DEVELOPMENT OF EXCAVATION TECHNOLOGIES
AT THE CANADIAN UNDERGROUND RESEARCH
LABORATORY

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Abstract

Several countries, Canada being among them, are developing concepts for disposal of used fuel from
power generating nuclear reactors. As in underground mining operations, the disposal facilities will
require excavation of many kilometres of shafts and tunnels through the host rock mass. The need to
maintain the stability of excavations and safety of workers will be of paramount importance. Also,
excavations required for many radioactive waste repositories will ultimately need to be backfilled and
sealed to maintain stability and minimize any potential for migration of radionuclides, should they
escape their disposal containers. The method used to excavate the tunnels and shafts, and the rock
damage that occurs due to excavation, will greatly affect the performance characteristics of repository
sealing systems. The underground rock mechanics and geotechnical engineering work performed at
the Canadian Underground Research Laboratory (URL) has led to the development of excavation
technologies that reduce rock damage in subsurface excavations.

This paper discusses the excavation methods used to construct the URL and their application in
planning for the construction of similar underground laboratories and repositories for radioactive
wastes.

1 Introduction

Many technological advances have been made at Canada’s Underground Research Laboratory (URL)
in the fields of excavation methodology, excavation design, the modelling of rock response and the
characterization of the severity or rock damage around subsurface excavations. At the URL, damage
around the shaft and underground tunnels constructed in hard granitic rock was minimized through
controlled blasting techniques. These techniques reduced the amount of overbreak and fracturing
outside the design tunnel perimeter by reducing the charge-energy in the blastholes close to the final
walls. The techniques essentially eliminated overbreak and reduced fracturing around the shaft and
tunnels located in low-to-intermediate stressed, moderately fractured rock on the 240-m-deep level
and more highly stressed, sparsely fractured rock on the 420-m-deep level. Controlled blasting
resulted in better quality excavation with a smaller blast-induced Excavation-Damaged Zone (EDZ)
than conventional drill and blast methods. Geotechnical information acquired during site
characterization provided a good understanding of the in situ rock stress conditions. This information
was used to implement designs that reduce the stress-induced failure in the rock surrounding
excavations and the EDZ.
1.1 Geological Setting

The URL is sited in the Lac du Bonnet granite batholith, which is one of many late-tectonic granitic plutons intruded in the western Superior Province of the Canadian Shield late in the Kenoran event and has been age-dated as 2,680 ± 81 Ma. The batholith has an areal extent of 1,400 km² on surface (Figure 1) and extends in depth to between 6 and 25 km. The granite is pink near surface and surrounding fracture zones and a grey colour elsewhere. Naturally occurring micro cracks and foliation in the rock play a role in fracture formation (Everitt et al., 1996). The URL provides several stress regimes, (Martin, 1989) and degrees of fracturing in the same rock mass. The isometric diagram in Figures 1 and 2 shows the regions at the URL in which excavation work has been carried out.

According to the geological characterization carried out before and during URL construction, the number of fractures decreases with depth. The in situ stress distribution varies within the regions and is strongly influenced by the fracture zones also shown in the isometric diagram in Figure 1. The in situ stress was determined from 80 hydraulic fracturing tests and more than 1,000 overcore tests carried out since 1981. It was found that the horizontal stress magnitude increases almost linearly down to 270 m and then increases more rapidly below the fracture zone at the 270 m depth.

Generally, the granite rock at the URL has an elastic modulus of 55.3 GPa, a Poisson’s ratio of 0.18, a specific gravity of 2.67, a uniaxial compressive strength up to 200 MPa, and a tensile strength of 11.1 MPa. The rock mass quality is generally considered to be very good, having a Norwegian Geotechnical Institute (NGI) Tunnelling Quality Index Q between 50 and 250. The maximum principal stress at the 240-m-depth is in the order of 26 MPa oriented at an azimuth of 40° and a plunge of 8° from the horizontal. The intermediate and minor principal stresses at this depth are about 17 MPa and 13 MPa respectively, with the minor stress being sub-vertical. The magnitude and orientation of the rock stress change significantly in the sparsely fractured region below the 240 Level. At the 420 Level, the maximum principal stress is 60 MPa at an azimuth of 145° and a plunge of 11°. The intermediate and minor principal stresses at this depth are about 45 MPa and 11 MPa respectively.
2 URL Construction

