CONSTRUCTION MONITORING ACTIVITIES IN THE ESF STARTER TUNNEL

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ABSTRACT

In situ design verification activities are being conducted in the North Ramp Starter Tunnel of the Yucca Mountain Project Exploratory Studies Facility. These activities include: monitoring the peak particle velocities and evaluating the damage to the rock mass associated with construction blasting; assessing the rock mass quality surrounding the tunnel; monitoring the performance of the installed ground support; and monitoring the stability of the tunnel. In this paper, examples of the data that have been collected and preliminary conclusions from the data are presented.

I. INTRODUCTION

An underground test facility known as the Exploratory Studies Facility (ESF) is planned as part of the characterization of a site for a potential high level nuclear waste repository at Yucca Mountain, NV. The North Ramp Starter Tunnel (NRST) is the first underground portion of the ESF to be constructed. The NRST is approximately 200 ft long, with a horseshoe shaped cross-section, approximately 30 ft high and 30 ft wide.

Geotechnical monitoring of the NRST was conducted in order to confirm design and construction methods and to provide data for long term performance evaluation. The monitoring activities presented in this paper are grouped into three tasks:

1. evaluation of mining methods,
2. monitoring of ground support systems,
3. monitoring of drift stability.

This paper presents a brief description of the monitoring activities and methods used, a sample of typical results, and preliminary conclusions on the construction methods and ground support design for the NRST. Additional data reduction and interpretation are ongoing.

II. DESCRIPTION

A. Evaluation of Mining Methods

The NRST was excavated by drill and blast methods using a sequence of small headings. The top heading consisted of several rounds of blasting. These headings were designed so that the pilot drift would remain several rounds ahead of the slashes. The lower bench was excavated after the top heading was completed. It was also excavated in 3 sections: a center bench, a north bench and a south bench. Smooth wall blasting techniques were used for each of the headings. The different headings are illustrated in the cross section of the tunnel shown in Figure 1.

Evaluation of this mining method is being conducted by correlating blasting, rock mass quality, blast damage, and opening stability. This work will provide a basis for any future drill and blast construction that may be performed in the ESF. Data for this evaluation was gathered in the three following subtasks:

1. monitoring of blasting activities,
2. blast damage assessment,
3. rock mass quality estimation.

1. Monitoring of the blasting activities

Monitoring of the blasting activities consisted of: monitoring blast induced ground vibrations; collecting data from the blast, including blast design and field implementation of that design from the constructor; and recording the condition of the excavation surface to document the results of the smooth wall blasting. Monitoring of blast induced ground vibrations was conducted using two blasting seismographs, each of which produce an analog record of the seismic ground velocity, and also calculate the peak particle velocity (PPV) of the ground from the blast.

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2. Blast damage assessment

Blast damage was estimated using the following: the number of half cast blast holes evident on the excavation surface, the number and frequency of blast fractures on the excavation surface, and the depth of blast induced fracturing away from the excavation surface.

In smooth wall and presplit blasting it is common for the perimeter holes to leave half casts. When half casts are present it is a direct indication that no overbreak has occurred. As part of the evaluation of blasting, the number of half casts were recorded for each round. The degree of overbreak, if present, was not quantified.

A secondary method of evaluating blasting involves evaluation of the rock mass conditions. Geologic parameters were gathered as part of the rock mass quality estimation that was conducted in the NRST. Rock mass quality estimation is a method of evaluating ground conditions based on geologic parameters, stress, strength, hydrologic parameters, etc. It also incorporates conditions caused by blast damage (i.e., blast fracturing). Further discussion of the rock mass quality estimates follows in the section below.

Work was also done to directly quantify the amount of blast damage and the extent of the blast damage zone. Instrumentation boreholes were drilled for the installation of multi-point extensometers (MBPXLs) and stress gages (as discussed below). Before the instruments were installed, the holes were borescoped and the images recorded on video with the intent of determining if there is any increase in the amount of fracturing observed in the rock near the tunnel opening.

3. Rock mass quality estimation

Rock quality estimates were conducted independently in the top heading and the lower bench portion of the NRST. Rock mass classification systems are generally accepted indices used to evaluate ground conditions and rock support. They are empirical indices based upon case studies of mining, civil, and other underground excavations. The Q system by Barton and the Rock Mass Rating system (RMR) by Bieniawski were the classification systems selected.

One of the primary parameters in the system is the estimation of rock quality designation (RQD). RQD is typically derived from drill cores. In this application it was necessary to determine an equivalent RQD in the tunnel. A method employing linear fracture frequency in the tunnel ribs and face was used to determine a volumetric fracture frequency. A relationship developed by Hudson & Priest was used to estimate the tunnel RQD from the fracture frequency.

