



Ground support strategies to control large deformations in mining excavations

by Y. Potvin* and J. Hadjigeorgiou†

Synopsis

Tunnelling under squeezing ground conditions poses significant challenges. There are fundamental differences in the choice of support for mining as opposed to civil engineering. This paper reviews ground support strategies that have been used to control large deformations in mining excavations based on field data from five different mines in Australia and Canada. Recommendations are made for improved ground control strategies in squeezing rock conditions.

Introduction

Large-scale deformations, often referred to as squeezing conditions, pose a considerable challenge in the construction and maintenance of underground excavations in rock. Although a variety of failure mechanisms are possible, it is generally accepted that squeezing conditions imply a reduction in the cross-sectional area of an excavation as a result of a combination of induced stresses and relatively weak material properties.

A working definition of squeezing rock has been provided by the International Society for Rock Mechanics (ISRM): 'Squeezing of rock is the time dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may terminate during construction or continue over a long-time period', Barla (1995). Although most of the attention has focused on the construction of transportation tunnels, Aydan *et al.* (1993), Steiner (1996), Barla (2007) etc., several underground mines have to deal with squeezing rock conditions. This paper draws from recent work of a Task Force on Squeezing Rock in Australian and Canadian mines.

There are important differences in the choice of support strategies for squeezing rock conditions between mining and civil engineering. In civil engineering we have access to a range of support systems that can arguably manage ground deformation during

and after the construction phase of tunnelling. Use of some of these systems in a mining environment, however, is prohibitively expensive and would involve considerable delays in development and production mining. Other important differences between civil and mining projects in squeezing rock conditions include the service life of excavations, desired rate of advancement and convergence tolerance limits.

Tunnelling under squeezing rock conditions

Terzaghi (1946) provides one of the earliest definitions of squeezing rock behaviour in tunnelling: 'Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite of squeeze is a high percentage of micaceous minerals or of clay minerals with a low swelling capacity.' Furthermore, he distinguishes between squeezing rock at moderate depth and squeezing rock at great depth to provide estimates of the resulting rock loads on the roof of tunnels.

Barton *et al.* (1974) have defined squeezing rock as 'plastic flow of incompetent rock under the influence of high pressure.' Squeezing rock conditions are part of the stress reduction factor (SRF) in the Q system whereby 'mild squeezing pressure' results in an SRF rating within 5–10 and 'heavy squeezing rock pressure' is given a value ranging from 10–20. It follows that the presence of squeezing conditions results in a reduction in the Q rating of a rock mass.

* Australian Centre for Geomechanics.

† Université Laval.

© The Southern African Institute of Mining and Metallurgy, 2008. SA ISSN 0038-223X/3.00 + 0.00. Paper received Apr. 2008; revised paper received Jun. 2008.

Ground support strategies to control large deformations in mining excavations

Phenomenological observations of squeezing

Aydan *et al.* (1993) provide a phenomenological description of squeezing in rocks by distinguishing between three failure mechanisms:

- Complete shear failure: this involves the complete process of shearing of the medium
- Buckling failure: this type of failure being generally observed in metamorphic rocks (i.e. phyllite, mica schists,) and thinly bedded ductile sedimentary rocks (i.e. mudstone shale, siltstone, sandstone, evaporitic rocks)
- Shearing and sliding failure: observed in relatively thickly bedded sedimentary rocks and it involves sliding along bedding planes and shearing of intact rock.

Theoretical criteria for squeezing rock

It is recognized that squeezing conditions are associated with high stresses and weak rock masses. In the Q system for example, Barton *et al.* (1974) suggest that the ratio of maximum tangential stress (calculated from elastic theory) to unconfined compression strength (σ_θ/σ_c) can be used to define squeezing rock pressure. Singh *et al.* (1992) suggest that a necessary condition for squeezing rock conditions is that:

$$\sigma_\theta > q_c$$

where σ_θ is the tangential stress and q_c is the uniaxial crushing strength of the rock mass.