The main underground excavations at the URL include a 443-m-deep shaft, major developments at the 240 and 420 Levels, shaft stations at the 130 and 300 Levels and two 1.8-m-diameter bored ventilation raises. Figure 2 is an isometric view of the URL. Shaft collar excavation and construction of the surface facilities took place between 1983 and 1984. Excavation of the 2.8 m by 4.9 m rectangular shaft to a depth of 255 m began on 1984 May and continued for the remainder of the year. The loop of horizontal excavations on the 240 Level and a bored ventilation raise to surface were completed by 1987. The shaft was extended with a circular profile (4.6-m diameter) to a depth of 443 m in 1988, followed by the excavation of the 420 Level and a ventilation raise to the 240 Level over the next three years.

Considerable time and effort were put into the preparation of relevant design specifications, development of appropriate contract formats and ongoing project management. Innovative equipment was utilized for construction activities and quality control procedures were implemented to ensure that data were reliable and made available to the research program during every step of construction. Research and construction activities were fully integrated to allow simultaneous collection of geotechnical information during excavation (Kuzyk et al., 1986).

2.1 Shaft Sinking

The shaft, sunk by a drill and blast excavation method in two phases, utilized a benching method of advance in the upper rectangular section and a full-face method in the lower circular section (Figure 3). During the first phase of sinking, it was found that the benching method did not adapt well to the controlled blasting method. The rectangular section proved to be a poor geometry because it resulted in stress concentrations and unacceptable damage at the corners. To improve on the quality of blasting, the shaft extension was excavated using a circular cross-section and a full-face method of advance. Controlled blasting principles were applied to the full-face blast designs. This change resulted in much less damage to the walls of the shaft and improved stability (Hagan et al, 1989, Kuzyk, Versluis, 1989).

In the circular shaft extension below the 255-m depth, two single-boom jumbos and a Cryderman mucker were set up on a Galloway sinking stage for drilling and removing broken rock. The Galloway stage (Figure 4) was designed to integrate both shaft sinking operations and the geotechnical characterization activities. One shift per day was dedicated to geotechnical characterization activities while the remaining two shifts were used for shaft sinking operations. Generally, 2.3-m-long full-face blast rounds were advanced.

2.1.1 Shaft Blast Design

Figure 5 shows the blast design for the full-face blast round used to excavate a circular shaft configuration. The blast design was based on blast holes being drilled in four concentric rings. A total of 73 blast holes were drilled, including six holes within the cut and 67 blast holes in the four rings. Blast holes in the inner three rings were charged with high-strength ammonia gelatin dynamite used in production blasting. The perimeter blast holes in the outer ring were charged with a blasting product formulated for perimeter blasting application. Both products were nitroglycerine-based explosives.

2.2 Level Development

A drill and blast method was employed to excavate most of the tunnels, test rooms and ramps needed for geotechnical projects on the 240 and 420 Levels of the URL. The method is flexible and cost
(a) Benching Method  (b) Full-Face Method

**Figure 3:** Shaft excavation methods used at the Canadian URL.

Figure 4: Galloway stage showing working decks, Cryderman mucker and drill jumbos.

(a) Geometry and delay allocation in cut and helper blastholes.  
(b) Delay allocation for concurrent Rings 1 to 4

**Figure 5:** Full-Face Blast design for the excavation of the circular shaft.
effective in hard, igneous rock and can ultimately be used to excavate a used nuclear fuel repository. To minimize damage to the final walls, controlled blasting principles were incorporated into the blast design. Quality control and inspection procedures were also deemed to be imperative. In the early stage of construction, the pilot and slash method shown in Figure 6(a) was applied.

This method was very effective and produced good results from an excavation damage perspective. However, the tunnel had to be advanced in two steps, first by excavating a pilot heading and then by following with a slash to create the final geometry. The typical advance per blast with this method was about 2.4 m. For improved performance a full-face blasting method, as shown in Figure 6(b), was applied in the later stages of construction. This eliminated the need to advance the tunnels in two steps. Although the full-face method was not as effective at reducing blast induced damage, the results were still very acceptable and much better than expected with the application of controlled blasting principles. The full-face method increased the advance per blast to 3.5 m.

2.2.1 Tunnel Blast Designs

The controlled drilling and blasting designs were optimized during the development of the tunnels on the 240 and 420 Levels to produce high quality walls with minimal fracturing and over break. For the most part, tunnels at the URL are relatively small, having dimensions of about 3.5 m wide by 3.3 m high. Tunnel profiles have curved crowns and inverts to reduce stress concentrations around the perimeter and minimize stress related failure.

