts of peaking energy. The project is located approximately 16 km (10 m) northwest of Rome, Georgia, in the Valley and Ridge Physiographic Province.

Construction of the project required the excavation of a 12-m (39.5-ft) tunnel, 640-m (2,100-ft) long. Construction also included the placement of a 10.7-m (35-ft) diameter reinforced concrete liner in the tunnel. The construction of the water tunnel has been completed.

The zone of anticipated low strength occured along a 111.3-m (365-ft) stretch of the tunnel starting approximately 193.5-m (635-ft) upstream from the portals.

The investigative program included geological mapping, rock borings, geophysical testing, laboratory and field testing of core samples, construction of an exploratory adit and test chamber.

Design of the excavation and rock reinforcement will be discussed. During the excavation representatives from engineering mapped the geological conditions, observed the rocks response to excavation, and monitored rock movements with instrumentation of the excavated surfaces.

Field tests of in-situ rock properties were made during the excavation of the tunnel.

INVESTIGATION PROGRAM

From 1977 through 1979, a series of borings were drilled along the tunnel alignment to determine the geological conditions. Geophysical data were acquired using crosshole velocity measurements, uphole velocity surveys and downhole measurements (sonic velocity, gamma radiation and borehole verticality). Water pressure testing was conducted in selected borings.

An exploratory adit was excavated during 1978-1979 above the centerline of Penstock no. 2. A test chamber, the same diameter as the upper half of the planned tunnel, was constructed at the end of the exploratory adit to provide information on the behavior of the rock during and after excavation. This test chamber also revealed additional rock exposure for the measurement of structural features and geological conditions.

During the fall of 1980, a detailed field and laboratory program was undertaken to define the rock mass properties for use in the design of the tunnel and shaft. Two additional core holes were located off the tunnel centerline and carefully oriented and angle-drilled to: 1) obtain rock conditions in the bifurcation area, 2) intersect steeply-dipping joints at an angle that was appropriate for lab testing, and 3) assess the frequency and spacing of steeply-dipping joint sets. Detail line surveys were conducted to determine fracture extent, spacing, opening, and orientation. Seismic refraction surveys were conducted along the walls of the exploratory-drainage adit to determine the in situ velocity, elastic modulus, and thickness of loosened zones.

During the excavation of the tunnel, plate jacking tests and cross-hole and downhole geophysical tests were conducted in shale and siltstone units from the upper bench excavation to determine in-situ rock properties.

LOCAL GEOLOGY

Stratigraphy

Paleozoic rocks of Pennsylvanian and Mississippian age occurred along the shaft and tunnel alignment and were identified as 17 distinct units. The zone of interest is a

25.9-m (85-ft) thick layer of moderately-hard, massive-bedded, organic shale with very thin sandstone laminae and siderite layers (unit 16). Below the zone is a 100-m (330-ft) thick very-hard, massive bedded, medium- to course-grained, fossiliferous crystalline limestone (unit 17). Above the zone is a 19.8-m thick (65-ft) moderately-hard, thin- to medium-bedded, clayey siltstone (unit 15).

Structure

The strength and stability of a rock mass is highly affected by the discontinuties: bedding, joints, faults, and shear zones. Measurements indicated that bedding in unit 16 at tunnel grade would strike N72°E and dip 13° to the northeast (perpendicular to the tunnel axis). Jointing occured in two sets: 1) a predominant set with a strike of N 60-70°E and dipping 70-75°NE and 2) an infrequent set with a strike of N 30-40°E and dipping 30°NE. The investigation program did not indicate possible faults or shear zones.

Ground Water

The ground-water conditions for the tunnel are topographically, lithologically, and structurally controlled. Water pressure testing of the shale, unit 16, indicated very low permeabilities with no measurable water take. The overlying siltstone, unit 15, had a permeablity of approximately 5.0×10^{-5} cm/sec.

ROCK PROPERTIES

Rock properties from field and laboratory testing are presented in Table 1.

DESIGN

The design criteria for the tunnel excavation was to provide a stable opening during and after construction for the placement of the concrete lining. The design was also intended to minimize the disturbance of the surrounding rock mass so that it would be able to resist excessive deformation of the tunnel lining upon internal pressurization. The excavation was stabilized for loadings due to gravity, blasting vibrations, hydrostatic forces due to ground water in joints and bedding planes, and additional loads caused by destressing the rock as the excavation proceeded. The rock stabilization designs take these conditions into account, while retaining the flexibility required to handle unexpected rock conditions.