Aydan *et al.* (1993) used strain, as opposed to rock mass strength, to define the squeezing potential of a rock mass. They argued that there is an analogy between the axial stress-strain behaviour of rocks under laboratory conditions and tangential stress-response of rock surrounding excavations. They identified five states in an idealized stress-strain curve: (1) elastic; (2) hardening; (3) yielding; (4) weakening and (5) flowing, Figure 2. The normalized strain levels η_p , η_s , η_f were defined as:

$$\eta_p = \epsilon_p / \epsilon_e; \eta_s = \epsilon_s / \epsilon_e; \eta_f = \epsilon_f / \epsilon_e;$$

It is then possible to use the ratio of peak tangential strain at the circumference of the tunnel (ϵ_θ^a) to elastic strain (ϵ_e^e) to define various degrees of squeezing. (see Figure 2.)

It is advantageous to use strain rather than the strength of the rock mass as a design criterion since it is easier to measure *in situ* deformations. Hoek (2001) demonstrated that recorded strain could be used as a tool to predict squeezing potential. Based on the results of parametric finite element models he developed approximate relationships for the strain of a tunnel and the ratio of support pressure to *in situ* stress. This information was used to provide a 'first estimate' of tunnel squeezing problems, Figure 3.

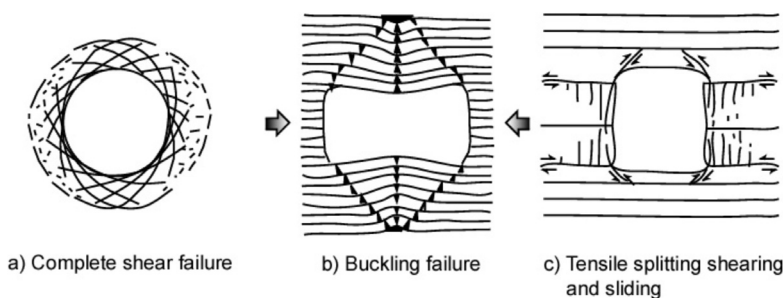


Figure 1—Classification of failure forms of tunnels in squeezing rocks, after Aydan *et al.* (1993)

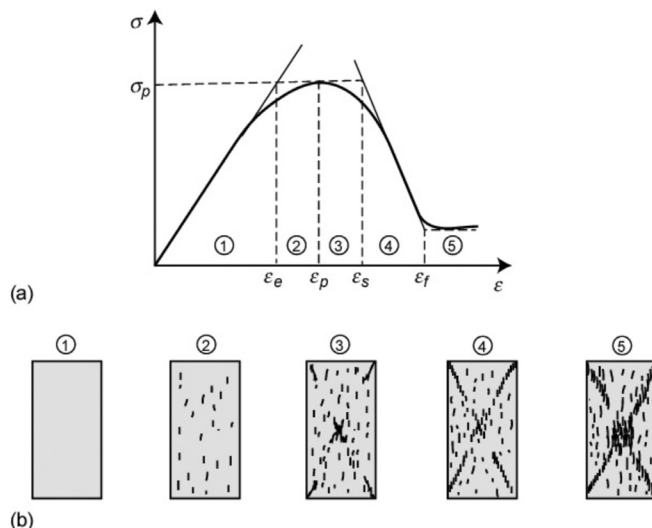


Figure 2—Idealized stress-strain curves and associated states for squeezing rocks, after Aydan *et al.* (1993)

Ground support strategies to control large deformations in mining excavations

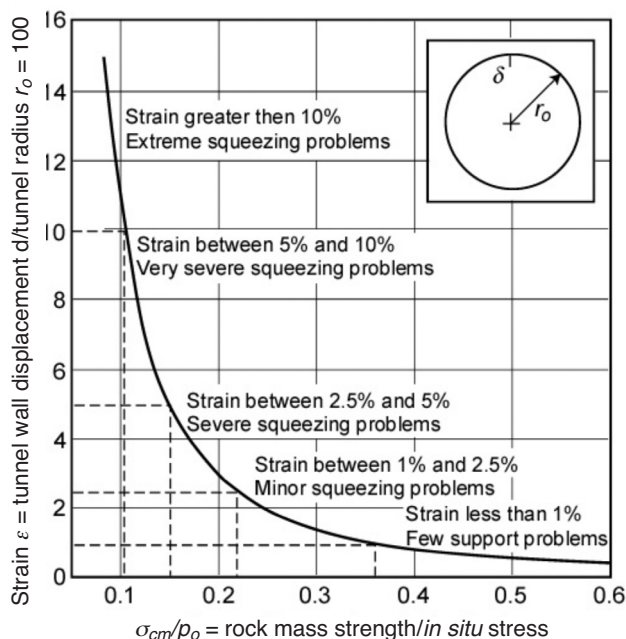


Figure 3—Tunnelling problems associated with different levels of strain (after Hoek, 2001)