About 65 blastholes are drilled in each round (Figure 7 shows a typical arrangement). A triangular-shaped cut is advantageous because the blastholes are more widely spaced. Three relief holes are drilled and reamed to a diameter of 89 mm or 100 mm at the centre of the face. The cut blastholes are positioned to reduce the effect of sympathetic detonation or dynamic pressure desensitization caused by the earlier firing charges. Blastholes, generally drilled 38-mm diameter, are charged with high-strength ammonia gelatin dynamite or pneumatically loaded ammonium nitrate explosive products. When charged with ammonium nitrate, blastholes are bottom initiated with an additional primer charge of dynamite to ensure a strong detonation front.

![Figure 6: Tunnel excavation at URL.](image)

![Figure 7: Typical tunnel blast design.](image)
Typically, 18 cushion and 21 perimeter blastholes are drilled. Low-shock-energy explosives and 11-mm-diameter high-strength detonating cord have been tested and found to be effective in the cushion and perimeter holes respectively, as fragmentation is primarily the result of energy provided by expanding gases. However, for ease of loading and other practical reasons, a product containing a blend of ammonium nitrate and formed polystyrene beads has been used in the cushion holes as well. Regular long-period shock tube detonators having periods from 0 to 18 are normally used but electronic detonators have also been tested and found to be very beneficial. The detonator delay sequence, charge mass per delay, burden and spacing are determined for specific blast patterns.

### 2.2.2 Long Blast Round Design

On the 240 Level, during the excavation of the tunnels shown in Figure 8, blast round lengths were increased to demonstrate that controlled blasting could be carried out in longer rounds as well. A total of six longer rounds were excavated (See Figure 2 for location). Rounds were increased in increments of about one metre to a final length of about 8.7 m (Kuzyk et al., 1994). For the most part blast designs for the longer rounds were consistent with those used for 3.5-m-long rounds, Figure 7 being typical. However, there were some exceptions. The drill pattern and charge configuration was adjusted to compensate for small changes in ground conditions.

![Figure 8: Schematic showing surveyed length of the long blast rounds](image)

The controlled blast designs produced a very high quality tunnel wall with minimal bootleg (blast hole remnants). Only minimal scaling of loose rock was required on the tunnel walls and crown. Ground support or protective screening was not required. Blast No. 216-05, shown in Figure 9, is typical of the performance of long blast round design. The blast broke a total surveyed length of 8.63 m, resulting in a blast-round-length to tunnel diameter ratio > 2.0. There were no bootlegs in the centre of the round, indicating 100% pull.

Fragmentation was very good, as can be seen from the photograph in Figure 10. The size of the muck was uniform, seldom exceeding 400 mm in size. The muck pile was level and evenly distributed. For the longer blast rounds, the muck-pile extended as much as 20 m from the face. The long uniformly sized muck-pile is very amenable to removal by continuous mucking systems. With the application of controlled blasting, a very high quality tunnel wall with minimal bootleg can be achieved.
3 Excavation Stability Study

An Excavation Stability Study (ESS) was undertaken to evaluate factors affecting tunnel stability and the extent of excavation damage in underground openings as a function of tunnel geometry and orientation, geology, and excavation method at the 420 Level of the URL (Figure 2 shows location). As noted earlier, the horizontal stress magnitudes and the ratio of maximum to minimum principal stresses at this depth are high relative to other levels of the URL. The stress conditions can be considered adverse in terms of excavation design. For example, a mechanically excavated circular tunnel oriented perpendicular to the principal stress direction experienced extensive ‘breakout-notch’ failure in the crown and invert Figure 11.

3.1 Design of the ESS Tunnels

The ESS comprised a series of tunnels having ovaloid, elliptical and circular profiles shown in Figure 12. A second circular tunnel, excavated by controlled drill and blast, provided comparison to the mechanically excavated circular tunnel noted above. Two- and three-dimensional numerical modelling was used to aid with the tunnel profile design. The peak compressive boundary stress and its extent along the tunnel crowns and inverts of each profile are shown in Figure 12. The tunnels had sections in both granite and granodiorite lithology, providing a comparison of damage in rock types with different strength characteristics.