Excavation of the tunnel was divided into a top heading and bench operations to limit raveling of the excavation face along the N60-70°E joint set. The philosophy of design was that by using rock reinforcement the rock became self supporting. Rock stabilization design was based on average conditions for two groups of rock, classified simply as soft rocks and hard rocks. A primary bolt pattern was designed for each group. Bolt length and spacings were chosen using standard rules of

TABLE 1

Property	Design Values		Excavated Values	
	Unit 16-Shale	Unit 15-Siltstone	Unit 16-Shale	Unit 15-Siltstone
Point load-diam.	8.89 MPa	7.72 MPa		
Point load-axial	30.06 MPa	62.1 MPa		
Brazilian tensile strength	5.14 MPa	5.16 MPa		
Unaxial compressive strength	52.54 MPa	57.02 MPa		
Triaxial compressive strength @ 300psi	46.45 MPa	66.97 MPa		
Direct shear strength @ 300psi	1.56 MPa	1.97 MPa		
Density	2.67 Mg/m^3	2.67 Mg/m^3		
Dilation	0.15 cm	0.26 cm		
Cohesion	5.51 MPa	9.31 MPa		
Angle of friction	38.5°	51°		
Youngs modulus-lab	14.06 GPa	17.65 GPa		
"P" velocity-field cross-hole	5,212 m/s	4,840 m/s	4,460 m/s	4,175 m/s
"S" velocity-field cross-hole	2,150 m/s	2,360 m/s	1,900 m/s	2,209 m/s
"P" velocity-uphole	3,800 m/s	4,200 m/s	3,505 m/s	4,511 m/s
Youngs modulus-dynamic from cross-hole	30.68 GPa	37.16 GPa	23.96 GPa	34.06 GPa
Youngs modulus of intact rock-seismic	7.67 GPa	9.29 GPa	5.99 GPa	8.51 GPa
Youngs modulus of loosened zone-seismic	1.92 GPa	2.32 GPa	1.50 GPa	2.13 GPa
Youngs modulus of loosened zone-jacking test	--	--	2.76 GPa	4.14 GPa
Blast loosened zone	0.91-1.52 m	--	0.30-0.60 m	0.0-0.30 m
Rock mass rating	47	43	55-67	54-68
Rock mass strength determination	--	--	180-500	500-580

thumb. Checks were then made to assure that the design would reinforce joints against shear movement, prevent wedge failure in the roof, and restrain the loosened zone around the opening. A supplemental bolt pattern was designed for both support systems for use in problem areas.

For the section of tunnel discussed, the primary bolt pattern was installed on a 1.7-m (5.5-ft) spacing radially through the upper 300°. However, in the crown the bolts were installed into the walls 45° above the horizontal. The bolts were 3.7-m (12-ft), No. 11, Grade 60 reinforcing bars that were fully-resin-grouted. Shotcrete and mesh was placed to 30° below the springline. When necessary, the bolts in the supplemental pattern were placed between the primary pattern reinforcing. In addition spot bolts were placed as necessary. During construction the reinforcing pattern was modified to decrease the reinforcing cycle.

RESPONSE TO EXCAVATION

Initial inspections of the low strength shale zone indicated low permeabilities, thus only minor water flow was anticipated into the excavation. However, as the excavation progressed, wet conditions were noted along the shale bedding. The wet conditions generally reduced the efficiency of shotcrete application to the rock surface and increased the frequency of air slaking along bedding.

Excavation confirmed that bedding and the N60-70°E joint set were the dominant structural discontinuities in the low strength shale zone. Fractures present within the zone were commonly masked due to blast shattering along bedding of the fissile shale. The separations along bedding and the N60°-70°E joint set locally resulted in flat breakage in the crown and quarter arches, reducing the natural stability offered by an arch geometry. In several areas continued unravelling of the rock yeilded steeple-shaped overbreaks which ranged up to 1.5-m (5-ft) beyond the designed excavation limits.

The contractor had difficulty installing the specified rockbolts, wiremesh and shotcrete in the required eight hours. The excavation blast rounds were shortened by the contractor to 2.4 meter (8.0-ft) rounds in lieu of 3.0 meter (10.0-ft) rounds. Continued slaking and spalling of the low strength shale zone eventually required the shortening of blast rounds to 1.5 meters (5.0-ft). In order to increase the efficiency of the tunnel advance and the need for immediate sealing of the rock surface a phased shotcrete application was recommended with 3.0 meter blast rounds. The phased shotcreting required the immediate application of two inches of shotcrete after excavation and rock bolting. Then, following rockbolt and wiremesh installation, a final two inch coat of shotcrete was added. To facilitate continued excavation, lagging of the wire mesh and second shotcrete application was allowed in areas of the shale zone where more stable ground was encountered.

Cracking and sagging of the shotcrete was common throughout the low strength shale zone. One major problem with the shotcrete was its failure to obtain a quick initial set, resulting in inadequate bonding to the rock surfaces. It was also difficult to achieve the application of a wet-shotcrete mix on to an already wet rock surface. Shooting the wet shotcrete to the crown and quarter arches often resulted in a 20-40 percent rebound of the mix. Slaking of the rock after shotcrete application commonly resulted in cracking of the shotcrete. To provide a method of measuring continued cracking and sagging spackling compound was applied to shotcrete cracks and periodically inspected to evaluate the extent and rate of movement.

To alleviate overbreak, pre-blast support spiling-bolts were installed at low angles to the crown and quarter arch surfaces. In general the spiling bolts were only partially effective in the shale zone and some were shot out with the next blast round. Finally, in two locations, continued unravelling of rock and shotcrete spalling required the installation of channel-iron mine straps for support along the crown.

INSTRUMENTATION

For safety and to verify the design assumptions, the tunnel was instrumented during construction to monitor the behavior of the rock mass. The instrumentation needed to be simple to install, durable, and allow rapid data collection and interpretation. The instrumentation also had to be sophisticated enough to allow differentation between various types of deformation.

The instrumentation system originally planned utilized two types of instruments; convergence points and extensometers. Due to problems with loss of convergence points their use was discontinued.

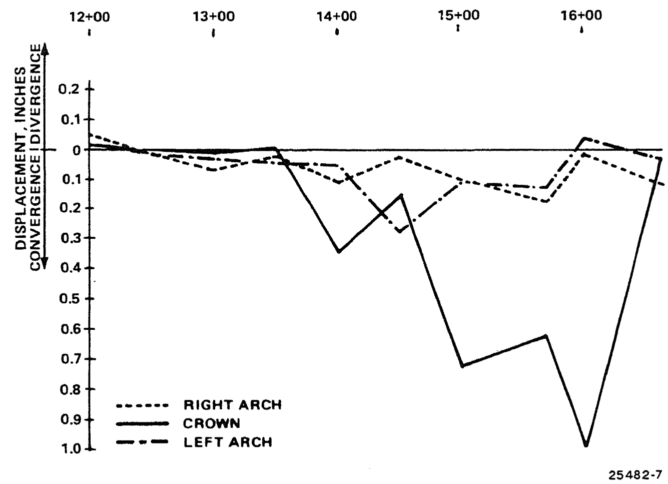
The second type of instrument used was the borehole extensometer. The single point borehole extensometers installed in the tunnel consisted of a No. 8, Grade 60 reinforcing bar, with a grouted anchor up to 0.6 m (2-ft) long installed in a 0.04 m (1 1/2 in) diameter hole, with a collar grouted to the rock at the wall of the tunnel. Measurements of the relative movement between the collar and the bar were taken using a depth micrometer. Borehole extensometers were originally intended to be installed in the crown and quarterarches at approximately 61 m (200-ft) intervals along the main tunnel axis. Each location was to have 2 borehole extensometers installed, one 1.5 m (5-ft) deep and one 4.6 m (15-ft) deep. However, in order to allow the phased shotcrete application to lag behind the advancing face, additional instrumentation was required to more closely monitor ground behavior. To achieve this requirement, borehole extensometers were installed 4.6 m (15-ft) deep in the crown and arches at 15.2 m (50-ft) intervals along the tunnel, in addition to the originally designed dual installations every 61 m (200-ft). Before the bench was excavated, 4.6 m (15-ft) long borehole extensometers were installed in the spring-lines, at the same stations as the crown and arch instruments.

The schedule for reading the extensometers was daily until the face had advanced a distance of 2 tunnel diameters past the instrument locations, weekly until the movements appeared to have ceased, and monthly after apparent stabilization, or as often as necessary to confirm the stability of the rock mass. This monitoring program was continued until the lining was placed. After each reading was taken, it was compared to previous readings. If a large difference was noted, the reading was immediately checked. In some extensometers blast-induced disturbances of the reference collars indicated false movement. If a discrepancy persisted, readings were taken every shift until the problem was resolved or explained. Plots of deformation versus time were primarily used to evaluate the instrument data. Other plots, such as deformation versus depth and deformation versus distance from the working face, were used as necessary to complete the picture.

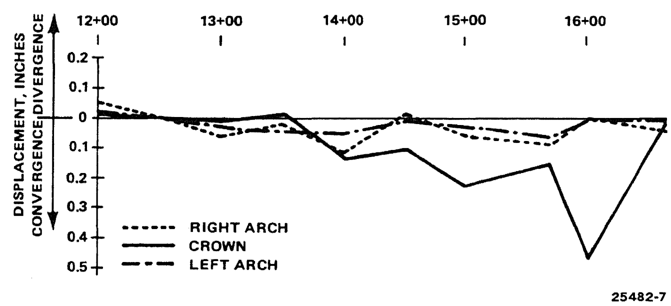
Based on calculations as well as professional judgement, it was decided that wall movements (relative to the 4.6 m (15-ft) deep extensometer anchor) of up to 5.1 mm (0.2 in) would be considered allowable, and those over 7.6 mm (0.3 in) would be considered a sign of possible instability and excessive loosening. In areas where the movements exceeded 7.6 mm (0.3 in), treatment was based on a thorough analysis of the cause of the movements.

As shown in the figures, the movements recorded at instrumented stations in the shale were some of the largest observed anywhere along the tunnel . At station 16+00 the total movement in the crown was slightly over 25 mm (1.0-in). Approximately one half of this movement occurred during the excavation of the top heading. As the bench was excavated past station 16+00, the movements were again triggered and convergence proceeded at an undesirable rate. Station 16+00 was located at the interface between the shale and siltstone. When excavated, this area was very blocky and wet. The data indicated that significant movements took place at depths greater than 1.5 m (5-ft) from the excavated surface and possibly deeper than 4.6 m (15-ft). These movements were probably the result of loosening and moving of rock blocks, especially in the crown area. This movement may have been partially driven by water pressure in the rock. Nine meter (30-ft.) long tensioned rock bolts were installed in the converging area, and were successful in stabilizing it.

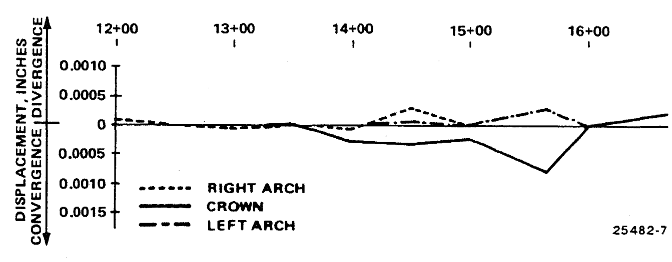
In general, the springline instruments showed that very little movement took place as the bench was excavated. Measurements of convergence that were recorded were determined to be the result of loosening of very near surface rock slabs or shotcrete.



TOTAL DEFORMATIONS MEASURED AT 15 FOOT DEEP EXTENSOMETERS



DEFORMATIONS MEASURED AT 15 FOOT DEEP EXTENSOMETERS AFTER BENCH EXCAVATION



DEFORMATION VELOCITY MEASURED AT 15 FOOT DEEP EXTENSOMETERS

DISCUSSION

The results of the pre-construction exploration indicated that the shale would be the weakest material encountered in driving the tunnel. It was felt, based on the test chamber in the adit and the measured rock properties that if the top heading could be reinforced with the specified rockbolts, wiremesh, and 10 cm (4-in) of shotcrete in the required 8 hours there would not be a problem in excavating through the shale. If problem areas were encountered the supplemental pattern of bolts and spot bolts could be used.

The contractor had difficulty installing the rock reinforcing within the specified 8 hours after blasting. This resulted in a loosening of the crown and slabbing of the shale. The contractor elected to shorten the advance rounds to meet the 8 hour requirement for installation of reinforcement.

The shortened round impacted the schedule and the design was modified to allow installation of rockbolts and 5 cm (2-in) of shotcrete within 8 hours of blasting. The application of wiremesh and the remaining 5 cm (2-in) of shotcrete would lag behind the advance. This modification in design was allowed only with an increase in instrumentation to monitor rock movement. If movement occurred the wiremesh and remaining shotcrete was to be applied immediately.

In general, the shale behaved as anticipated. The problems encountered, rock covergence and additional tensioned bolt support, may have been avoided if quality shotcrete, controlled blasting and timely reinforcement could have been achieved. It was unfortunate that the weakest material of the tunnel was one of the first encountered. The designer, contractor, and owner were still learning to work together.

As a result of the movements that occurred, post-excavation testing was performed. The two main parameters of concern were the deformation modulus of the rock and the depth of the loosened zone. The results of this testing is presented in Table 1 and was within the required limits used in the tunnel lining design. The tunnel was lined in 1984 and no distress has been observed since that time.

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