More recently, Singh *et al.* (2007) suggest that there is in fact a critical strain beyond which squeezing problems may be encountered during construction. This was defined as the tangential strain level at a point on the opening periphery. Rather than setting this strain as 1%, based on current experience, they suggest that this can be calculated based on the orientated intact rock properties and the *in situ* modulus of deformation. This approach can then lead to a squeezing index (SI) defined as:

$$SI = \frac{\text{Observed or expected strain}}{\text{Critical strain}} = \frac{u_r / a}{\epsilon_{cr}}$$

where u_r is the radial closure and a is the radius of the opening.

Table I summarizes the squeezing level classifications proposed by Aydan *et al.* (1993), Hoek (2001) and Singh *et al.* (2007). It is noted that Singh *et al.* (2007) use the same squeezing level descriptions as originally proposed by Aydan *et al.* (1993).

Empirical criterion for squeezing

Barton *et al.* (1974) define mild squeezing rock pressure

when (σ_θ/σ_c) is between 1 and 5, and heavy squeezing rock pressure when (σ_θ/σ_c) is greater than 5. Barton (2000) proposed that the rock mass compressive strength (σ_{cm}) can be estimated from:

$$\sigma_{cm} \approx \gamma Q_c^{1/3} \quad (\text{MPa})$$

where γ is the rock density (t/m^3) and Q_c is determined from $Q_c = \frac{Q\sigma_c}{100}$.

Singh *et al.* (1992) have complemented some of the field data used in the development of the Q system by Barton *et al.* (1974) to propose a direct relationship between the depth of a tunnel and Q. They suggested that a necessary condition for squeezing rock in tunnelling is that:

$$H > 350Q^{1/3}$$

where H is the overburden (m).

Mining under squeezing rock conditions

Case studies of squeezing rock conditions have been presented by Potvin and Slade (2007), Mercier-Langevin and Turcotte (2007) and Sandy *et al.* (2007). In this work a preliminary benchmark was established whereby squeezing ground was defined as one experiencing closure greater than 10 cm during the life expectancy of a supported drive, generally up to 18 months to 2 years. Considering that most mechanized mining accesses are in the order of 5 m x 5 m, a 10 cm closure corresponds to a strain of approximately 2%. This is in close agreement with the three criteria proposed from the civil engineering literature summarized in Table I. It corresponds to the upper limit of class 2 (minor squeezing problems). Squeezing ground conditions in mines are characterized by significant failure of ground support systems that often necessitates considerable rehabilitation work.

The discussion in this section is based on site visits at five mining operations in Australia and Canada experiencing squeezing ground conditions.

Necessary conditions

It is recognized that in all mines experiencing squeezing ground conditions there was a prominent structural feature present. This could be a dominant fracture set, intense foliation or a shear zone. The stress field was also considerably high. Furthermore, the host rock type, where squeezing was most dominant, was characterized by weak intact rock strength (less than 60 MPa). Although these two conditions appear to be necessary, the degree of squeezing has been observed to increase in the presence of localized

Class #	Hoek (2001)		Aydan <i>et al.</i> (1993)		Singh <i>et al.</i> (2007)	
	Squeezing level	Tunnel strain	Squeezing level	Tunnel strain	Squeezing level	SI
1	Few support problems	$\epsilon_t < 1\%$	No squeezing	$\epsilon_\theta^a / \epsilon_\theta^o \leq 1$	No squeezing	$SI < 1.0$
2	Minor squeezing problems	$1\% < \epsilon_t < 2.5\%$	Light	$1 \leq \epsilon_\theta^a / \epsilon_\theta^o \leq 2.0$	Light	$1.0 < SI \leq 2.0$
3	Severe squeezing problem	$2.5\% < \epsilon_t < 5\%$	Fair squeezing	$2.0 \leq \epsilon_\theta^a / \epsilon_\theta^o \leq 3.0$	Fair squeezing	$2.0 < SI \leq 3.0$
4	Very severe squeezing problem	$5\% < \epsilon_t < 10\%$	Heavy squeezing	$3.0 \leq \epsilon_\theta^a / \epsilon_\theta^o \leq 5.0$	Heavy squeezing	$31.0 < SI \leq 5.0$
5	Extreme squeezing problem	$\epsilon_t > 10\%$	Very heavy squeezing	$\epsilon_\theta^a / \epsilon_\theta^o > 5.0$	Very heavy squeezing	$5.0 < SI$