Assessment of the damage in the ESS tunnels involved visual observation of failure and damage with respect to each excavation geometry, corelogging and geological mapping, borehole camera surveys and microvelocity probe (MVP) surveys in several borehole arrays at various locations in the ESS tunnels. The MVP was developed at Keele University to measure seismic velocity of the rock parallel to boreholes radiating from the walls and floor of the tunnels. Lower velocity signifies increased fracturing or damage. Convergence measurements in arrays comprising six pairs of diametrically-opposed pins installed in the tunnel walls at representative locations were used to determine the onset of non-linear / non-elastic behaviour, which is indicative of accumulated damage in the rock. Also, microseismic events identified with an array of acoustic emissions / microseismic (AE/MS) sensors were recorded and provided useful information regarding the timing and frequency of events and their
approximate location. The evaluation of the relationship between near-field stresses and displacement, damage and microvelocity variations was aided by the use of two- and three-dimensional numerical modelling (Read et al, 1997).

![Figure 11: Stress-induced ‘notch’ in the mechanically excavated circular tunnel.](image)

![Figure 12: Tunnel profiles designed for the ESS with maximum compressive boundary stress shown for each shape.](image)

### 3.2 Results of the ESS

Tunnel geometry and orientation influences the damage around the tunnel. By designing an optimal shape for a given stress field, the amount of damage can be reduced even in the most adverse stress conditions. However, it is extremely important to have a good understanding of the stress tensor in order to design the proper tunnel profile.

In the ESS, six of the ovaloid (one being elliptical) openings were excavated with major and minor axis aligned with $\sigma_{\text{max}}$ and $\sigma_{\text{min}}$ to minimize the peak tangential stress at the tunnel periphery for each aspect ratio. The maximum stress around the tunnels ranges from 100 to 169 MPa, thus some tunnels remained relatively stable with only minor amounts of slabbing, while break-out ‘notches’ tended to occur at more highly stressed locations. Reducing the peak stress magnitude and gradient at the tunnel periphery reduces the potential for damage localization, a necessary precursor for large-scale ‘notch’ development.

The most stable tunnel profiles (U1, U2 and U3 having aspect ratios shown in Figure 12) were excavated in the upper sub-level on the 420 Level to align with the maximum principal stress (11.1° from horizontal). These openings show some minor spalling across a broad area of the crown and invert as the tunnel profile adjusts to the near-field stress conditions (Figure 13). The spalling initiates at perturbations in the excavated profile (e.g., resulting from drill misalignment or the ‘look-outs’ required for the drill and blast method). This spalling process occurs primarily within a few hours to a few days after blasting and generally smooths out the tunnel profile.

From array borehole observations, the visible EDZ around the various profiles was limited to a thin skin ranging from 0 to 80-mm thick in the crown and sidewalls and 210 mm in the invert. Damage leading to ‘notch’ development was limited to the blast rounds in the circular tunnel.

Excavation damage is more apparent in inequigranular coarser-grained rock (such as granite) than in equigranular finer-grained rock (such as granodiorite).
The excavation method influences the development of damage. However, when compared to mechanical methods, the only notable effect of the controlled drill and blast excavation method appeared to be to increase the extent of macroscopic damage in the tensile regions or regions of stress reduction around underground openings.

![Excavated profile segments aligned with the direction of σ₁ and σ₂.](image)

**Figure 13:** Excavated profile segments aligned with the direction of σ₁ and σ₂.

### 4 Blast Damage Assessment

A Blast Damage Assessment (BDA) study was carried out on the 240 Level of the URL (Figure 2 shows location) to evaluate the excavation disturbed and damage zones in tunnels excavated under relatively low *in situ* stress conditions (Martino, 2002). Regular blast round lengths of about 3.5 m were excavated. The tunnel layout was elliptical in cross-section with dimensions of 4.4-m wide by 3.5-m high.

In the BDA, the excavation disturbed and damaged zones were characterized by drilling radial borehole arrays similar to those employed in the ESS for monitoring damage at locations of interest. A micro-seismic velocity probe (MVP) was again used to conduct seismic characterization of the rock mass surrounding the tunnel.

To provide information concerning geological structure, core produced from diamond drilling of the array boreholes was examined and logged. The core was orientated upon recovery from the borehole so the *in situ* orientation of fractures and other features could be recorded. In addition to monitoring in array boreholes, surface mapping of the tunnel walls provided an overall understanding of fracture pattern and structure in relation to activities during the excavation of the tunnel. However, surface mapping provides limited information on the depth of cracking.