The Q system incorporates parameters such as the RQD; joint properties, such as the number of joint sets, roughness, and infilling; hydrologic condition; and stress condition. Q values may vary from $10^2$ to $10^3$ representing exceptionally poor to exceptionally good ground conditions. The data base for the Q system allows comparison to rock support systems used for excavations with similar Qs.

The RMR system also uses RQD, hydrologic conditions, and joint properties. Differences lie in the use of rock strength, joint spacing, and joint orientation. RMR values can range from 0 to 100 representing very poor rock to very good rock respectively.

The Q and RMR systems can be directly compared by using a general empirical relationship:

$$RMR = 9 \ln Q + 44$$

A site specific relationship for the two systems may be developed in the future when more data become available.

B. Monitoring of Ground Support Systems

The ground support used in the NRST consists primarily of bolts, mesh, and shotcrete (steel fiber type). The portal area ground support also incorporated the use of lattice girders in the shotcrete. The sequence of rock support coincided with the excavation sequencing. The initial support system used 10 ft Split Set rock bolts and 6X6 inch wire mesh. This support was installed shortly after each round. After several rounds had been shot, approximately 20 to 40 ft of advance, shotcrete was applied. Shotcrete specifications required a minimum of 6 inches be applied. The final component of rock support was the installation of pattern bolts. Cement grouted bolts were installed on 5 ft centers throughout the facility. The final design used 1 1/8 inch diameter bolts with lengths ranging from 8 to 24 ft.

Rock Bolt Load Cells (RBLCS) were installed to monitor the performance of rock bolts in the tunnel. A total of fifteen bolts at 5 stations in the tunnel, and 3 bolts on the high wall above the portal were instrumented with RBLCS as shown in Figure 1. The standard fully grouted pattern bolts were used for these installations. The cells were passively installed, applying only a nominal initial load of approximately 10% of the bolt strength. The bolts have and will continue to be monitored over the long term to determine if any load transfer to the bolts is occurring. This data coupled with ground movement information described below will be used to determine the effectiveness of the ground support, and will also contribute to the tunnel design verification process. This type of RBLCS installation (i.e., using fully grouted tensioned bolts) will be evaluated as part of the data reduction process.

C. Monitoring of Drift Stability

Long term monitoring of drift stability is being conducted using three types of instruments. Convergence pins were installed on the tunnel perimeter to monitor rock movement at the surface of the opening. Multipoint Borehole Extensometers (MBPXs) were installed to measure ground movements at depth in the rock. Borehole stress gages will be installed past the end of the starter tunnel to monitor stress changes in the rock.

Convergence measurements are being used to monitor the closure of the tunnel. Measurements began with the early stages of excavation using temporary construction monitoring.
stations for worker safety. Five permanent stations have consequently been installed to monitor the long term ground movements. (Convergence measurement stations at 5' and 25' shown in Figure 1 are two of the temporary construction monitoring stations.)

Three MPBXs were installed to monitor long term ground movements and to measure the relative motions within the rock mass. Data from these instruments consist of differential ground movements from the tunnel surface (borehole collar), to each anchor point in the borehole. A vertical and a horizontal MPBX were installed at the location of the deepest permanent convergence station. A vertical MPBX was installed at the location of the next deepest permanent convergence station.

Six stress gages will be installed past the end of the NRST. The NRST will be extended using a tunnel boring machine (TBM) to create the North Ramp of the ESP. The stress gages will be installed before the TBM begins to excavate, in boreholes located outside of the area that is to be excavated by the TBM.

The combination of convergence pins, MPBXs and stress gages will allow the measurement of ground motions near the face of the tunnel and the measurement of stress changes at two points outside the region that is excavated. Changes in stress and deformation will be monitored when the tunnel is extended by the TBM. Long term changes will also be monitored to establish tunnel stability and the impact of potential seismic ground motions on the tunnel.

Knowledge of both the stress and displacement (strain) can contribute to calibration or validation of numerical models of the openings. This in turn enables the designers to evaluate the performance of the tunnel design.

III. Results

A. Evaluation of Mining Methods

Blast vibrations were monitored to better understand the grounds reaction to blasting and therefore better design blast rounds, ground support, and the openings. Design criteria to minimize ground vibrations were developed as part of the blast design, therefore ground vibrations were monitored from each blast round. Typically the seismographs were placed near the blast, within 150 ft, where practical (i.e., out of fly rock areas, haulage ways, etc.). Two separate locations approximately 50 ft apart were selected to obtain data at different distances from the blast.

Each blast design incorporated up to 16 delays, and had a total firing time up to approximately 10 seconds. Field implementation of the rounds would vary as conditions dictated. Conditions such as overbreak or underbreak from the previous round, or atypical fracture zones would require the constructor to modify the round as appropriate. Changes to the round were documented by the constructor. These data were then used for analysis of the blasting data.

Each of the two blasting seismographs included a triaxial geophone and recording unit. The triaxial geophone was used to record ground velocities in three perpendicular directions. The velocities for each axis were stored in analog form and printed on strip charts. The seismographs also calculated a maximum peak particle velocity (PPV) for each event using a vector sum calculation of the three axes.
A preliminary analysis of the blasting data was conducted using only the maximum PPV for each round. Further analyses can be conducted by considering the blast associated with each delay. The separation common for tunnel delays (0.1 to 1 sec.) makes it feasible for each delay to be considered separately and multiple PPVs calculated for each round, thereby increasing the data available for each blast.

Analysis of the data was conducted by the traditional scaled distance approach. In this approach, the maximum PPVs from different blasts are compared using scaled distances, which incorporate the effect of the travel distance from the blast to the geophone and the effect of the amount of explosive charge used. The scaled distance, SD, is calculated from:

$$SD = \frac{R}{W^{1/2}},$$

where

- $R =$ the distance from the charge to geophone (ft), and
- $W =$ the weight charge of explosive (lbs).

Figure 2 presents a scatter plot of the blast data for the top heading of the NRST. For comparison, the standard equation commonly used for general ground conditions has also been plotted. This equation results from a large number of field measurements. This general propagation equation can be expressed as:

$$PPV = 160/R^{1/2}W^{1/2},$$

where PPV is in (in/sec).

![Figure 2: Peak Particle Velocity versus Scaled Distance](image)

Note that the data do not appear to fit the general equation for the near field, i.e. at an SD of less than 30 ft/ft$^{1/2}$. The near field data indicate that jointed tuff at the NRST has a higher attenuation than predicted by the equation.

When all data are compiled and reviewed it is anticipated that a site specific equation of attenuation for the tuffs at Yucca Mountain can be established. This could then become a valuable design tool for further blast excavations in the ESF. These data may further be used in assessing natural seismic wave attenuation and the effects of seismic waves on repository design.

A count was made of the blast hole half casts left from each round. None of the rounds in the top headings left half casts, as overbreak was a continual problem. Approximately 17 out of 25 rounds in the bench headings left half casts. Here overbreak was not as significant. There was a good correlation between the locations of half casts and the rock quality estimates discussed below.

The evaluation of the blast damage zone to this point is inconclusive. The observations from boreholes to date, indicate that the ground has zones of considerable fracturing and vugs located randomly along their length. The spatial variability of these zones and the quantity of fractures and vugs have masked any identifiable fracturing due to blasting, making it impossible to quantify the extent of the blast damage zone.

Results of the rock quality estimates are shown in Figures 3-5. The Q rating of the rock ranged from 0.05 to 17.4 representing extremely poor to good ground conditions. The RMR ratings were 34 to 68 spanning poor to good ground conditions. Note, that due to the variability of the rock and the nature of rock quality estimates the Q and RMR estimates are expressed as a range and not a singular value. The presence of shear zones, large voids and joints with large apertures result in the lower bound rating. The upper bound ratings are indicative of areas with competent rock, and tight joints with little or no infilling. As indicated in the figures, there was a substantial improvement in ground conditions with depth. The bench was observed to have fewer open vugs, and tighter joints. Improvements from the portal to the end of the tunnel were similarly noted. Generally the upper bound readings improved substantially, while the lower bound readings remained relatively constant, as small fracture and shear zones were still present, although less predominant.

![Figure 3: Q Estimates for the Top Heading](image)
B. Monitoring of Ground Support Systems

The RBLCs have been measuring the load histories of 15 bolts in the tunnel and 3 bolts on the high wall. Figures 6 and 7 are load histories for bolts at the high wall and the 55 ft station, respectively. The bolts on the high wall showed an initial decline in load for the first 2 months and now appear to be stable at their current load. The bolts in the tunnel have typically stabilized more rapidly, within a few weeks. Monitoring of these RBLCs will continue long term, to supplement other data for design verification.

C. Monitoring of Drift Stability

Figures 8 and 9 are closure readings from 2 stations in the tunnel. Some steady state convergence is typical for openings of this size. In cases such as these, convergence rate is commonly monitored, as being more indicative of ground control problems. Convergence rate and changes in convergence rate are being calculated as part of this monitoring process.

The MPBX stations have just recently been completed, therefore data from these instruments are not yet available.
Information from stress gages and MPBxs have not yet been obtained, but will be in the near future. This data is expected to be of use in establishing the response of the ground to excavation, when the tunnel is extended with the TBM. Knowledge of both the stress and displacement will contribute to the validation of numerical models of jointed tuff.

References


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