Ground support strategies to control large deformations in mining excavations

alteration such as mica, chlorite, and tochilinite. The presence of alteration results in much weaker intact rock strength (less than 10 MPa). Another contributing factor to large-scale deformation is orientating the excavation parallel to the dominant structural feature (foliation, etc.).

Phenomenological failure mechanisms

The bulging or buckling of rock layers parallel to the lower part of the footwall is often first noticed. Floor heave is also often associated with this initial movement. Alternatively or in addition, the movement can also be in the higher part of the hangingwall. Essentially, if the weak rock mass and/or 'key' geological feature is in the footwall, the movement will be, as first described, in the lower footwall side. If the weak rock mass is in the hangingwall, the movement will be in the upper hangingwall side. And if the drive is entirely located in the weak rock mass, both mechanisms may occur simultaneously. An interpretation of these mechanisms, including the driving force trajectories, is given in Figure 4.

An interesting departure in behaviour has been the recognition that the thickness of foliation layers had a direct impact on the rate of convergence but also on the performance of support systems. A first distinction made was that thin foliation layers as observed in three of the benchmarked mines lead to heavy squeezing while the thicker layers present at the two other mines resulted in moderate squeezing. Table II proposes two categories of mining squeezing ground, with some of the observed characteristics relevant to each category.

Category 1 squeezing ground can experience closure greater than 2 m (40% strain) over a short period of time (few months). This is clearly in the upper limit of the 'Class 5, extreme squeezing ground' from Table I, which suggests

strain greater than 10%. The Category 2 would correspond approximately to a Class 4 squeezing ground in Table I (5 to 10% strain).

Where the layers are thin, the rock may shear around the reinforcement. The bulging of the rock mass is rapid and extensive and high rates of convergence are observed, especially just after the cut is taken, Figure 5. The depth of the heavily broken material can reach several metres.

Where the layers are thicker, the reinforcement is more susceptible to shear or even guillotined. The bulging movement observed starts with the opening of joints followed by a relatively slow rotation of large blocks inside the drive (lifting of the footwall layers near the floor, Figure 6, or toppling of the hangingwall layers near the shoulder, Figure 7. The depth of failure is generally within approximately 3 metres inside the walls. Fundamentally, the failure mechanism is the same, but the shearing effect on reinforcement is accentuated, the rate of failures slower and the appearance of the deformed wall can be significantly different.

Support under squeezing rock conditions

Mining experience has shown that it is not a realistic option to stop deformation in squeezing ground. It has been demonstrated that such an approach results in frequent rehabilitation and high support costs. This has now resulted in most mines pursuing a modified support strategy whereby the objective becomes one of controlling rather than arresting the degree of squeezing.

An effective ground support system for squeezing ground employs both reinforcement and surface support units. Tendon reinforcement is required to maintain the self-

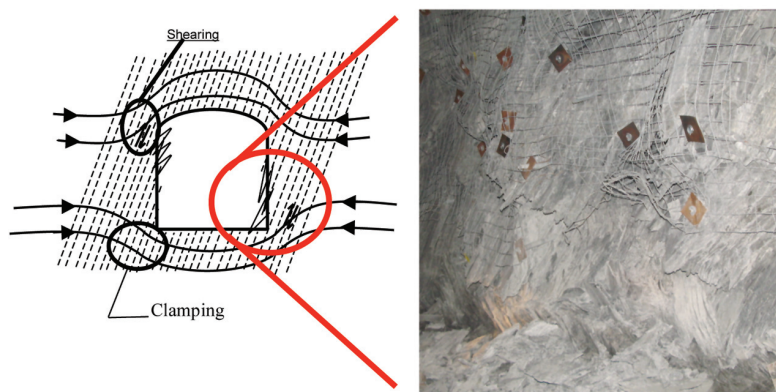


Figure 4—High horizontal stresses create shearing in the top left and bottom right corners and clamping in the top right and bottom left corners

Category	Category 1	Category 2
Squeezing level	Heavy squeezing	Moderate squeezing
Rock layers	Thin (10s of millimetres)	Thick (10s of centimetres)
UCS	Approximately 10 MPa	Approximately 50 MPa
Maximum rate of convergence	100s of mm per month	10s mm per month
Depth of broken rock mass	Up to 6 m	Up to 3 m

Ground support strategies to control large deformations in mining excavations



Figure 5—Examples of floor heaving and extensive bulging of the wall: (a) 6 May, 2004 (b) 25 September, 2004



Figure 6—Footwall slabs rotating upwards



Figure 7—Hanging wall slabs buckling (after DeRoss, 2007)

supporting capacity of the rock mass surrounding the excavation. Furthermore, it is intended to keep together the reinforced rock mass unit around each tendon and mitigate the rate of convergence of the drive. The reinforced volume usually reaches from 1.8 to 2.5 m inside the tunnel wall, depending on the bolt length. The zone of reinforced rock mass is referred to in this paper as ‘the reinforced shell’ around the excavation.

There are clear advantages in employing tight (high density) reinforcement patterns as they result in a stronger reinforced shell and more uniform surface deformations. This minimizes localized surface support failures. Surface support elements aim at containing the immediate rock mass surrounding the excavation. In squeezing rock conditions, successful surface support should deform to accommodate the resulting high deformations while maintaining the integrity of the reinforced rock unit.

Retainment

Prompt application of shotcrete prevents or retards rock mass degradation and keeps the rock mass together. A drawback of shotcrete in squeezing conditions is the high stiffness of the liner. Use of fibrecrrete (shotcrete with fibres) can cater only for relatively small deformations and soon becomes ineffective at deformations greater than a couple of centimetres. Several mines have consequently abandoned the use of fibrecrrete (alone) as a suitable support in squeezing rock conditions.

Welded mesh is a passive support system. It does not prevent or retard rock mass degradation but it can retain broken rock confinement and can deform considerably before failing. Another practical concern is that mesh failures often occur along the overlap where mesh sheets are joined together. This can be addressed by reinforcing the overlap area using zero gauge mesh straps. The main disadvantage

Ground support strategies to control large deformations in mining excavations

of mesh is its low overall strength capacity. This can be compensated (at least partially) by using a tight reinforcement pattern resulting in smaller exposed surface between bolts. Another advantage of a tight reinforcement strategy is that it results in more uniform wall deformation and limits localized excessive stretching of mesh. The LaRonde Mine has successfully implemented the use of mesh in combination with a relatively high density of reinforcement for squeezing rock conditions.

Several mines have also explored the use of mesh embedded in between two layers of shotcrete or fibrecrete. This results in a stiff system that provides support at relatively small-scale deformations. Unfortunately shotcrete will crack at low level deformation. Consequently, several mines experiencing squeezing rock conditions are moving away from this support practice as it results in disproportionate and difficult rehabilitation. Another problem associated with shotcrete embedded mesh, which is more ductile than fibrecrete, is the outside layer of shotcrete failing in large slabs.

Australian mines, currently operating in squeezing rock conditions, favour the use of fibre reinforced shotcrete, installed 'in-cycle', and then applying mesh on top. The resulting surface support system, which keeps the rock mass together, is initially stiff until the shotcrete cracks and then acts as a soft system with the mesh containing the large shotcrete plates produced by the excessive wall deformation, Figure 8. The main drawback of this surface support system is its high cost. This, however, can become acceptable if it can be demonstrated that it can significantly reduce rehabilitation work.

Reinforcement

In squeezing rock conditions the failed zone can extend several metres behind the walls of an excavation. Consequently, it is possible that a reinforcement unit may be entirely contained within the failed rock mass. Under these conditions the aim is no longer to pin unstable blocks, but rather to provide some degree of confinement within the broken rock mass and to create a 'reinforced shell'. As stiff bolts, such as fully grouted resin bars, cannot accommodate



Figure 8—Fibre-reinforced shotcrete cracks due to large deformation and the large plates formed are retained by mesh

large deformations, they tend to snap and break. This has led to the use of reinforcement units that can yield or display ductile behaviour. This ductility can be achieved by stretching of the tendon in partially debonded bolts or sliding between the tendon and the rock mass in friction bolts. The amount of elongation that can be obtained from stretching a bolt is a function of the length of the debonded section and the elastic modulus of the steel. This can be a limitation in highly deformable ground (squeezing ground Category 1). However, in addition to stretching, there is probably some slippage and further debonding happening. The use of debonded rebars has been shown to perform quite well at some of the benchmarked mines.

Squeezing rock is often associated with heavy shearing. Most reinforcement units do not perform well when submitted to heavy shearing as they tend to bend and lock the sliding mechanism. This phenomenon has been observed *in situ* for both split sets and cone bolts, Figure 9.

In squeezing ground, the loss of bolt heads is very common as the reinforcement units often cannot tolerate large deformations and fail at the weakest point, which is the bolt head. Split sets in particular display a strong tendency to lose their bolt rings. This is accentuated when split sets are installed at an angle using a jumbo. This results in preferential loading on one side of the bolt ring (point loading) and premature failure. Another contributing factor to ring failure is caused by 'over hammering' of the bolt. This common quality control problem results in a weakened split set ring that fails rapidly when required to accommodate large deformations.

Ground in shear locks the sliding mechanism and can also guillotine thin wall bolts such as split sets and Swellex. It is recognized that under these conditions, solid bar rock bolts provide a greater resistance to shear. On the other hand, ground in shear can inhibit a solid bolt capacity to deform, slide and yield. Mercier-Langevin and Turcotte (2007) report some success with a hybrid bolt developed for squeezing ground conditions at LaRonde. The bolt consists of a resin rebar installed inside a friction bolt that acts as a sleeve for the resin rebar. In fractured ground this configuration prevents the resin from escaping. It results in greater resistance to shear while also increasing the frictional resistance of the friction bolt. Furthermore, this configuration provides a stronger head to the bolt. *In situ* and laboratory

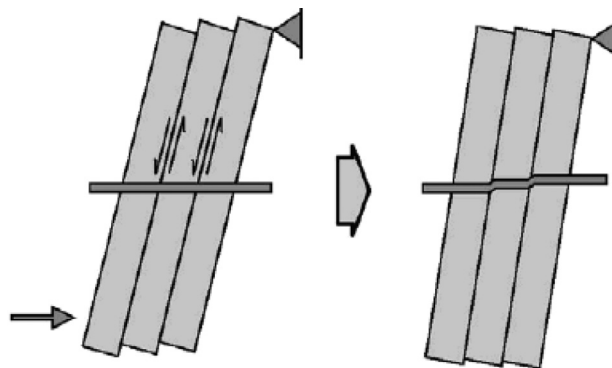


Figure 9—Shearing along foliation that may result in either direct shear failure of the tendons or 'locking' of the bar, which then fails in tension, after Mercier-Langevin and Turcotte (2007)

Ground support strategies to control large deformations in mining excavations

pull testing on the hybrid bolts have shown a nearly ideal behaviour, with a stiff early reaction at low displacement followed by almost plastic behaviour under higher loads (15 to 20 tons), Figure 10.

Support system performance

Table III summarizes current support practice at five mines operating under squeezing conditions. It is recognized that the selection of ground support in squeezing ground is still evolving and this will be revised. The evolution of support systems at Perseverance has been summarized by Tyler and Werner (2004). Furthermore, as operations go deeper, even under similar geological conditions the degree of squeezing is expected to increase, necessitating further modifications to the support strategy.

Potential gains in support system performance

In heavy squeezing conditions (category 1) increased performance can arguably be achieved by improving coverage at the lower walls of the excavations. This would involve installing support 'floor to floor'. This approach would be similar to civil engineering applications where 'floor to floor' is often the minimum standard. In fact, in certain projects it is necessary to install a steel culvert on the floor. Another area of potential improvement is to strengthen the reinforced shell of broken rock. This can be achieved with a higher density of bolts, and/or with 'better' performing bolts. The hybrid bolt as used at LaRonde seems to provide such an option.

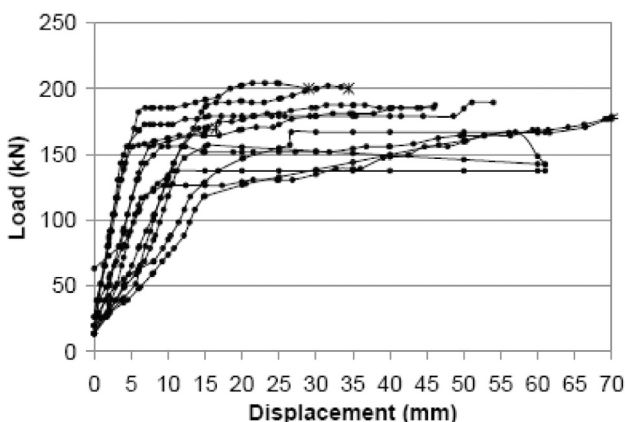


Figure 10—Results from *in situ* pull tests on hybrid bolts (after Mercier-Langevin and Turcotte, 2007)

In moderately squeezing ground (category 2), two of the mines used in this benchmark study used an initially stiff surface support (in-cycle layer of fibrecrete (50 to 75 mm thick), which begins to yield once the fibrecrete cracks. This surface support was combined with a soft reinforcement (2.4 m long 46 mm split sets, at a low density of 1.3 × 1.5 m) and complemented by 6 m twin strands cable bolts, on a very wide pattern. The potential advantages of using fibrecrete in this type of conditions should be explored further.

A support system is only as strong as its weakest point. It is highly desirable that the reinforcement works in unison with the surface support. Maintaining the connection between the two is critical. Straps and mesh-straps are particularly efficient to improve the connection between the reinforcement and the surface support. As the excavation surface deforms, it pulls on the straps and the load is distributed along the whole strap and transferred to all the bolts connected to the strap. In addition to its effect of creating a shell of reinforced broken rock mass, the reinforcement can then offer some extra resistance to surface deformation and slow down convergence.

A common recommendation in civil engineering tunnels in squeezing rock is to construct and calibrate numerical models based on *in situ* monitoring. Unfortunately relatively little monitoring is being used at mines experiencing squeezing ground conditions. Mines that have done extensive convergence measurements have often not captured the early deformation period, when the convergence rate is very high. As a result, a total quantified 'picture' of the deformation does not exist at any of the benchmarked mines. This is an area that is currently being addressed by one of the mines.

Conclusions

It is recognized that there is no unique solution to controlling large-scale deformations in rock. Of interest, however, is the clear dichotomy between Australian and Canadian ground support practices in the use of surface support. Australian mines use an excessive amount of fibrecrete, while Canadian operations rely primarily on weld-mesh, sometimes complemented with mesh straps. In Australian mines it is thought that in-cycle shotcrete can retard rock mass degradation and control large deformations. Using a thick layer of shotcrete, however, results in a support that is too stiff for squeezing ground, often requiring a further layer of mesh and thus creating a composite liner that is initially stiff followed by a ductile behaviour (after the fibrecrete has cracked).

Table III

Support practice at mines operating under squeezing rock conditions

Category 1: Heavy squeezing		Category 2: Moderate squeezing	
Thin layers (10s of millimetres). UCS approx. 10 MPa. 100's of mm closure per month. Depth of broken rock mass up to 6 m.		Thick layers (10s of centimetres). UCS approx. 50 MPa. 10s mm closure per month. Depth of broken rock mass up to 3 m	
Mine site	Support	Mine	Support
Mine #1	Split sets (1 x 1 m); 75 mm fibrecrete + weld mesh	Mine #4	Split sets and cable bolts (1.3 x 1.5 m); 50 to 75 mm fibrecrete + weld-mesh
Mine #2	Debonded bar and Swellex (1 x 1.5 m); 75 mm fibrecrete + weld mesh + 50 mm fibrecrete	Mine #5	Split sets, resin bar and cable bolts (50 to 75 mm); fibrecrete + weld-mesh
Mine #3	Hybrid bolts (0.8 x 0.8 m); weld-mesh		

Ground support strategies to control large deformations in mining excavations

Another difference in approach is how Australian mines favour a relatively soft reinforcement shell, using a wide spacing of split sets. This is in contrast to Canadian experience where a stiffer shell can be obtained by using higher density of bolts with higher capacity, but with yielding capability (such as Swellex or hybrid bolt). This has allowed LaRonde to control high deformation in squeezing ground without using shotcrete.

Acknowledgements

This paper draws from experience gained during the Squeezing Ground Task Force. The input of personnel from Maggie Hays, Black Swan Nickel, Waroonga and Perseverance is gratefully acknowledged. The opinions expressed in the paper are those of the authors and do not necessarily reflect the opinions of participating mines.

References

- AYDAN, Ö. AKAGI, T., and KAWAMOTO, T. The squeezing potential of rock around tunnels: theory and prediction. *Rock Mechanics and Rock Engineering*, vol. 2, 1993. pp. 137–163.
- BARLA, G. Squeezing rocks in tunnels. *ISRM News Journal*, 3/4, 1995. pp. 44–49.
- BARLA, G., BONINI, M., and DEBERNARDI, D. Modelling of tunnels in squeezing rock. *3rd Iranian Rock Mechanics Conference*, Oct. 2007. pp. 1267–1285.
- BARTON, N., LIEN, R., and LUNDE, J. Engineering classification of rock masses for the design of tunnel supports. *Rock Mechanics*, vol. 6, no. 4, 1974. pp. 189–239.
- BARTON, N. *TBM Tunneling in Jointed and Faulted Rock*, 2000. pp. 173.
- DEROSS, J. 10130 Goose RHS Drive Closure Investigation, Unpublished. Internal Memorandum, Black Swan Nickel Operation, Norilsk Nickel Australia. 2007.
- HOEK, E. Big tunnels in bad rock. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, vol. 127, no. 9. September 2001. pp. 726–740.
- MERCIER-LANGEVIN, F. and TURCOTTE, P. Evolution of ground support practices at Agnico Eagle's LaRonde Division: innovative solutions to high stress ground. *Rock Mechanics: Meeting Society's Challenges and Demands*. Eberhardt, Stead Morrison (eds.). 2007. pp. 1497–1504.
- POTVIN, Y. and SLADE, N. Controlling extreme ground deformation; Learning from four Australian case studies. *Challenges in Deep and High Stress Mining*. Y. Potvin, J. Hadjigeorgiou and D. Stacey (eds.), Chapter 40, 2007. pp. 355–361.
- SINGH, B., JETHWA, J.L., DUBE, A.K., and SINGH, B. Correlation between observed support pressure and rock mass quality. *Tunnelling and Underground Space Technology*, vol. 7, 1992. pp. 59–74.
- SANDY, M.P., GIBSON, W., and GAUDREAU, D. Canadian and Australian ground support practices in high deformation environments. *Deep Mining 07*, Y. Potvin (ed.). Australian Centre for Geomechanics, 2007. pp. 297–311.
- SINGH, M., SINGH, B., and CHOUDHARI, J. Critical strain and squeezing of rock mass in tunnels. *Tunnelling and Underground Space Technology*, vol. 22, 2007. pp. 343–350.
- STEINER, W. Tunnelling in squeezing rocks: case histories. *Rock Mech. Rock Engng.* 1996, vol. 29, no. 4. pp. 211–246.
- TERZAGHI, K. Rock defects and loads in tunnel supports, *Rock tunneling with steel supports*. R.V. Proctor and T.L. White (eds.), The Commercial Shearing and Stamping Co., Youngstown, Ohio, 1946. pp. 17–99.
- TYLER, D.B. and WERNER, M. A case study of ground support improvement at Perseverance Mine. *Ground Support in Mining and Underground Construction*. Villaescusa and Potvin (eds), 2004. pp. 53–63. ◆

FOR SALE

Two x 20 MW DIESEL POWER GENERATORS (for sale in India)

Man B&W D.G. Sets: Fitted with Siemens Alternator (German make)
Type-9L 58/64, Capacity 10 MW each
RPM-428, Rated BHP-14246
Fuel recommended: L.S.H.S / H.F.O
Hours run: 50,000 hrs approx.
Plant situated in Orissa, India.

Contact Details:

The General Manager
Tel: +91 6784-240272/240345/240320
Fax: +91 6784-240626
Email: facorccp@dataone.in
bls_facorccp@bsnl.in
facorccp@gmail.com