#### 4.1 Characterizing the EDZ

During excavation of underground opening, regions of altered stress will develop around the opening. Elastic analysis shows these regions extend out to five times the radius of a circular tunnel. Figure 14 shows schematically the four components of the altered stress zone around a tunnel excavated in stressed rock at the URL.
The excavation-disturbed zone extends from 2 to 5 tunnel radii from the wall, where the stress changes are smaller and the re-distributed stresses have not measurably changed the rock properties. The excavation-disturbed zone may also have decreased hydraulic pressure due to drawdown of groundwater resulting from drainage into the tunnel. If the excavation disturbed zones of two adjacent tunnels overlap, the combined effect may influence the development of the disturbed and damaged zones around either or both openings.

The Excavation Damage Zone (EDZ) can be divided into failed and non-failed zones. Failed zones exist where rock has physically detached from the opening wall. The non-failed zone of the EDZ can be divided into inner and outer zones. The inner zone is characterized by sharp changes in permeability and seismic velocity and is where most visible excavation-induced cracks exist. The outer zone shows a more gradual change in these properties as characteristics return to background values (Read 1996, Martino and Chandler, 2004).

4.2 Results of the BDA
The extent of the blast damage measured in arrays located on the 240 Level during the BDA study is shown in Figure 16. Analysis of results indicated that there should be no stress induced-failure around the BDA tunnel. Thus, blasting is the main cause of the damage. In the upper part of the BDA tunnel, the outer zone is
more predominant than at other places around the tunnel, suggesting that gravity may be acting to open some micro-cracks and reduce the seismic velocity in the rock near the tunnel crown.

The typical background S-wave velocity is 3,200 m/s around the BDA tunnel, with velocity decreases of up to 400 m/s in the upper right and lower left of the tunnel. P-wave velocities around the BDA tunnel had a background velocity of 5,400-5,600 m/s. Velocity decreases ranged from 1,300 m/s in the lower left quadrant to 400 m/s in the upper left quadrant. Hydraulic conductivity was calculated at 10-8 m/s with a hydraulic conductivity of 10-13 to 10-14 m/s outside the EDZ. Hydraulic conductivity is defined as the ratio of flow velocity to driving force for a saturated porous medium.

Geophysical surveys in array boreholes carried out during earlier controlled blasting tests (Potier et al., 1992) measured lower seismic (P-wave) velocities indicating increased damage to depths of only 0.15 to 0.30 m. This suggested that the damage zone is less than 0.30 m from the rock surface. Other work showed that damage extends well beyond 1.0 m if controlled blasting is not used (Everitt et al., 2003).

Figure 15: Excavation disturbed and damaged zones. Figure 16: The inner and outer damage zones (Martino, 2002).

5 Conclusions

The construction of an underground research laboratory provides an excellent opportunity to demonstrate suitable excavation techniques that can be applied to the construction of a repository for nuclear fuel waste disposal. To improve long-term stability and safety for workers during the operation of these facilities, rock damage by blasting can be minimized. When constructing a repository, it will be beneficial to minimize fractures surrounding the emplacement rooms, as they are potential pathways for the hydraulic transport of radionuclides. Adapting controlled drilling and blasting principles can help achieve this objective.

The robustness (versatility and adaptability) of the controlled drilling and blasting method has been demonstrated during in shaft sinking and tunnelling at the Canadian URL. The long tunnel blast round demonstrations have shown that the blast round advance can be increased up to 8 m without loosing quality of performance. A very high quality tunnel wall with almost 100% pull can be achieved with long blast rounds. Rock fragmentation was also demonstrated to be very satisfactory.

The Excavation Stability Study demonstrated design criteria for achieving mechanically stable openings excavated in the most adverse stress conditions at the URL. The study showed that tunnel stability is sensitive to tunnel shape, variations in geology and, to a lesser extent, the excavation method.

Excavation damage assessments carried out in the Blast Damage Assessment study demonstrated that the inner zone of damage characterized by sharp changes in permeability and the seismic velocities can be significantly reduced by controlled drilling and blasting.
The results of the blasting methods demonstrated at the Canadian URL are dependent on the geological conditions, such as fracturing, *in situ* stresses, and rock properties. These results cannot be guaranteed by applying the same blast designs in the rock mass planned for other underground facilities. It is, therefore, highly recommended that appropriate blasting techniques be developed for specific *in situ* conditions using a similar approach adapted at the Canadian URL.
References:


